DESIGN OF

URBAN DRAINAGE SYSTEMS

Notes for an Educational Course

presented by

Stormwater Consultants

PART B

DESIGN OF URBAN DRAINAGE SYSTEMS

A Professional Development Course
Presented and Published by

STORMWATER CONSULTANTS
3 Westchester Court
Bolingbrook, Illinois 60439
(312) 739-3810

Material Compiled and Lectured by

Herbert G. Poertner
President
Stormwater Consultants

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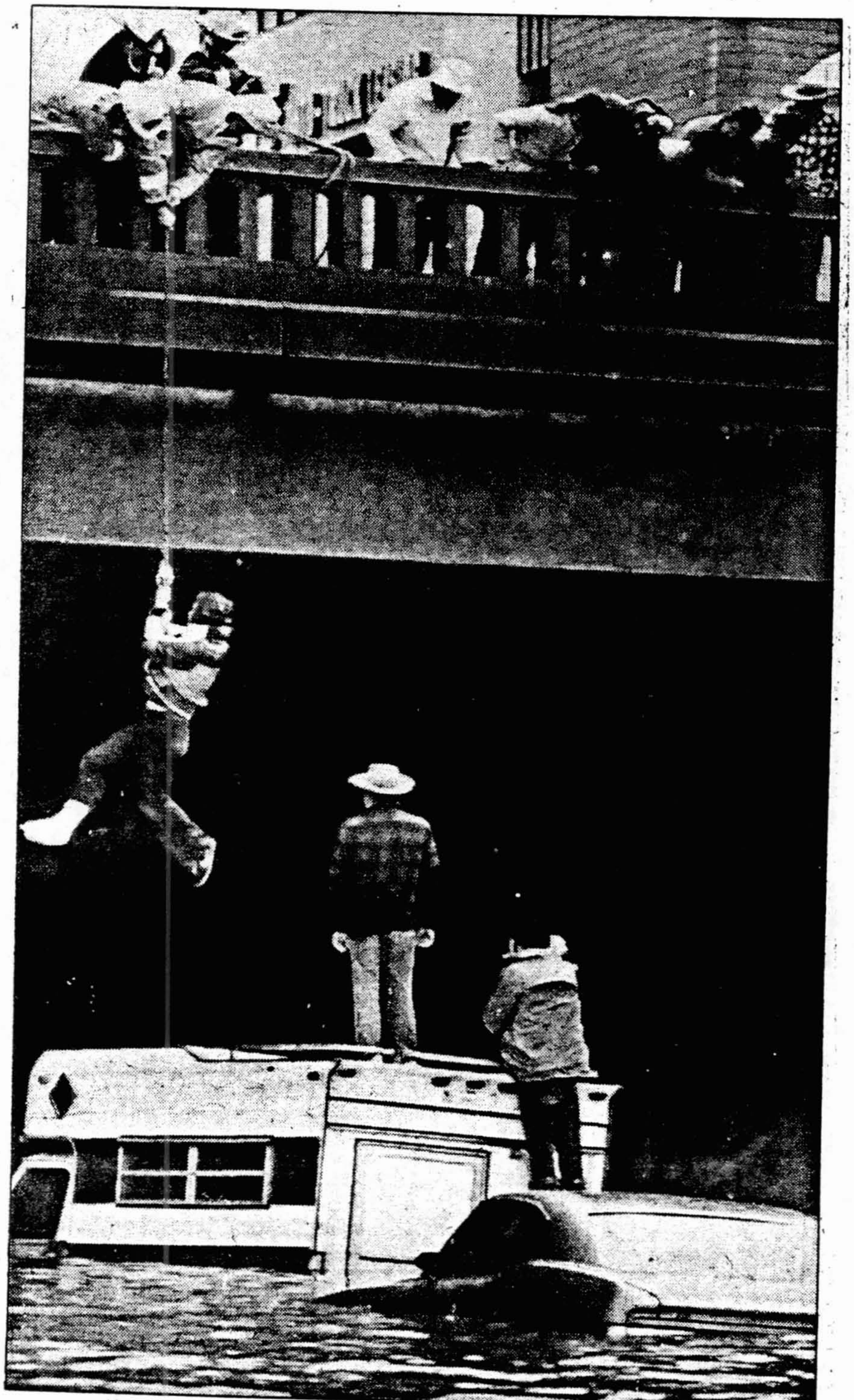
AP Laserphoto

Freeway rescue

Rescue workers pull a motorist to safety from the flooded Long Beach Freeway Sunday in Los Angeles as other motorists wait their turn. Heavy rains pounded southern California throughout the day, inundating many low areas with water and causing some flash flooding.

Chicago Tribune-Feb. 28, 1983

DESIGN OF URBAN DRAINAGE SYSTEMS

(Part B)

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1. Adequate Inspection by Qualified People -
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4. Financing Problems - Storm Water Mgmt -
5.

STORMWATER DETENTION: Overview

collection + storage

Capture Excess runoff—
with controlled release —

1.0 Objectives of Detention; Urbanization Effects

1.1 Reasons For Providing Detention Facilities

Reduce peak flows into downstream drainage systems, Both natural and man-made, to control flooding, erosion, sedimentation and quality of receiving waters.

Reduce initial costs of providing a drainage system.

Provide opportunities for multiple-purpose use of drainage systems: e.g., recreation, groundwater recharge, enhancement of property values, aesthetics, irrigation, guarding the public health.

TABLE 1

OBJECTIVES IN REQUIRING DETENTION
--in order of importance using 100 as "most important"--
(Source: Survey by American Public Works Association, 1980)

Objective	Rank
Reduce Downstream Flooding	100
Reduce Cost of Drainage Systems	71
Reduce On-site Flooding	70
Reduce Soil Erosion	66
Capture Silt	64
Improve On-site Drainage	63
Reduce Pollution from Stormwater	56
Improve Aesthetics	53
Enhance Recreational Opportunities	51
Replenish Groundwater	42
Supplement Domestic Water Supply	36
Capture Water for Irrigation	35
Other	22

1.2 Undesirable hydrologic products of land development:

- .Larger peak flow
- .Shorter "time to peak"
- .Higher stage in downstream drainage channels
- .Increased runoff volume
- .Increased flow velocities
- .Increased soil erosion and sedimentation
- .Receiving water quality adversely affected

Increases both peak surface runoff flows and volumes.

Increase is largely dependent upon land uses before and after development. (Not unusual for peak discharge to increase by a factor of five or six).

Other factors (some of which are dependent upon land use) are: degree of imperviousness, land surface slopes, surface roughness, antecedent moisture condition, and soil types.

1.3 Increased surface flows produce increased flow velocities and stages in downstream channels and pipe networks. Adverse downstream impacts are:

- .flooding
- .soil erosion and sedimentation
- .pollution of receiving water bodies

2.0 Stormwater Detention and Flow Attenuation

2.1 Concept and results

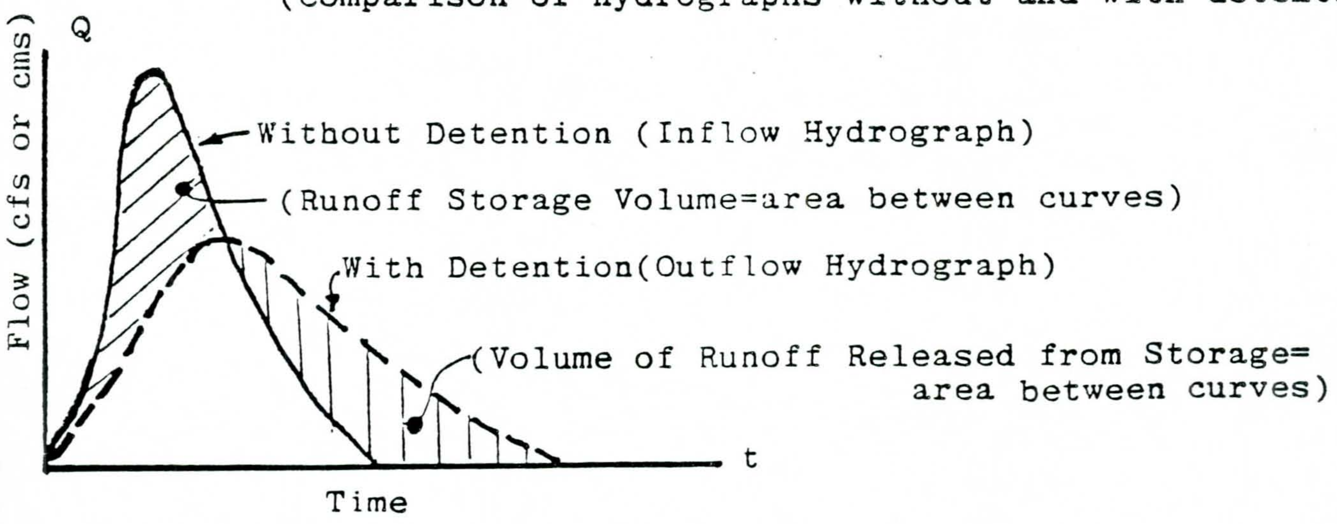
Temporarily store the excess runoff and release the stored volume at controlled rates compatible with: (1) capacities of downstream drainageways (natural and man-made) and (2) adopted regulations and ordinances.

2.2 Results obtained by detention storage:

- .runoff is spread over a longer time period;
- .peak flows into downstream areas are thereby controlled in magnitude;
- .total runoff volume may, or may not, be reduced (dependent upon rate of infiltration into soils).

FIGURE A. CONCEPT OF STORMWATER DETENTION

(Comparison of Hydrographs without and with detention)



In Figure A, area between inflow hydrograph and outflow hydrographs equals volume of excess runoff stored, and released. The two volumes are ordinarily equal, except for infiltration and miscellaneous losses.

3.0 Benefits of Storage

3.1 Attenuates peak flows and kinetic energy of surface runoff

Helps control soil erosion downstream and, thereby, controls amounts of pollutants transported into downstream areas. Sediment accumulations in downstream areas are also less.

Helps control flood stages in open drainageways downstream, and excessive surcharging of storm sewer networks--thus reducing floodwater damages.

3.2 Stormwater treatment

While in storage, pollutant concentrations will be reduced somewhat through natural processes such as sedimentation, flotation and oxidation. Once captured, stormwaters may also be given special treatment.

3.3 Groundwater recharge

Amount of recharge is dependent upon infiltration capacity of the soil, hydrostatic head, and sum-total of durations of storage events.

3.4 Reduces frequency and damages of sewer overflows

Important in areas of flat terrain where hydraulic gradients are small.

In combined sewer systems, reduces damages from sewage overflows and backups into basements, streets, land and receiving waters.

3.5 Economic benefits:

Minimizes needed sizes of storm sewer pipe diameters and drainage channel cross-sections--and associated costs.

Detention facilities may serve multiple-purpose uses, resulting in larger dollar value of benefits per dollar of facility cost.

Detention ponds, carefully designed and properly maintained can enhance property values, especially for abutting parcels.

4.0 Terminology

4.1 Terminology associated with detention storage has not been standardized. Suggested terminology is given below.

detention: temporary storage of excess surface runoff--either on, below or above the ground surface--accompanied by controlled release of the stored water.

on-site detention: temporary storage of runoff on the same land development site where the runoff is generated--frequently required as a condition for subdivision plat approval.

on-stream detention: temporary storage of runoff in a principal drainage system; i.e., in receiving streams or conduits.

off-stream detention: temporary storage accomplished "off line"; i.e., not within a principal drainage system.

detention pond: a stormwater detention facility, natural or man-made, which maintains a fixed minimum water elevation between runoff events except for the lowering resulting from losses of water due to infiltration or evaporation.

detention basin: a facility that empties completely between runoff events.

5.0 Places to Store Excess Runoff

5.1 Three places to store runoff:

- .at ground level--in ponds, basins, infiltration pits, and on paved areas;
- .underground--in oversized drains and sewers, caverns or tanks, dry wells, and within porous rock strata;
- .aboveground--on rooftops.

5.2 Survey by the APWA Research Foundation , (in 1980)

Of 325 communities returning a completed survey questionnaire, 219 reported having detention facilities (average was about 58 facilities per agency reporting detention).

Nearly 40 percent of agencies reporting none said detention facilities being built, planned, or a priority item for near future.

Types of facilities reported in use are shown in Table 2.

TABLE 2

DETENTION FACILITIES IN USE IN THE UNITED STATES AND CANADA
(Source: Survey by American Public Works Association, 1980)

Type of Facility	Total in Use		Ownership			
			Private		Public	
	(No.)	(Percent)	(No.)	(Percent)	(No.)	(Percent)
Dry Basin	6053	47.8	4913	81	1140	19
Parking Lot	3134	24.7	2982	95	152	5
Pond	2382	18.8	1199	50	1183	50
Rooftop Storage	694	5.5	644	93	50	7
Underground Tank	160	1.3	142	89	18	11
Oversized Sewer	135	1.0	83	61	52	39
Underground Tunnel	9	0.1	8	89	1	11
Other	116	0.9	64	55	52	45
Totals	12,683		10,035	79	2,648	21

No. 1 Parking lot in Fairfax County, VA showing infiltration trench.

No. 2 Detention pond in Los Angeles County, CA — multiple-purpose use, fishing derby.

No. 3 Depressed parkway in downtown Denver, CO — multiple-purpose use, passive recreation and stormwater storage.

FIGURE 7-4

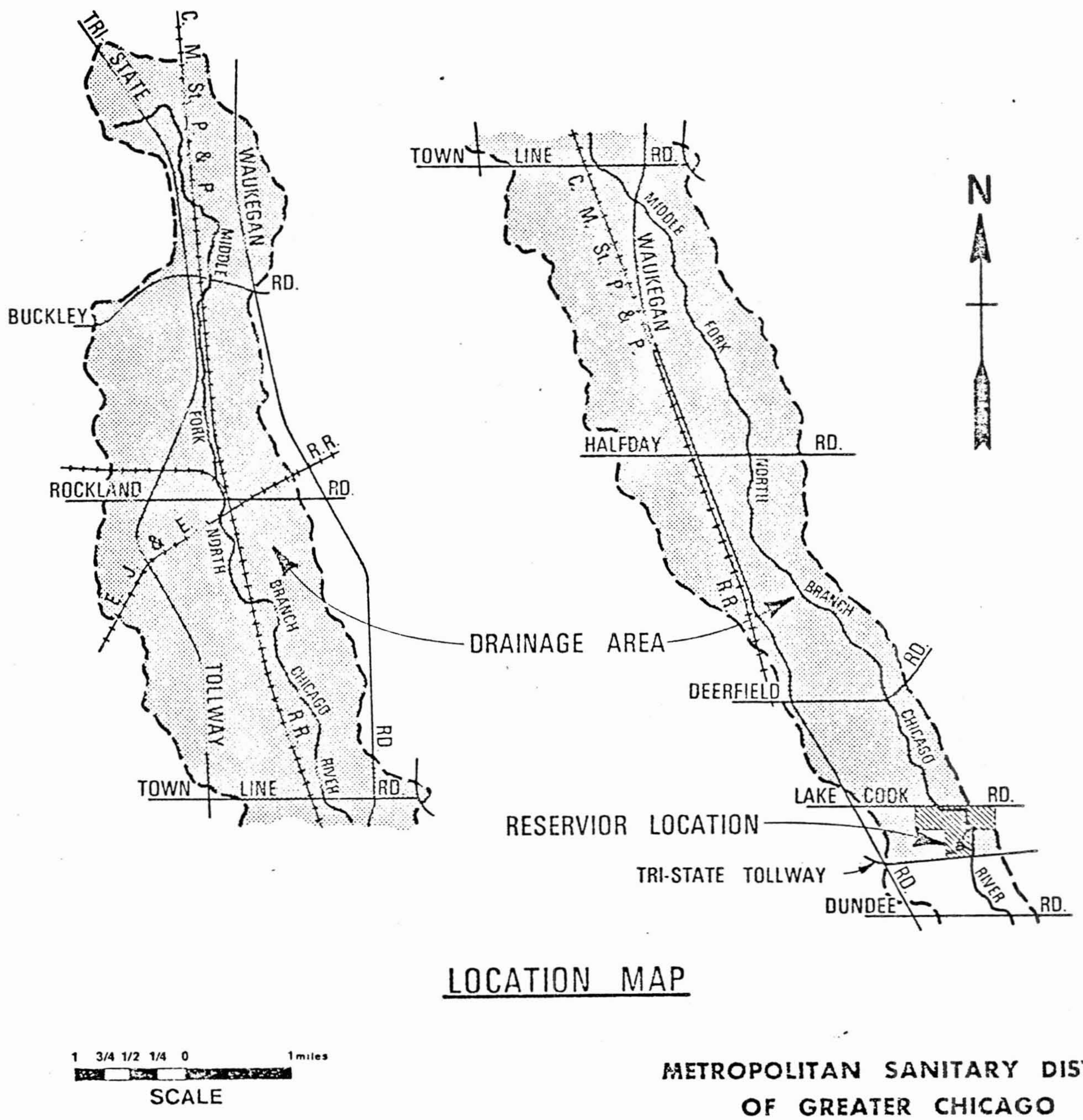
EXAMPLES: APPLICATION OF STORMWATER MANAGEMENT DETENTION FACILITIES

I EXHIBIT A

MIDDLE FORK NORTH BRANCH OF THE CHICAGO RIVER RESERVIOR

PROJECT NO. 73-063-2F

DRAINAGE AREA	20.7 SQ. MILES
DESIGN STORM	100 YEARS
PUMPING STA. CAPACITY	111.0 C.F.S.
CONSTRUCTION COMPLETED	11-1-74
CONSTRUCTION COSTS	$2,900,000
LAND AREA	22 ACRES
LAND COST	(FURNISHED BY OWNER)



LOCATION MAP

METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

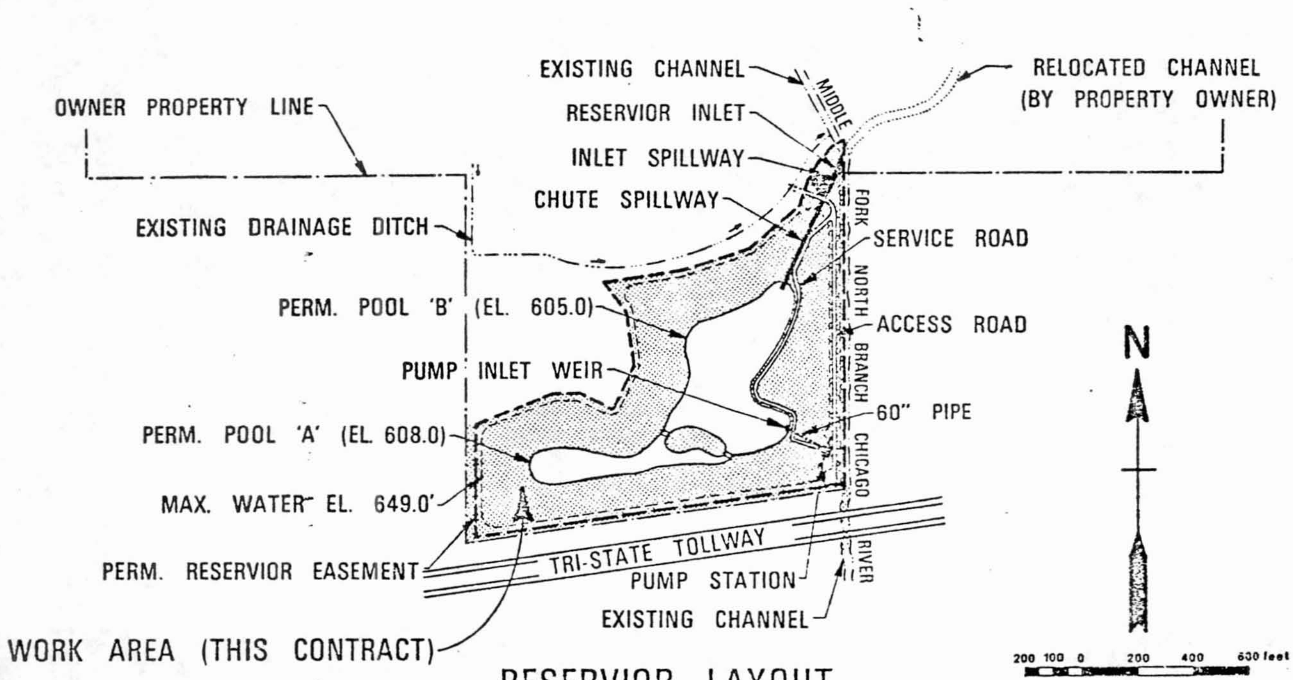
FLOOD CONTROL SECTION

J.G.N. JULY 1973

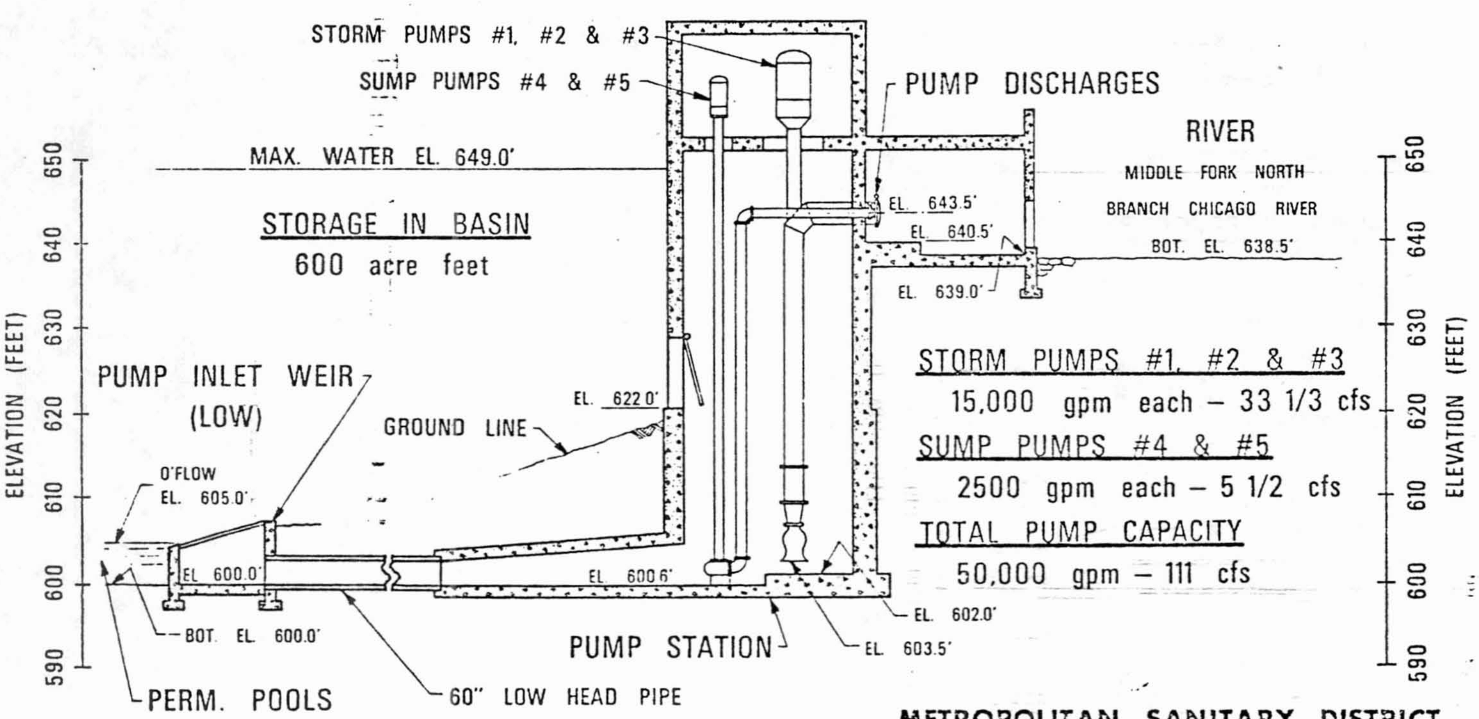
I EXHIBIT B

MIDDLE FORK NORTH BRANCH OF THE CHICAGO RIVER RESERVIOR

PROJECT NO. 73-063-2F



RESERVIOR LAYOUT

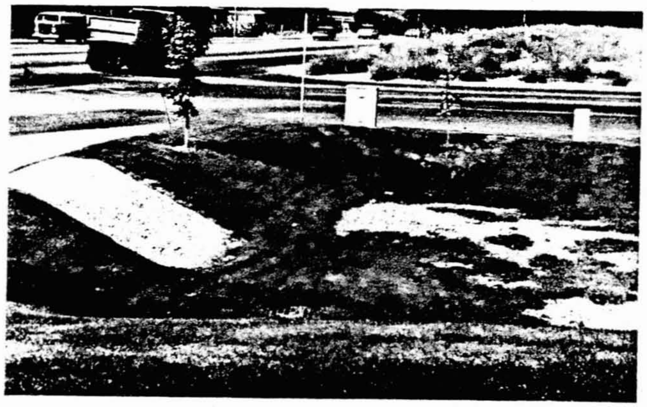


PROFILE

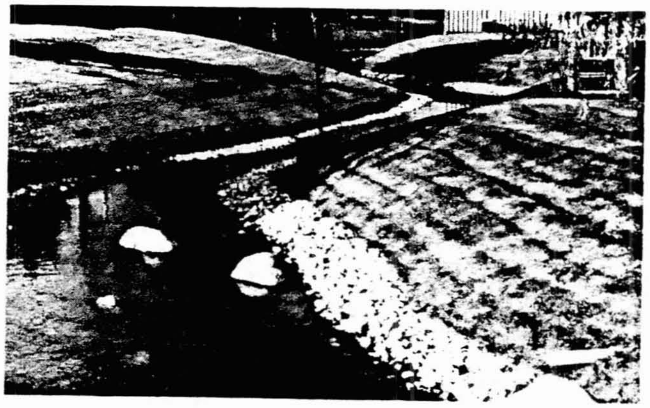
METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

FLOOD CONTROL SECTION

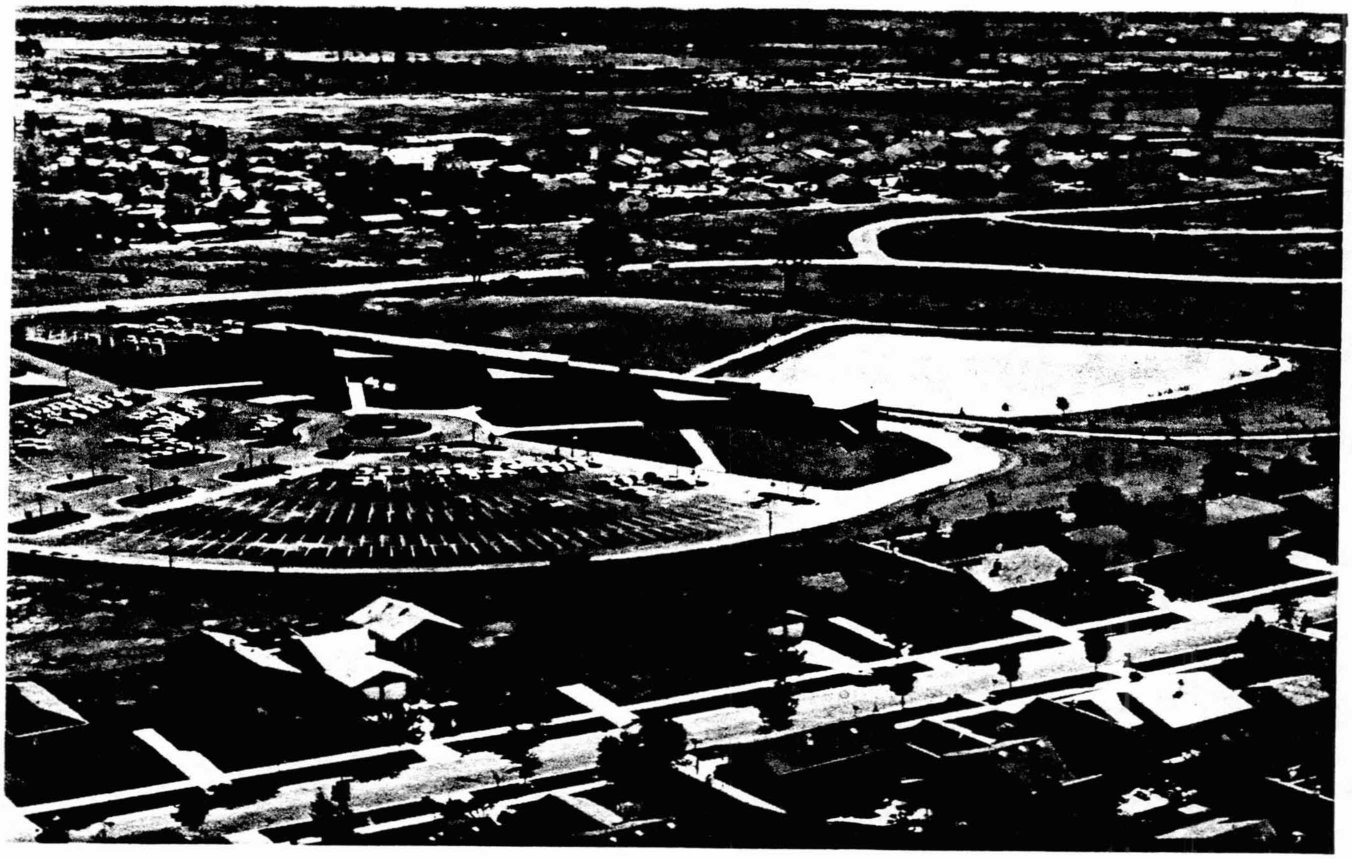
J.G.N. **JULY 1973**



No. 4 Small detention basin adjacent to office building in Ogden, UT.



No. 5 Small detention pond in residential subdivision, Northbrook, IL.



No. 6 Stormwater detention pond at the Village Hall in Bolingbrook, IL

5.3 Provision of Detention Facilities; How Required

Detention facilities are required to be "provided" by:
- .land developers alone in 15% of the masterplans
- .local governments or special districts alone in 45% of the masterplans
- .combination of developers and public agencies in remaining 40% of the masterplans

Means used to require detention storage

Means most used:
- .subdivision regulations
- .zoning ordinances
- .building codes

Other means used:
- .miscellaneous stormwater ordinances regulating drainage, soil erosion, sedimentation and land grading
- .prior agreement with developers
- .deed covenants
- .stormwater management manuals, and
- .case-by-case site review

Publications Suggested for Review

1. Poertner, H.G., Stormwater Management in the United States, Stormwater Consultants, 3 Westchester Ct., Bolingbrook, IL., 60439, September 1980, 240 pp.

2. Poertner, H.G. and Reindl, J., "United States Practices in Detention of Urban Runoff", paper presented at Surface Water Impoundments Conference, University of Minnesota, Minneapolis, June 2-5, 1980.

3. Northeastern Illinois Planning Commission, Suggested On-Site Stormwater Detention Ordinance, Chicago, March 1980, 55 pp.

4. Poertner, H. G., et al, Urban Stormwater Management, Special Report No. 49, American Public Works Association, Chicago, 1981, 285 pp.

STORM DRAINAGE COST FUNCTIONS

Walter J. Rawls[1]

(Installation Costs Only)
No Land or Maintenance

NEED FOR STORM DRAINAGE COST FUNCTIONS

The economic and environmental constraints force urban developers to consider a wide range of alternative plans to prevent flood damage and enhance the utility of land.

Storm drainage facilities are a major urban development cost and planners must be able to estimate their cost without resorting to costly full engineering studies.

FACTORS INFLUENCING COST OF STORM DRAINAGE SYSTEMS

Developed area
Capacity
Slope of area
Design return period
Type of urban development
Geographical location of the area
Pipe sizes

DEVELOPMENT OF STORM DRAINAGE COST FUNCTIONS

Need simple technique to examine the economic effect of:

Scale of development.
Degree of protection.
Structural and nonstructural alternatives for controlling runoff.

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1 Hydrologist, U.S. Department of Agriculture, Science and Education Administration, Hydrology Laboratory, Building 007, BARC-West, Beltsville, Maryland 20705

Technique should:

Use easily obtained variables.
Be based on local or regional data for typical development patterns.

Form of cost equation:

Log linear (Rawls & Knapp, 1972).

Standard regression techniques can be used for calibrating the cost equations to basin and design variables.

Equations can be updated using construction cost indexes (EPA, 1976).

Types of cost equations:

Storm Sewer (Rawls & McCuen, 1978):

$$C_T = 4{,}952 S^{-0.138} Q^{0.516} A_D^{0.262} \quad (1)$$

in which C_T = storm sewer costs, in 1976 dollars; S = average ground slope, as a percentage; Q = total capacity, in cubic feet per second; and A_D = total developed area, in acres. This equation, which is based on 70 projects located around the U.S., provided a correlation coefficient of 0.80 and a standard error of estimate of \$59,400 which is about 28% of the mean project cost.

Detention Basin (Rawls & McCuen, 1978):

$$C_F = 2691\ A^{0.574} \quad (2)$$

in which C_F = detention basin cost, in 1976 dollars; and A = total drainage area, in acres. Eq. 2, which is based on 34 storm-water detention projects in the Washington, D.C. area, resulted in a correlation coefficient of 0.89 and a standard error of estimate of \$9,507. The foregoing detention-cost equation is based on the assumption that the detention basin will control peak runoff so that it will be the same before and after urbanization for the 2-yr to 5-yr return period. Also, the foregoing equation does not include engineering design costs and land costs.

COMBINING COST FUNCTION WITH HYDROLOGIC DESIGN PROCEDURES

Hydrologic Assessment Variables:

Imperviousness - measure of development.
Design return period - degree of protection.
Time of concentration - efficiency of alternatives for controlling runoff.

Hydrologic Design Procedures:

Peak Runoff Estimating.

There are many peak runoff estimation procedures which could be used. For this example I am going to use the rational formula.

$$Q = CIA \tag{3}$$

$$C = 0.14 + 0.65\,(Imp) + 0.05(S) \tag{4}$$

$$t_1 = \frac{1.05\, L^{0.24}}{S^{0.16}\, Imp^{0.26}} \tag{5}$$

$$t_c = 1.67\, t_1 \quad \text{(NOTE: } t_c \text{ is the major variable)} \tag{6}$$

in which Q = peak flow in cfs; C = a dimensionless runoff coefficient; I = average rainfall intensity, in inches per hour for a period of time, t_c (called the time of concentration); A = the size of the drainage area, in acres; Imp = the ratio of impervious area to total area; S = slope of the main channel, as a percentage; t_1 = time lag in minutes (time between centroid of the rainfall hyetograph and the centroid of the runoff hydrograph), L = length of main drain, in feet; and t_c = time of concentration in minutes (Equations 4 and 5 came from Schaake, 1976; equation 6 from Soil Conservation Service, 1975).

Average rainfall intensities for various return periods and durations can be obtained from the National Weather Service, 1976, and put into the following form:

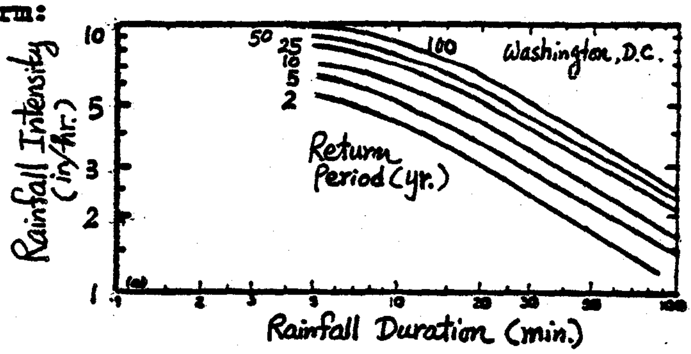

FIG. 1.—Rainfall-Intensity-Frequency-Duration Curves for: (a) Washington, D.C.;

Nomographs based on average conditions can be developed for preliminary costing using equations 1, 3, 4, 5, and 6. Following are examples of how the nomographs can be developed:

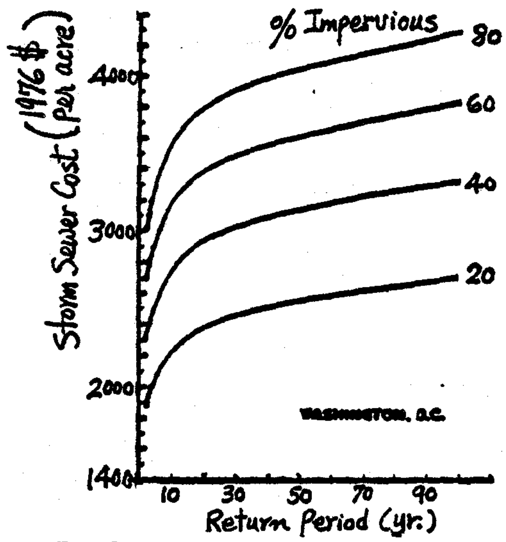

FIG. 2.—Storm Sewer Cost as Function of Return Period and Imperviousness for Washington, D.C. (1 acre = 0.405 ha)

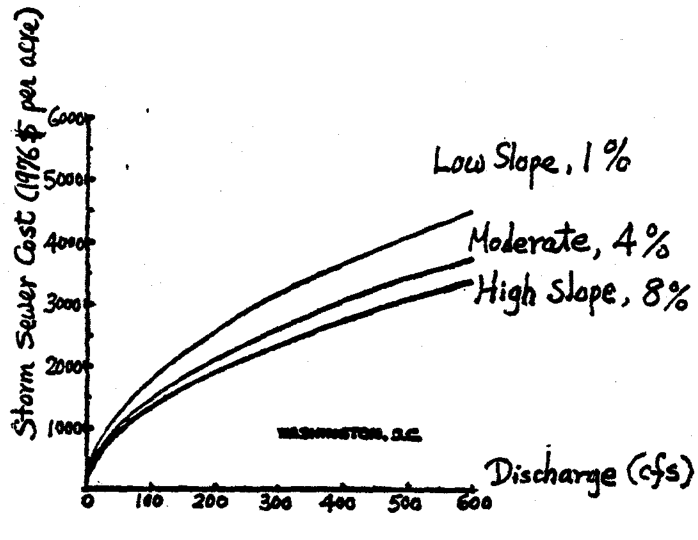

FIG. 3.—Storm Sewer Cost as Function of Discharge and Slope for Washington, D.C. (1 acre = 0.405 ha; 1 cu ft = 0.028 m³)

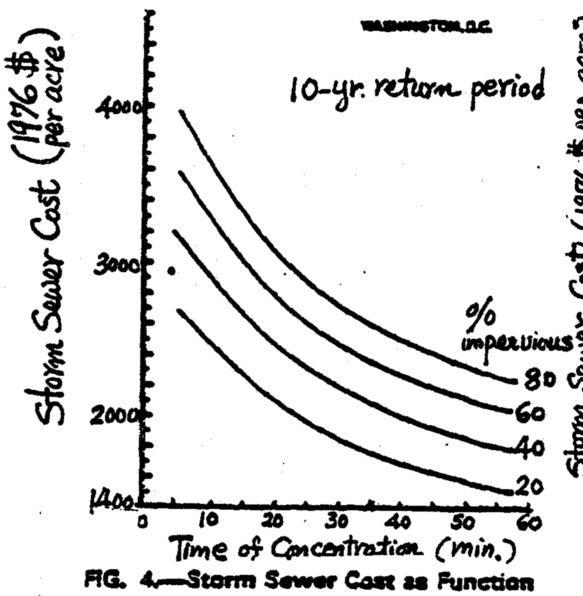

FIG. 4.—Storm Sewer Cost as Function of Time of Concentration and Imperviousness for a 10-Yr Return Period for Washington, D.C. (1 acre = 0.405 ha)

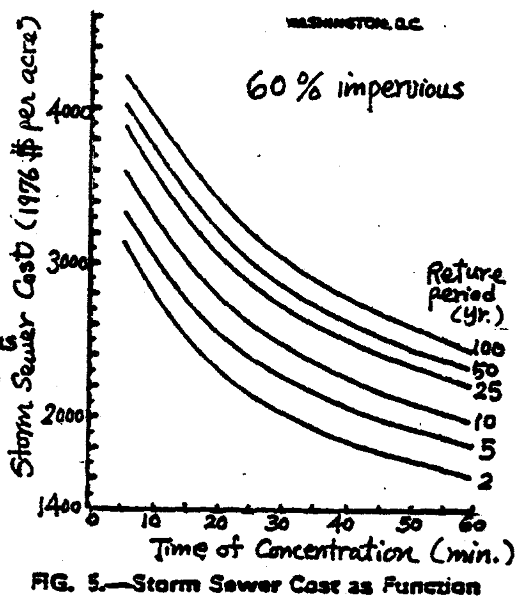

FIG. 5.—Storm Sewer Cost as Function of Time of Concentration and Return Period for 60% Imperviousness for Washington, D.C. (1 acre = 0.405 ha)

The equations and actual basin conditions should be used for more accurate costing.

EVALUATING THE ECONOMIC EFFECT OF STORM WATER MANAGEMENT ALTERNATIVES

Figures 2, 3, 4 and 5 can be used directly for analyzing the effects of imperviousness, design return period and slope on cost.

The effect of grassed waterways, lined channels, detention basins, etc. can be determined using time of concentration (Figures 4 and 5) as an index.

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"Construction Cost Indexes," Environmental Protection Agency, Office of Water Program Operations Municipal Construction Division, Washington, D.C., 1976

Curtis, D. C., and McCuen, R. H., "Design Efficiency of Stormwater Detention Basins," Journal of the Water Resources Planning and Management Division, ASCE, Vol. 103, No. WR1, Proc. Paper 12938, May 1977, pp. 125-140.

"Five-to-60 Minute Precipitation for the Eastern and Central United States," NOAA S/T76-2497, National Weather Service, Washington, D.C., 1976.

Grigg, N. S., and O'Hearn, J. P., "Development of Storm Drainage Cost Functions," Journal of the Hydraulics Division, ASCE, Vol. 102, No. HY4, Proc. Paper 12009, Apr., 1976, pp. 515-526.

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EXHIBIT A

EXAMPLES OF CONSTRUCTION COST OF STORMWATER MANAGEMENT FACILITIES

(land not included) From: "Urban Stormwater Management" Chapter 13, Amer. Public Works Ass'n. 1981

A. Small Facilities, Chester County, Pennsylvania

RHONDDA —	**Basin No. 1**	**Basin No. 2**	**Basin No. 3**
Land Use	Housing Development	Housing Development	Housing Development
Type of Basin	Basin	Pond	Basin
Surface Area	0.2 ha (0.5 ac)	0.7 ha (1.7 ac)	0.6 ha (1.4 ac)
Storage Volume	1,850 cu m (1.5 ac-ft)	4,565 cu m (3.7 ac-ft)	2,590 cu m (2.1 ac-ft)
Drainage Area	7.8 ha (19.4 ac)	4.7 ha (11.64 ac)	12 ha (30 ac)
Construction Cost	$12,000	$5,000	$10,000
Year of Completion	1977	1977	1977

PLEASANT GROVE —		
Land Use	Housing Development	Housing Development
Type of Basin	Pond	Pond
Surface Area	0.4 ha (1.0 ac)	1.7 ha (4.2 ac)
Storage Volume	4,935 cu m (4 ac-ft)	28,370 cu m (23 ac-ft)
Drainage Area	111 ha (275 ac)	70.4 ha (174 ac)
Construction Cost	$50,000	$80,000
Year of Completion	1980	1980

	Tarrencoyd Basin	**New Kent Apartments**	**Marydell Farms**
Land Use	Housing Development	Multi-family Housing	Housing Development
Type of Basin	Basin	Basin	Permanent Pond
Surface Area	0.26 ha (0.63 ac)	0.5 ha (1.2 ac)	0.9 ha (2.3 ac)
Storage Volume	6,290 cu m (5.1 ac-ft)	9,500 cu m (7.7 ac-ft)	13,200 cu m (10.7 ac-ft)
Drainage Area	29.2 ha (72.15 ac)	14.2 ha (35 ac)	36.8 ha (91 ac)
Construction Cost	$16,000	$8,000	$25,000
Year of Completion	1977	1973	1972

B. Large Facilities, Chicago, Illinois

MIDDLE FORK, NORTH BRANCH OF THE CHICAGO RIVER RESERVOIR

Land Use	Mixed
Type of Basin	Pond, pumped removal
Surface Area	8.9 ha (22 ac)
Storage Volume	740,100 cu m (600 ac-ft)
Drainage Area	33.1 sq km (20.7 sq mi)
Construction Cost	$2,900,000
Year of Completion	1975

Source: Bernard Hankin Builders, Exton, Pennsylvania

EXAMPLES UNIT COSTS FOR STORAGE AND AREAS SEWERED, STORMWATER MANAGEMENT FACILITIES

(updated to June 1980, 1980 EPA Construction Cost Index for Sewer Systems)

Type Name	**Cost/Volume $/ac-ft**	**Cost/Area Served $/ac**
Small Basin		
a. Rhondda Basin No. 1	$10,760	$830
b. Rhondda Basin No. 3	6,405	450
c. Tarrencoyd Basin	4,220	300
d. New Kent Apartments	2,015	440
Small Pond		
a. Rhondda Basin No. 2	1,820	575
b. Pleasant Grove Basin No. 1	12,500	180
c. Pleasant Grove Basin No. 2	3,480	460
d. Marydell Farms	4,875	570
Large Pond		
Middle Fork, No. Branch of Chicago River Reservoir	7,330	330

TABLE 13-7
COSTS FOR CONSTRUCTION OF STORMWATER MANAGEMENT FACILITIES

Franklin Farm
Fairfax, Virginia
(1981 Estimate)

Note: 1 acre = 0.4 hectare
1 acre-foot = 1233.49 cubic meters

	1	2	3	4	5 Sarah's Pond	6	7	8	9	10	11	12
Type of Basin:	Dry	Pond	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry
Surface Area:												
2-year storm	0.41ac	1.47ac	0.55ac	0.28ac	1.75ac	1.06ac	0.88ac	0.04ac	0.36ac	1.95ac	1.78ac	0.6 ac
10-year storm	0.57ac	1.56ac	0.65ac	0.35ac	2.06ac	1.41ac	1.0 ac	0.49ac	0.64ac	2.04ac	2.04ac	0.69ac
100-year storm	1.01ac	2.29ac	1.33ac	0.85ac	4.14ac	2.49ac	2.37ac	0.78ac	1.63ac	4.17ac	2.81ac	1.17ac
Normal Pool	-	1.24ac	-	-	-	-	-	-	-	-	-	-
Storage Volume: (ac-ft)												
2-year storm	0.6	1.97	0.80	0.25	1.77	1.89	1.09	1.40	0.46	3.87	6.30	0.80
10-year storm	0.87	2.65	0.32	0.37	2.39	2.59	1.62	0.58	0.65	5.81	8.77	1.12
Drainage Area:	227.0ac	56.53ac	35.58ac	17.5ac	31.74ac	19.86ac	24.68ac	15.9ac	12,06ac	94.12ac	98.71ac	42.86ac
Total Cost:	$7,026	6,993	10,661	4,475	6,149	10,801	6,098	8,372	10,538	17,997	44,521	10,914
Cost Breakdown:												
Excavation	$2,500	2,310	4,250	1,500	2,500	6,250	1,800	4,568	4,000	10,000	30,750	6,427
Pipework	$3,276	4,183	5,139	2,415	2,669	2,826	3,412	2,247	1,978	5,570	5,441	3,055
Stabilization	$1,250	500	1,272	560	980	1,725	886	1,557	4,560	2,427	8,330	1,[illegible]32
Total Cost	$7,026	6,993	10,661	4,475	6,149	10,801	6,098	8,372	10,538	17,997	44,521	10.914

	13	14 Tuckaway Pond	15	16	Still Pond	Hannah's Pond	Sallie's Pond	Franklin Pond	Village Pond
Type of Basin:	Pond	Dry	Dry	Dry	Pond	Pond	Pond	Pond	Pond
Surface Area:									
2-year storm	0.66ac	0.26ac	0.42ac	0.11ac					
10-year storm	0.83ac	0.35ac	0.83ac	0.13ac					
100-year storm	1.08ac	0.53ac	1.10ac	0.16ac					
Normal Pool	0.46ac	-	-	-	1.82ac	0.52ac	0.51ac	3.34ac	0.95ac
Storage Volume: (ac-ft)									
2-year storm	1.54	0.25	1.41	0.09					
10-year storm	2.27	0.35	2.18	0.15					
Drainage Area:	55.84ac	10.5ac	54.8ac	8.3ac	104.83ac	16.64ac	12.4ac	26.91ac	13.26ac
Total Cost:	$6,557	6,725	13,438	10,690	4,429	1,508	9,074	18,393	4,115
Cost Breakdown:									
Excavation	$1,668	3,750	6,250	2,160	3,840	1,280	7,850	15,900	3,550
Pipework	$4,311	885	5,193	8,360	-	-	-	-	-
Stabilization	$ 578	2,090	1,995	170	589	228	1,224	2,493	565
Total Cost	$6,557	6,725	13,438	10,690	4,429	1,508	9,074	18,393	4,115

Source: "Urban Stormwater Management", Chapter 13, American Public Works Ass'n., 1981

II EXHIBIT C

INTER OFFICE MEMORANDUM

THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

DEPARTMENT: ENGINEERING DATE: April 7, 1978

TO: RAYMOND R. RIMKUS, Chief of Maintenance and Operations

FROM: FORREST C. NEIL, Chief Engineer

SUBJECT: M&O REQUIREMENTS FOR FLOOD CONTROL PROJECTS

In response to your memo of March 22, 1978, the attached tabulation has been prepared identifying the reservoirs, streams and channels with future MSD maintenance responsibility. The tabulation includes an estimate of the annual maintenance costs for each project along with a projected date for commencement of the maintenance.

The maintenance costs for the reservoirs have been estimated on the basis of previous estimates prepared by the Soil Conservation Service. In the cases where the maintenance is the responsibility of others, the MSD is still involved in semi-annual inspections. The tabulation lists an accrued cost of $1,000 for the inspection activity which is coded with an "I".

Where the MSD is responsbile for maintaining the structures and operating equipment for reservoir, the maintenance cost was estimated on the basis of the equations:

C = $2,000 + $2.00 X Storage (ac.ft.)

These projects are coded with an "S" in the Table.

Finally, some projects involve all the maintenance for a site including lawn mowing and debris removal. For these projects the cost was based on the equation:

C = $2,000 + $2.00 X Storage (ac.ft.) + $50.00 X Area (acres)

These projects are coded "F" in the tabulation.

Flood Control personnel will be available to provide additional details regarding the required maintenance for individual projects as they are completed.

Forrest C Neil

Forrest C. Neil

RC JJ FE

FED/RC/JJ:im
Attachment

cc: w/attachment
Dalton/Carlson/Jackson
Neil

Structure 15
Structure 18

Chapter III

Considerations in Planning and Designing Detention Facilities

1.0 Planning

1.1 Stormwater problems and solution priorities

- consider not only local problems, but also upstream and downstream problems and needs
- advisable to make a watershed study of the problems when feasible--at the minimum, a sub-watershed study
- establish priorities for solution of problems so that problems can be addressed in a logical sequence
- be practical concerning non-engineering constraints--e.g., required funds, interagency cooperation, politics, etc.

1.2 Identify goals and establish specific objectives

- do not neglect non-structural solutions which are often better and more economical
- consider "prevention" as well as "correction"
- strive for multiple-purpose use of detention--e.g., recreation, groundwater recharge, water supply, etc.
- multiple-objectives may be difficult to accomplish in a single detention facility; e.g., sediment and water quality control as well as flood control
- consider areawide goals and policies
- include preservation and protection of natural resources and wildlife in setting goals[1]
- be practical in view of "what is feasible" in light of laws and constraints (financing, local acceptance and support, etc.
- don't neglect aesthetic enhancement of the area
- secure cooperation of nearby public agencies having stormwater management authority

2.0 Engineering Design

2.1 Engineering impediments

- incomplete information and data analysis on precipitation and runoff
- difficulty of predicting runoff hydrographs accurately
- incomplete knowledge of soil conditions--before and after development
- limitations in adapting formulas, models and techniques for calculating accurate runoff rates and volumes
- unpredictability of long term urban growth and land uses
- space limitations for accomodating detention facilities
- severe restrictions sometimes imposed by limited hydraulic capacity and stability of downstream drainage systems
- legislative and administrative regulations that are difficult to meet with limited project funds

2.2 Types of storage facilities to consider

- .pond (maintains a minimum water elevation)
- .basin (empties between runoff events)
- .oversized drainageways (pipes or open channels)
- .underground "disposal" facility (infiltration)
- .combination of more than one type (e.g., pond + basin)

2.3 Design guidelines

- .keep water out of habitable areas
- .a positive outlet should be provided for emergencies
- .surfaces of detention basins should be sloped to drain adequately --paved surfaces 1 percent, paved channels 0.4 percent, grassed areas 2 percent
- .deeper portions of storage should be in more remote, least-used areas (safety, inconvenience)
- .be careful to accomodate runoff from upstream (when required)
- .design should be based on the hydraulic gradient
- .discharge structures should be dependable types
- .be conservative in calculation of storage capacity required
- .consider by-passing small flows around "flood control" facilities
- .take advantage of opportunities for multiple-purpose uses of stormwater facilities

2.4 Design factors to consider and/or calculate

- .local rainfall frequency, intensity, and duration curves
- .size and location of the drainage area tributary to the proposed storage facility
- .hydrologic data of the tributary area
- .a graph of the rate of inflow to the detention facility- cfs vs. time (i.e., the inflow hydrograph)
- .hydraulic capacity of the downstream drainage system
- .the storage volume required (based on inflow and outflow)
- .spillway, or other means, for release of stored water and for by-passing excess flows of exceedingly rare rainfalls that cannot be stored by the facility designed
- .time limitations for draining the stored runoff to permit storage of the next runoff event
- .reliability of electrical and mechanical systems for storage facilities requiring pumped discharge (standby equipment)
- .location of storage facility (rooftop, parking lot, park, etc.)
- .safety precautions
- .factors pertinent to efficient maintenance and operation, annual cost, and useful life of the facility
- .flood routing for runoff greater than the design capacity of the detention facility

2.5 Major Design Tasks

Establish the maximum and minimum water levels (constraints to be adhered to).

Size the facility for storage volume and geometric shape, and set critical elevations.

Design the inflow and discharge structures, including the emergency spillway (and by-pass structures, when used).

Design the details and/or specify the measures to be used for minimizing maintenance and operation problems, minimizing annual costs, and enhancing aesthetics and safety.

- .bank slopes (protection and stabilization)
- .headwalls and other details at inflow and outflow structures
- .safety precautions (fences, bank slopes, etc.)
- .landscaping (select slopes to permit mechanized mowing)
- .bottom slopes in basins to assure dry soil for multiple-purpose uses

3.0 On-Site Detention vs. Regional Detention Facilities

3.1 On-Site Detention

On-site detention has been a topic of controversy among public works administrators, developers, and engineers primarily because of the following negative aspects:

- .without proper design and controls, releases of stored runoff from multiple ponds may cause delayed peak flow hydrographs having greater peaks than the direct discharge hydrograph for a given drainage basin
- .the effective operation of a detention facility depends on the sensitive balance between the required storage capacity and flow characteristics of the outlet control structure
- .land may not be available or suitable for detention
- .safety requirements may be difficult to satisfy
- .discharge water quality may not be improved due to lower dissolved oxygen content
- .shallow detention ponds with permanent water pools are susceptible to eutrophication and declining water quality
- .variation of water depth during dry spells may effect the recreational value of multiple-purpose detention facilities
- .long-term effectiveness of detention facilities is questionable
- .public acceptance of on-site detention may be lacking
- .precautionary overflow measures must be provided
- .maintenance requirements may be high

The concept of on-site detention embodies numerous benefits; however, its application must be thoroughly evaluated in the overall drainage plan to minimize adverse effects. Careful planning, design, and construction are essential elements in achieving maximum befefits.

3.2 Regional Facilities

Regional detention facilities can provide runoff control for many square miles of tributary area. As an example, a floodwater reservoir completed in 1974 by the MSD of Greater Chicago collects excess runoff from 20.7 square miles of upstream drainage area.

- .designed for the 100-year rainfall, the storage capacity is 600 acre-feet
- .all of the storage is on publicly-owned land which overcomes the obstacle of "who will operate and maintain the facility. Responsibility is well defined
- .economics of construction and O & M favor the regional reservoirs
- .they are less likely to develop nuisance problems (weeds, erosion, mosquitos, water pollution, etc.)
- .satisfactory operation and maintenance is not usually a problem when such facilities are properly planned, designed, and constructed ...and when an adequate and assured source of funding is available on a continuing basis. Therefore, a longer design life should be assumed
- .multiple-purpose use of detention storage facilities is more feasible with larger detention facilities (football and baseball fields for basins; boating and fishing for ponds)

4.0 Identifying and Selecting Detention Sites

4.1 General Locations

.Off-stream (not in natural drainageway or pipe system)
.On-stream (in natural drainageways or oversized pipes)

Off-stream detention---

Advantage Potentials

Adds flexibility to design:

- .location choices
- .depth and area alternatives
- .layout alternatives
- .hydraulic alternatives

Low flows may be by-passed into downstream drainageways without loss of valuable storage capacity.

Stored runoff may be held for whatever time desired.

Opportunities for multiple-purpose use are enhanced.

Disadvantage Potentials

Land required for storage may be prohibitively expensive, or land may not be available.

Operation and/or maintenance expenses can be large.

Peak flow rates into storage may overload the inlet control devices; and, as a result, the needed attenuation of peak flows may not be provided.

<u>On-stream</u> detention

Advantage Potentials

Short duration rainstorms will be well damped.

Disadvantage Potentials

Not a substitute for on-site detention--unless the volume of storage provided also allows for contributions from all tributary upstream areas.

Long-duration rainstorms may produce a condition in which inflow and outflow are comparable in magnitude, and the water level (stage) stabilizes. The increase in flow caused by urbanization will, therefore, be passed into downstream reaches.

Increases stream stages in upstream areas.

Contributes to surcharging of storm sewers in upstream areas.

<u>On-site</u> detention

Advantage Potentials

Attacks or prevents problems at their sources.

Small areas for detention sites are feasible.

Construction cost of stormwater collection and transport system is less.

Ponds can serve as urban wildlife habitats.

Recreation opportunities available within walking distance.

Aesthetics improved (if properly maintained).

Protects areas along entire length of drainage system from new stormwater problems, and prevents compounding existing problems.

Offers choice of methods (rooftop, parking lot, ground surface, etc.).

Can sometimes be incorporated into other local programs (parks, etc.).

Disadvantage Potentials

Sometimes requires use of expensive, desirable building sites.

May result in a proliferation of randomly located detention facilities.

Maintenance of many scattered facilities is often a serious problem for local public agencies or property owners.

Environmental detriments and safety hazards can develop when maintenance is lacking.

4.2 Facility Siting

A key element of site selection for a detention system is the number, sizes, and positions of the facilities in the watershed. The objective is the overall control of peak flows, in an entire watershed, rather than control only at points immediately downstream of a detention pond or basin.

"Timing" of flows converging at downstream points should be analyzed by developing hydrographs of outflow from the various detention storage facilities and routing these hydrographs through the drainageway by means of a computer model such as the Penn State Runoff model (PSRM)[2], the Soil Conservation Service Model (TR-20), or others.

Four Mile Run Case Study[3] (Refer to Figure 1)

- .watershed area 19.5 Sq. Miles in Northeastern Virginia
- .discharges through City of Alexandria where considerable flood damages were common--for a mile upstream of confluence with Potomac River
- .in 1975, computer models were used to project streamflow impacts throughout the watershed, resulting from development in any subarea. Modeling is on-going as a source of information for making decisions regarding watershed development
- .Findings:
- ...development peak flow impacts are most sensitive to land development changes in the middle and upper-middle portions of the watershed
- ...the most beneficial results in controlling peak flows in downstream reaches can be achieved best by providing detention storage in the middle portions of the watersheds
- ...benefits of detention storage if provided in extreme upper and lower portions of the watershed were found to be minimal

4.3 "Random" Location of Detention

"Mismanagement" of stormwater (rather than management) may result from the random location of detention facilities in a watershed. Peak discharges at downstream points may be increased, rather than decreased.

A computer study[4] of a large watershed, in Virginia, along the Atlantic Coastal Plain, revealed that randomly-located detention facilities may either produce in the downstream reaches (refer to Figure 2):

- .no reduction of peak flows (detention scattered throughout watershed)
- .increases in peak flows (+25%) with detention located in downstream areas
- .a reduction of peak flows (-17%) with detention in upstream areas only.

The most effective position for storage in the watershed is dependent upon its "physical characteristics"; i.e., ground slopes, channel slopes, degree of land development, land uses, etc.

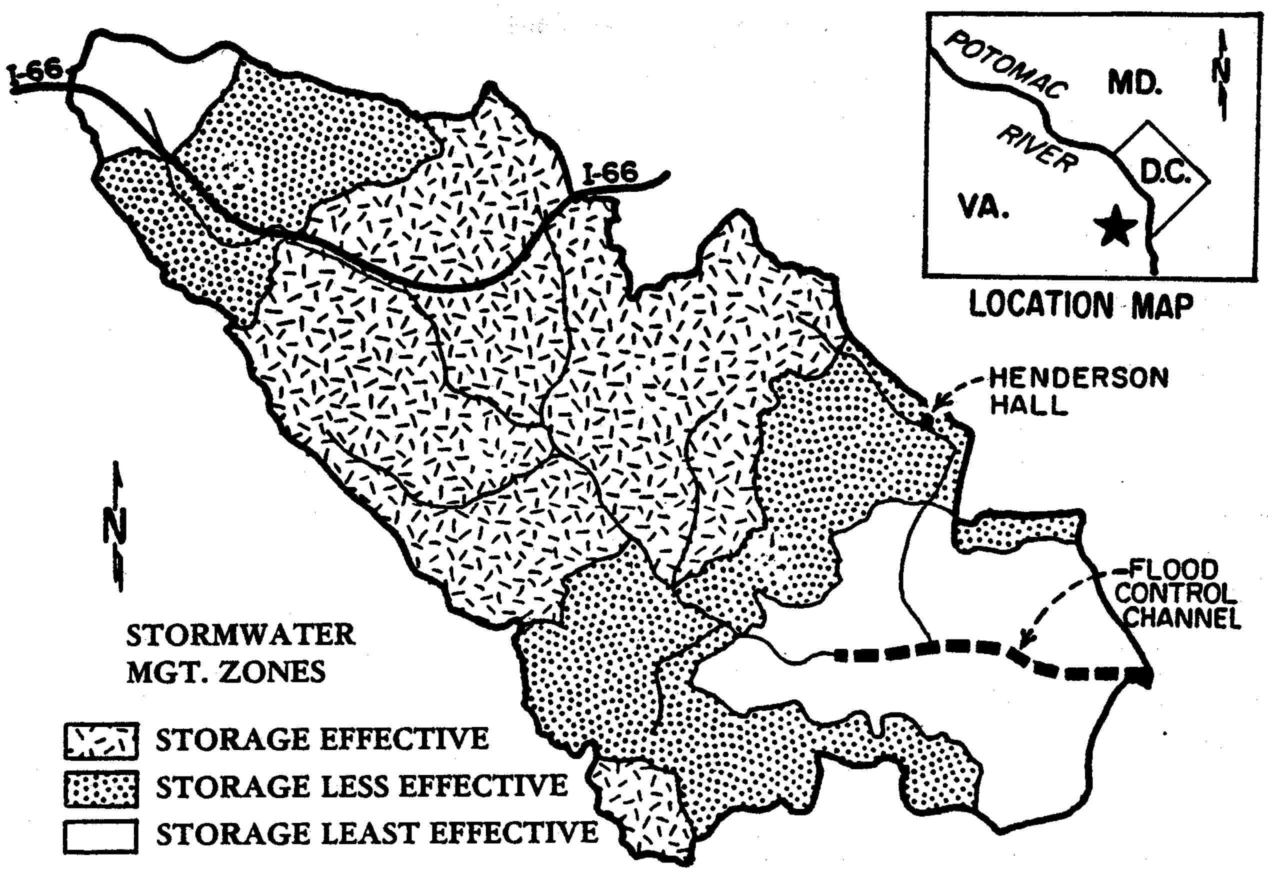


FIGURE 1
FOUR MILE RUN SENSITIVITY STUDY
STORMWATER MANAGEMENT ZONES

Source: "Four Mile Run Watershed Management Programs, Annual Report, October 1979," Northern Virginia Planning District Commission

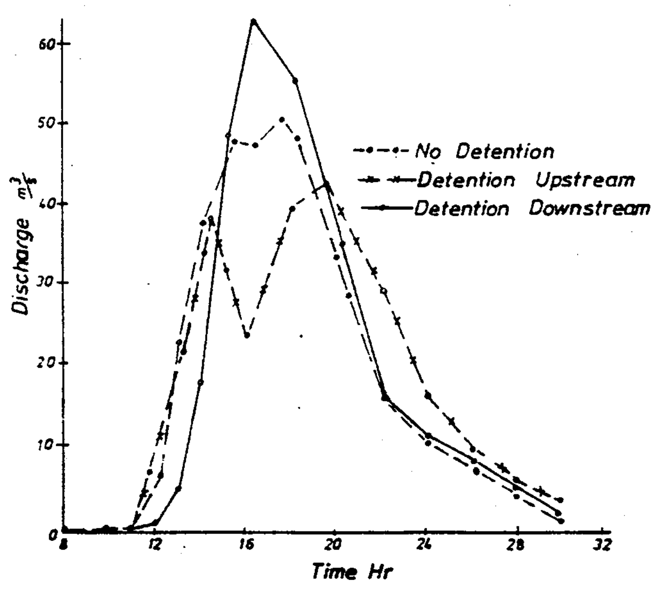


Fig. 2. Outflow Hydrographs from Tinkers Creek Watershed Showing Effects of On-Site Detention

5.0 Types of Detention Facilities: Applications

5.1 Ponds and basins in open spaces of land developments

Can enhance aesthetics of a neighborhood; however, a planned, funded,on-going maintenance program is a "must".

Multiple-purpose use is possible, thereby increasing acceptance by property owners and providing a source of maintenance funds.

Where normally-dry basins are built, positive drainage is essential to prevent unwanted swamps and weed growth--and to control maintenance costs. However, minimum grading of the natural topography is desirable and, usually, less costly.

5.2 Blue-green storage

Incorporation of stormwater storage in drainageways traversing roadways by using roadway embankments as dams and control structures.

Roadway embankments at control points should be protected to minimize bank erosion.

5.3 Major detention ponds

Careful planning and design together with emergency spillways or by-passes are essential for safety; i.e., to prevent unwanted or unexpected overflows and failure of dams.

Precautions should be taken in design and construction to minimize shoreline erosion attributable to ice, wind and wave action.

Provisions should be made in design to control sediment accumulations and water pollution in large ponds--especially where recreational use of the facility is contemplated.

5.4 Parking-lot detention

Excellent opportunity for storing excess runoff at low cost--especially on large lots (shopping centers, industrial/commercial, and multiple-family residential properties).

Should be planned so that inconvenience to pedestrians is minimized.

Storage depths should be kept below 12 inches ordinarily. In remote areas, 18 inches may be acceptable. If too deep, light watertight autos (e.g., Volkswagon beetles) may become buoyant and sail away. (Note: This happened at the Pentagon Building across from Washington, D.C.).

Porous pavement (asphalt or open-cell concrete blocks) can be used where subgrade has sufficient infiltration capacity.

Grass medians underlain by granular fill, or trenches filled with gravel or crushed stone may sometimes be used. Either subgrade soils must have sufficient infiltration capacity, or trapped runoff must be collected in pipes and conveyed at controlled rates to a drainageway or storm sewer.

5.5 Rooftop ponding

There is usually no problem with the weight of stored rain water as building codes specify roof live loads between 20 lbs/sq ft to 50 lbs/sq ft. Four inches of stored water weighs about 21 lbs/sq ft.

National building codes are generally permissive; however, design guidelines are given.

5.6 Underground storage

Can use holding tanks or oversized drain pipes with outlet controls to limit discharge rates.

Ordinarily limited to highly-congested areas where surface ponding is impractical, or on sites where topography is not suitable for surface storage.

"Pumping" may be required to discharge the stormwater stored underground.

5.7 Roadway drainage swales

Usually practical only in low-density residential areas where curbs can be omitted or designed to permit drainage into roadside ditches. Grass linings help retard flow.

In some instances, roadway surfaces having satisfactory infiltration rates can be used, but this may require extensive additional maintenance.

5.8 Ponding in yards

Although ponding in front or back yards may be required by some local governments, aesthetic and environmental conditions along with opposition by property owners deserves careful forethought.

Where soil infiltration capacity is high, subgrade disposal and groundwater recharge can be significant and, thereby, provide secondary benefits.

5.9 Check dams

Used to pool water where stream flows are highly concentrated--as in hilly areas where streambed gradients are appreciable.

Can reduce peak flows from the less-intense rainstorms and lengthen time of concentration at nearby downstream points.

Beneficial for control of stream erosion and sedimentation.

5.10 Sediment basins

Can trap the coarser fraction of materials transported in runoff (perhaps 70%±) in a reasonably short detention time interval.

Basin should be large enough to store excess runoff from a 10-year return frequency.

Provisions should be made in design to "screen" runoff to aid in removal of debris, oil and grease (where practical).

Means should be provided to handle overflows from larger rainstorms to prevent damages in nearby and downstream areas.

Benefits in reducing pipe scour, sediment accumulations, and water pollution in downstream areas can be significant.

A continuous, thorough maintenance and sediment-removal program is essential.

5.11 Stormwater disposal using dry wells and trenches

May be a viable alternative to aboveground storage, but only where soils have sufficient infiltration and storage capacities.

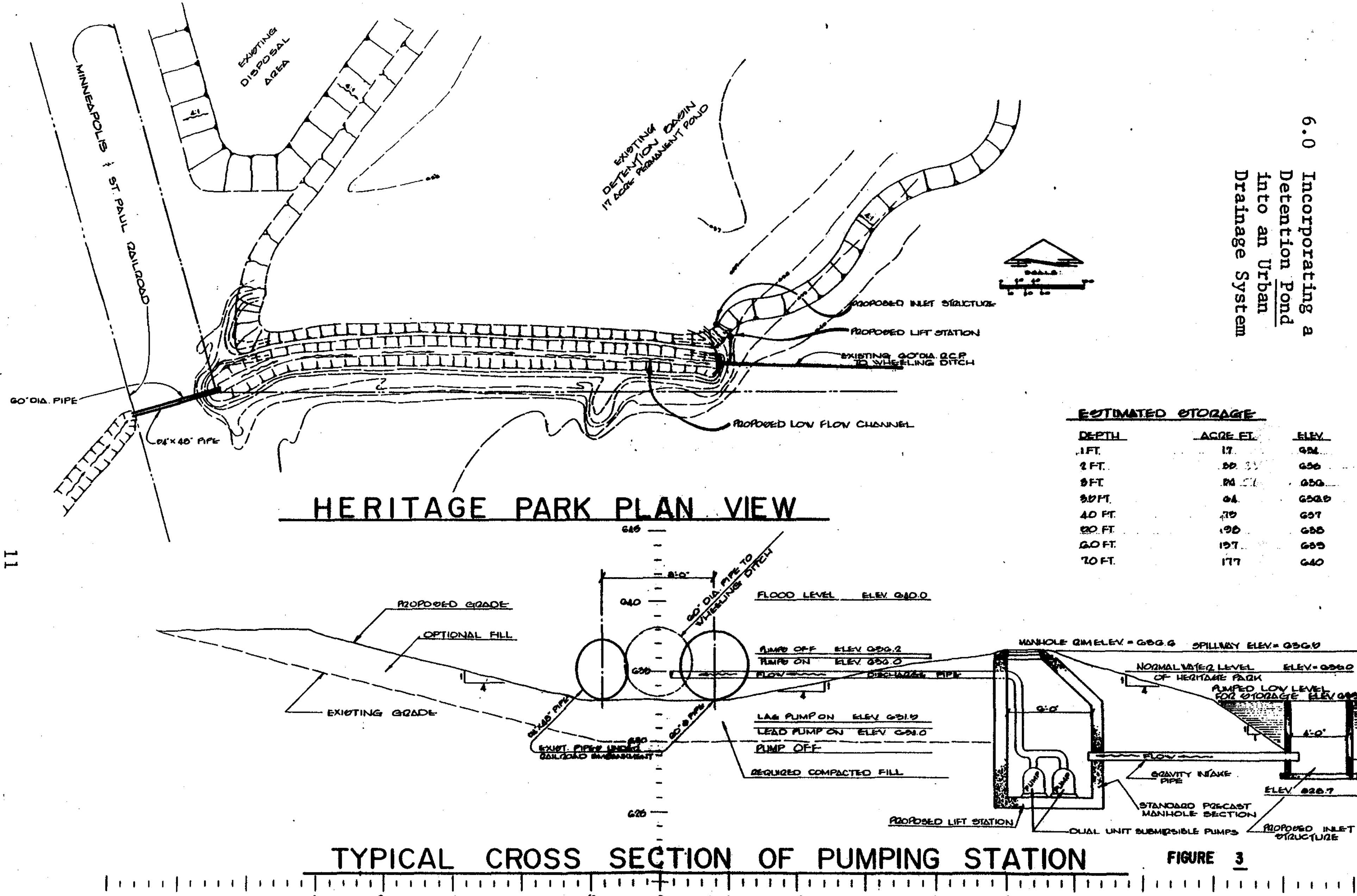
Deep soil sampling and testing is necessary to determine feasibility.

Existing groundwater levels and possible adverse impacts on local well providing potable water must be investigated.

Dry wells can serve dual purposes--runoff disposal and water storage. Care should be taken in design, construction and maintenance to minimize clogging of the permeable soil strata. Dry wells should extend into pervious soils, and be of sufficient depth to prevent seepage through downhill ground surfaces.

Lateral trenches filled with gravel or crushed stone can provide a means of storing excess stormwater and controlling peak discharges into drainageways. Protection against clogging must be provided and sub-soils must be permeable. A careful soil investigation is needed.

Groundwaters can be protected by filter sheets or bags located ahead of inlets to trap the "first flush" which ordinarily has high pollutant concentrations.



ESTIMATED STORAGE

DEPTH	ACRE FT.	ELEV.
1 FT.	17	634
2 FT.	33	635
3 FT.	49	636
3.5 FT.	64	636.5
4.0 FT.	79	637
5.0 FT.	98	638
6.0 FT.	137	639
7.0 FT.	177	640

FIGURE 3

EXHIBIT A

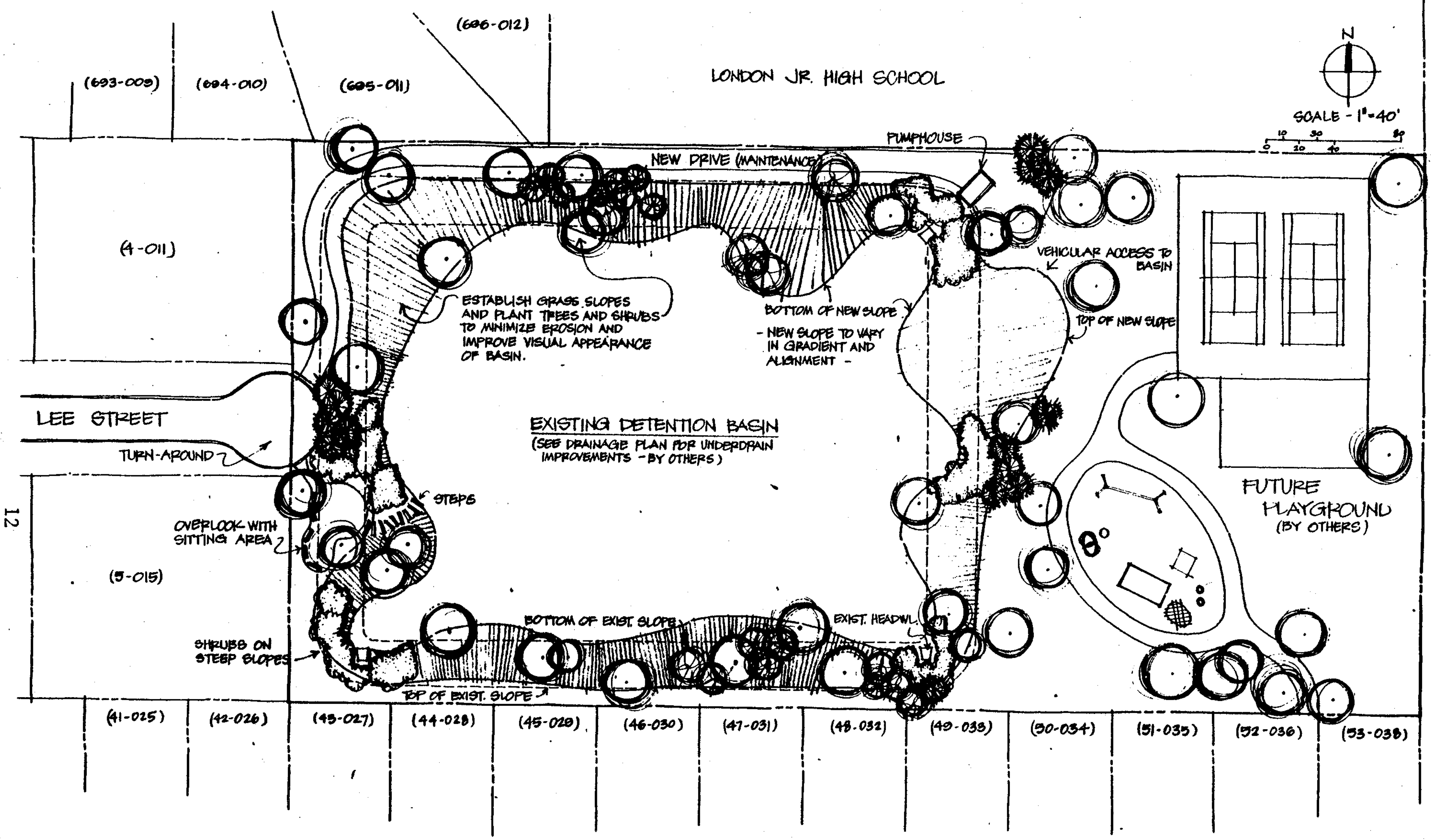


ILLUSTRATION NO. 5
HUSKY PARK DETENTION BASIN
LANDSCAPE ARCHITECTURAL IMPROVEMENTS
PREPARED BY: WM. E. ROSE & ASSOCIATES, INC.
HINSDALE, ILLINOIS

WHEELING DRAINAGE STUDY
WHEELING, ILLINOIS MAY, 1973

HERBERT G. POERTNER
CONSULTING ENGINEER

EXHIBIT B

8.0 Special Design Considerations for Public Agency Maintenance

8.1 It behooves public agencies to carefully examine subdivision plats, site plans, and engineering designs and specifications for those facilities that the public agency plans to accept for maintenance.

See Exhibit III-A at the end of this chapter for guidelines promulgated by Montgomery County, Maryland.

9.0 Precautions in Determination of Needed Storage Volumes[6]

9.1 A hydrograph of inflow must be developed and routed through the detention facility.

The four basic tasks are to develop:

.an inflow hydrograph,
.a stage-storage curve,
.a stage-discharge curve; then
.calculate storage vaolume needed so as not to exceed permissable discharge rates into the downstream drainage system.

The objectives are to store the excess volume of runoff which cannot be accomodated by the downstream drainage system without producing bank flooding, sewer surcharging, excessive erosion or scour, etc.

Peak discharges from detention facilities must either be calculated based on the hydraulic capacity of the downstream drainage system, or limited to peak flows as may be specified by the regulatory authorities.

A survey made in 1980 revealed that 45 different hydrology methods were being used in the U.S. and Canada.

Use of the Rational Method in computing flows and volumes is not appropriate except for small drainage areas (rooftops, small parking lots). Use of this method may result in calculated storage volumes that are less than actually needed. Also, application of the rational method has many limitation's (see reference 6 (pp 51-52).

Twenty percent (average) of communities do not permit use of the Rational Method for storage volum calculations.

9.2 Recommended Hydrology Methods

.Soil Conservation Service Curve Number Method[7] (TR-55) for watersheds less than 2000 acres;
.Soil Conservation Service Computer Model TR-20 for watersheds exceeding 2000 acres;
.Unit hydrographs, rainfall-runoff simulation models of which there are many;
.Modified Rational Method (may be useful for smaller drainage areas).

9.3 "Preliminary" Storage Volume Calculations

To expedite making a preliminary estimate of the needed storage capacity, a simplified design procedure can be used.[8] Standard triangular hydrograph shapes are assumed for both inflow and outflow hydrographs. Using this assumption, it is unnecessary to route the flood flow through the reservoir.

The "triangular" assumption is shown in Figure 3.

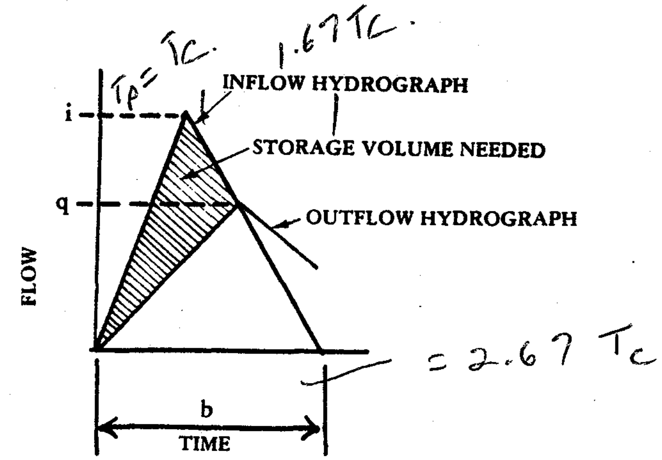


FIGURE 3

ASSUMED TRIANGULAR SHAPES
OF INFLOW AND OUTFLOW HYDROGRAPHS
FOR PRELIMINARY DESIGN

The needed storage volume can be found from the relationship:

$$S=0.5\, b\, (i-q)$$

where

- S = storage volume needed
- i = peak inflow rate
- q = peak discharge rate, and
- b = duration of inflow to basin

all espressed in consistent units.

The accuracy of this manual method was checked with several computer methods using a total of 1,029 different combinations of variables.

Watershed sizes ranged between 0.1 to 1.9 square miles; recurrence intervals between 10 year to 100 year storms; storm durations between 5 to 720 minutes; and outlet pipe diameters from 12 to 82 inches.

The method described above, using triangular hydrographs, was found to be accurate enough for preliminary design, or preliminary review of calculations for storage volume needed.

10.0 "Safety" in Design of Detention Facilities

10.1 Approaches:

- .keep people off the detention site; however, this is not always possible or desirable; e.g., multiple-purpose facilities,
- .provide escape aids,
- .make the onset of the hazards gradual, and
- .eliminate the hazards.

10.2 Outflow Structures

The force (varies with depth below water surface) of the water against a person's body may push that person into an outflow structure, or trap the person beneath the water surface where bottom discharge is used.

Several suggestions, illustrated in Figure 4, were given in a paper.[9] These appear to be excellent design suggestions to help eliminate the hazard of drowning.

10.3 Earthquake hazards; Dam or Embankment Failure

A seismic zone map for the United States is shown in Figure 5. The legend defines the probability of seismic action and damage by numerals indicated on the map.

This type of map is useful to engineers and geologists in determining what special provisions may be required in designing dams and embankments for water storage facilities.

Based upon analyses of earthquake hazards, the design engineer may decide to construct a dry basin rather than a pond. Or, it may be decided not to construct any type of water storage facility because of the risk and high costs entailed.

10.4 Railings and Fences

The use of fences to keep people off detention sites may sometimes be advisable, especially where small children are present nearby (elementary school, day care center, apartment dwellings, etc.).

Railings should be used at headwalls of inflow and outflow structures and at other hazardous structures near the shoreline.

11.0 Multiple-purpose Uses of Detention Facilities

Stormwater storage facilities can be designed and constructed to provide opportunities for multiple-purpose use, thereby making such flood control facilities a more important community asset. Such projects help meet other public needs and are recognized with great favor by local residents. An example of multiple-purpose use is shown in Exhibit C. In addition to flood control, the project includes potable water systems and recreation.

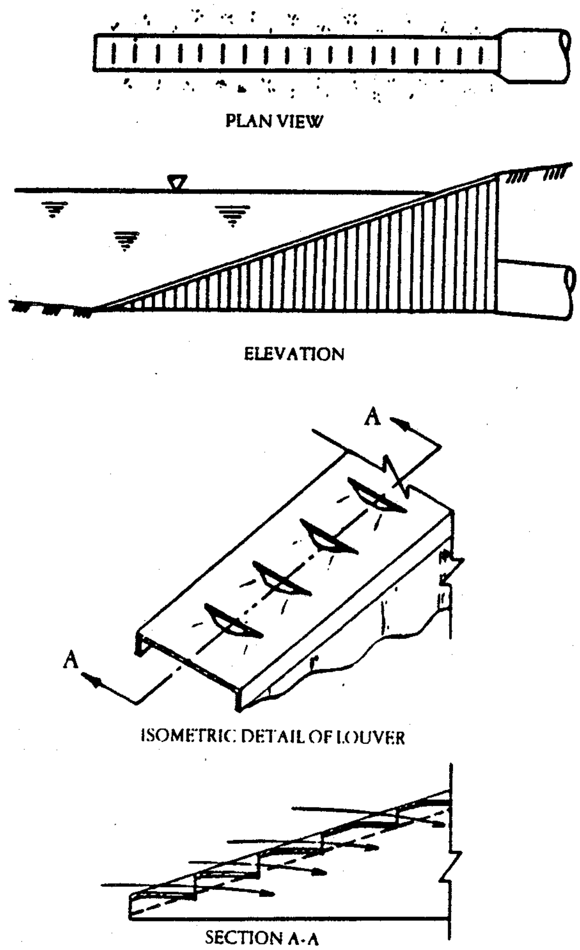


FIGURE 4a
NARROW FLUME OUTLET FOR DETENTION PONDS

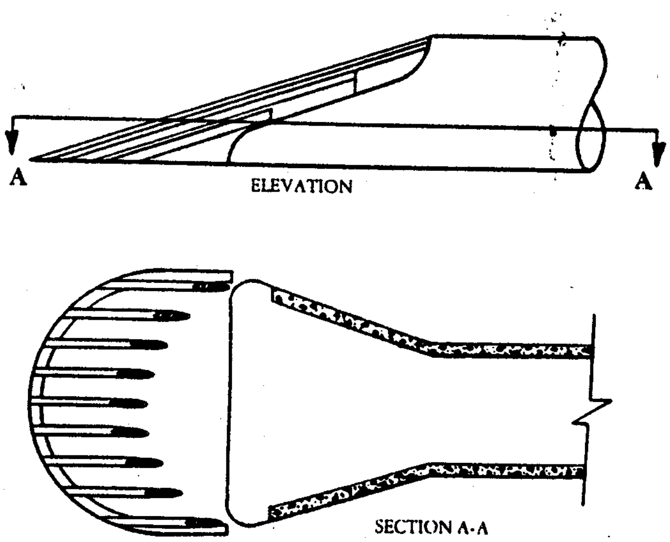


FIGURE 4b
CURVILINEAR TRASH/SAFETY RACK FOR STANDARD FLARED END SECTIONS

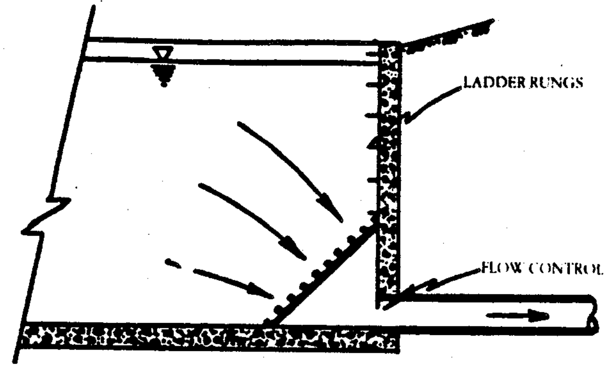


FIGURE 4c
SAFETY RACK FOR SUBMERGED OUTLET
(Rails May Be Horizontal to Facilitate Escape)

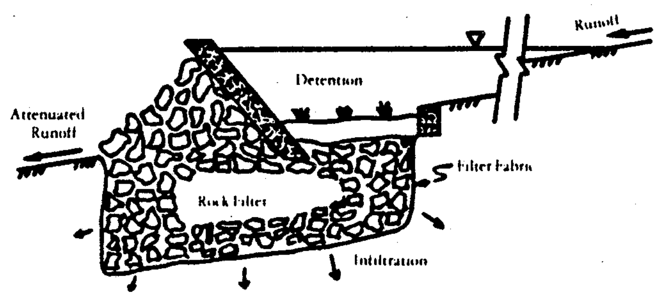


FIGURE 4d
POROUS DAM FOR DETENTION POND WITH LOW VELOCITY DISCHARGE

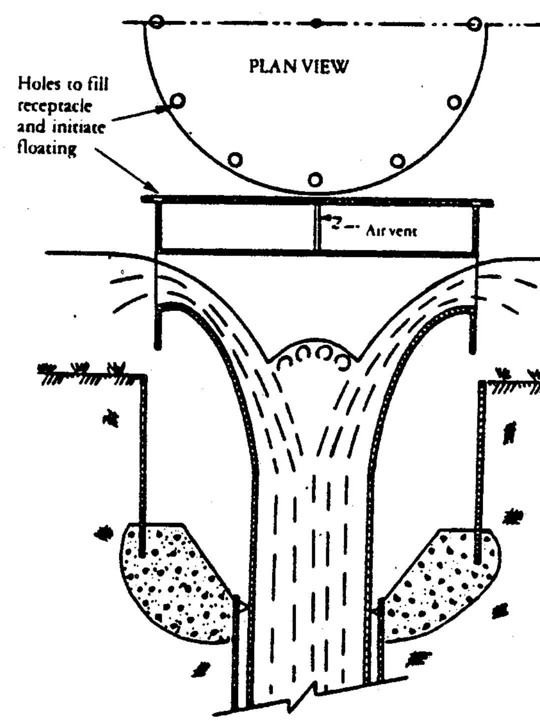
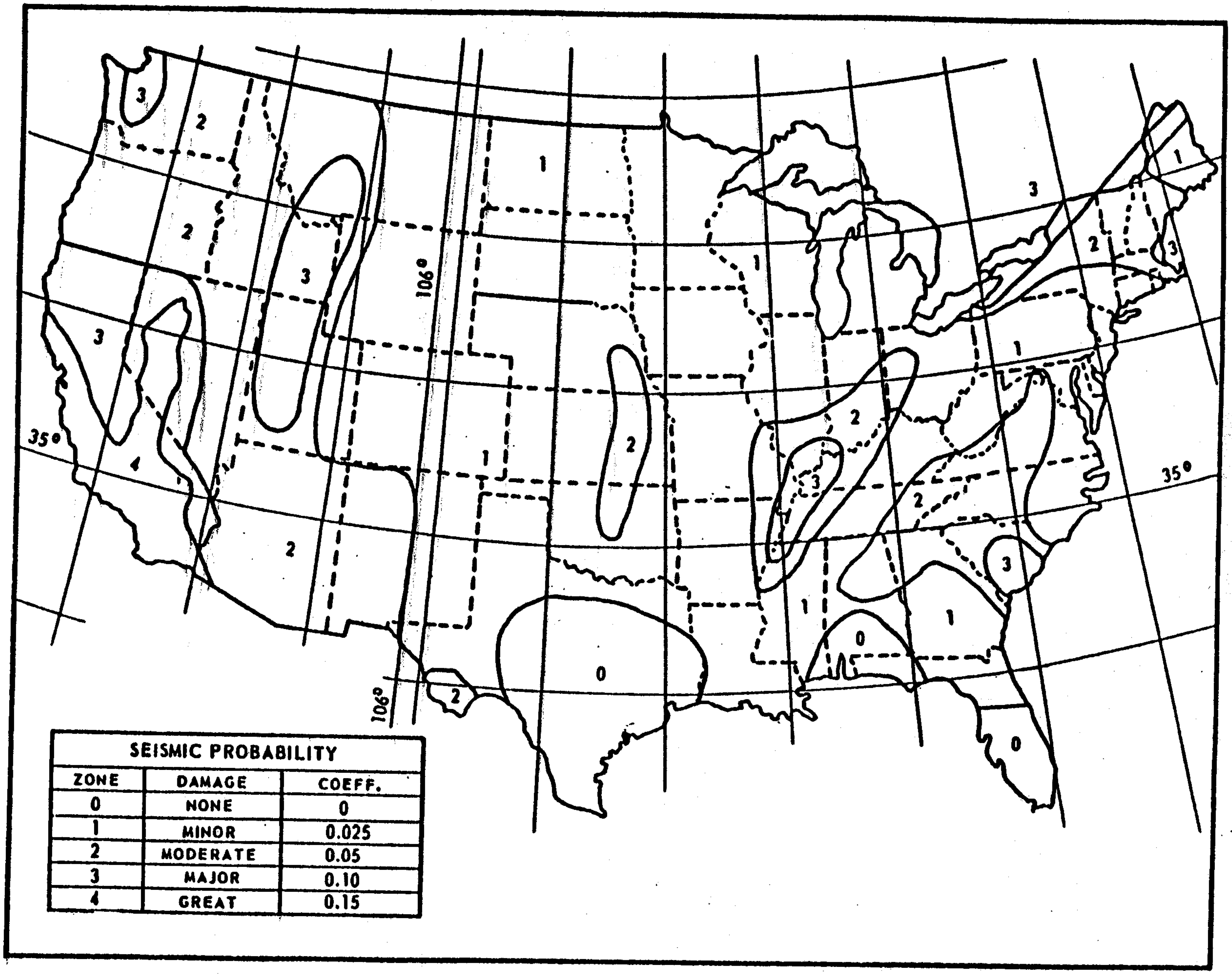


FIGURE 4e
FLOATING INLET WITH RECEIVING RECEPTACLE RECESSED

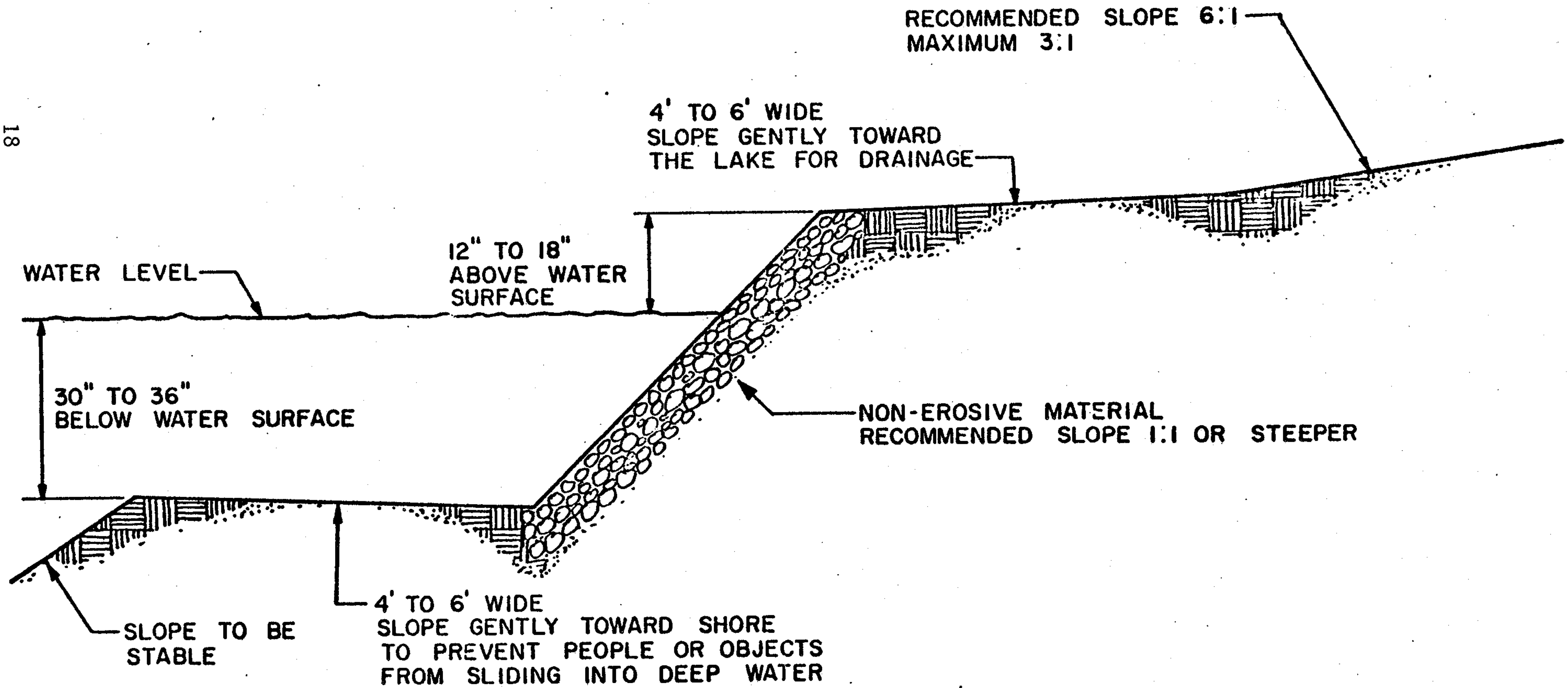
Source: "Safety Considerations in Urban Storm Drainage Design" Marcy, S. J. and Flack, Proceedings, Second International Conf. on Storm Drainage, Univ. of Illinois, Urbana, 1981



SEISMIC PROBABILITY		
ZONE	DAMAGE	COEFF.
0	NONE	0
1	MINOR	0.025
2	MODERATE	0.05
3	MAJOR	0.10
4	GREAT	0.15

FIGURE 5
SEISMIC ZONE MAP OF CONTIGUOUS STATES

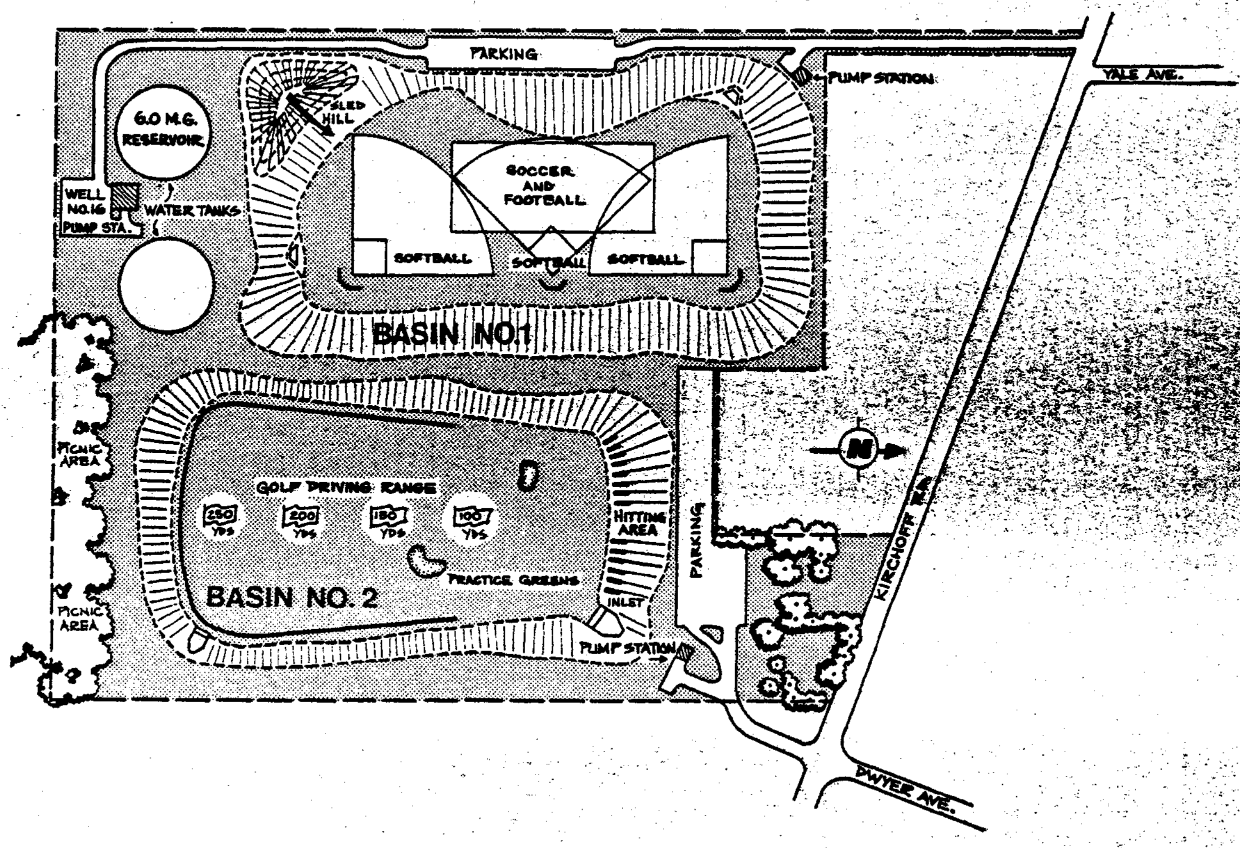
CROSS-SECTION OF LAKE SHORE



METROPOLITAN SANITARY DISTRICT
OF GREATER CHICAGO

DEC 1973 M.P

FIGURE 6



Flood Control Program

Wilke-Kirchoff Project **Arlington Heights, Illinois**

The Village of Arlington Heights has undertaken a major flood control program with a total construction cost of approximately $20 million, all of which is being financed with local funds.

Design concepts for the entire program call for multiple use of all properties wherever possible (i.e., flood control, recreational, and potable water systems). The Wilke-Kirchoff Project is a typical example, providing for all three uses on a 37-acre site. The First Wilke-Kirchoff Project provided 100 acre-feet of storm water detention; a 10,000 gpm potable water booster station taking suction from a 6,000,000-gallon reservoir; and lighted baseball, soccer, and football fields.

The Second Wilke-Kirchoff Project, presently under construction at a cost of $10 million, is the largest single project and consists of a storm sewer system, storm water detention basin, and storm water pumping station to serve a developed urban area of 539 acres. The 14-acre (140 acre-feet) detention basin will normally be dry and is being constructed with gentle side slopes so that it can be used for a miniature golf course and driving range with minimum maintenance costs.

An 8,100 gpm pumping station for dewatering of the basin is under construction at the northeast corner of the basin.

The storm sewer collection system consists of 35,000 feet of sewer ranging in size from 12-inch to 132-inch diameter. The storm sewers are located under developed village streets with 1,300 feet of 120-inch and 132-inch sewer being constructed by tunneling methods to minimize disruption to existing streets and utilities. Additional lengths of smaller diameter sewers are also being constructed by tunneling methods.

References

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2. Aron, G. and Lakatos, D. R., Penn State Runoff Model--Users' Manual, January 1980 Version, Civil Engineering Department, Pennsylvania State University, University Park, January 1980.

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4. Duro, J. O., "On-Site Detention: a Stormwater Management or Mismanagement Technique?" Proceedings, Second International Conference on Urban Storm Drainage, University of Illinois at Urbana-Champaign, June 15-19, 1981.

5. Joint Committee of the American Society of Civil Engineers and the Water Pollution Control Federation, Design and Construction of Sanitary and Storm Sewers, ASCE Manual of Engineering Practice No. 37 (and WPCF Manual of Practice No. 9) New York, NY., 1969.

6. Poertner, H. G., et al, Urban Stormwater Management, American Public Works Association, Chicago, 1981, Chapter 5.

7. Soil Conservation Service, U.S. Dept. of Agriculture, Urban Hydrology for Small Watersheds, Technical Release No. 55, January 1975 (revised 1981).

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PERSPECTIVES and GUIDELINES for DRAINAGE SYSTEM DESIGN

Report of a Survey by the American Public Works Association

Conducted by Herbert G. Poertner

Abstract

A perspective is given of current practices across the United States, and in some Canadian cities, in planning, designing, constructing, operating, maintaining and financing facilities and programs for urban runoff control. Facilities constructed on, or near, land development sites are discussed. The paper is based upon analyses of responses by 325 public agencies in a stormwater management survey completed in May 1980 by the American Public Works Association (APWA). Stormwater detention is highlighted. The survey is part of APWA Research Project 79-1, the objective of which was to prepare a manual on urban stormwater management.

Summary of Findings

Nearly half the 325 public agencies surveyed have established some requirements for stormwater management. Detention storage of excess runoff is often a part of overall stormwater management plans. Many local public agencies have established policies and criteria on control of surface runoff, regulation of floodplains and control of soil erosion and sedimentation.

Financing stormwater drainage facilities is most often accomplished by issuing general obligation bonds. Other methods, in sequence of popularity, are: appropriations from a public agency's annual budget, special assessments; and federal grants.

The design rainstorm most often specified by local agencies is the 10-year storm, followed in popularity by the 5-year, then the 2-year and 25-year rainstorms, the latter two being specified by 32 agencies and 31 agencies respectively. For detention basin sizing, the 100, 10 and 25-year storms are most used.

Almost half of the agencies responding permit drainage designs to provide for some street flooding. Most common are flooding depths ranging from 6 to 8 inches (15.2 to 20.3 centimeters).

Two hundred nineteen public agencies, more than two-thirds of the agencies surveyed, reported 12,683 stormwater storage facilities in use. This confirms a significant increase in use of storage during the eight-year period since 1972 when a similar APWA survey revealed 1,410 storage facilities being used by 99 public agencies. In the earlier survey,[a] only 43 percent of the respondents reported the use of storage.

About 48 percent of the 12,683 storage facilities reported in the 1980 survey are dry basins, 25 percent are parking lot basins, 19 percent are ponds and 5 percent are rooftops. The agencies reported, 1,513 stormwater storage

a. See APWA Special Report No. 43, Practices in Detention of Urban Runoff, pp 88-89 facilities in use for groundwater recharge and 378 other facilities having multiple-purpose uses.

Twenty agencies reported having 100 or more storage facilities of various types, and six agencies reported having more than 500 facilities. The maximum number reported is for Cook County, Illinois (outside the City of Chicago) where 1,700 storage facilities are estimated to be in use.

Local physiography is a factor that influences the type and size of storage facilities constructed by communities; however, regardless of physiographic characteristics, detention facilities are found in all areas of the United States and many parts of Canada.

"Reducing downstream flooding" was reported as the principal objective for requiring stormwater storage. Other important objectives, in order of importance, are reducing: drainage system costs, on-site flooding, and soil erosion. Following in order of importance are reducing water pollution, and improving aesthetics, recreation, groundwater recharge, surface water supply, and irrigation.

Twenty percent of the agencies will not accept the use of the Rational Formula as a satisfactory method for making hydrologic computations needed for the final design of a detention facility. A total of 45 different methods were reported in use for predicting runoff rates and developing inflow hydrographs for storage facility sizing. The "curve number method" (2) of the Soil Conservation Service (USDA), unit hydrograph methods, and the Modified Rational Formula method are favored.

About 40 different computer models are used for hydrologic computations, the most popular of which are: the Soil Conservation Service method described by TR-20(3), SWMM(4), and HEC-1(5). Other methods used are ILLUDAS(6), and STORM(7).

Emergency overflow structures are required by two-thirds of the agencies having storage facilities; however, less than half have established other safety requirements.

Less than one-fourth the agencies having detention facilities require low flow-by-passes, and only ten percent require forebays for sediment collection. The most popular outflow controls are weirs and spillways.

Weed growth in detention ponds and basins was reported as the most troublesome maintenance problem. Mowing grass and controlling sediment accumulations are other significant problems. About two-thirds of the agencies with 10 or more detention facilities have a periodic inspection program. Access to facilities is most often provided by easements; however, two-thirds of the agencies sometimes require dedicated rights-of-way.

References

1. American Public Works Association, Urban Stormwater Management, Chicago, 285 pp, 1981.

2. Soil Conservation Service (USDA), Urban Hydrology for Small Watersheds, TR-55, 91 pp., January 1975.

3. Soil Conservation Service (USDA), Computer Program for Project Formulation--Hydrology, TR-20, 1969.

4. University of Florida, Storm Water Management Model User's Manual, Version II, U. S. EPA Environmental Protection Technology Series, EPA-670/2-75-017, Cincinnati, Ohio 45268, 350 pp., March, 1975.

5. Hydrologic Engineering Center, Corps of Engineers, "Hydrologic Engineering Methods for Water Resources Development: Volume 4, Hydrograph Analysis", Publication HEC-IHD-400, Davis, California, 122 pp., October, 1973. (NTIS No. AD 774 261).

6. Terstreip, M. L., Stall, J. B., The Illinois Urban Drainage Area Simulator, Bulletin 58, Illinois Water Survey, Urbana, 90 pp., 1974.

7. Hydrologic Engineering Center, Corps of Engineers, "Guidelines for Calibration and Application of STORM", Training Document No. 8, 609 2nd Street, Davis, California 95616, 48 pp., December, 1977.

Part I Discussion of Pumping Station Design Procedures

1.0 Introduction

In most localities, storm water pumping stations only operate for a relatively short period of time during a year. This means that a substantial capital investment must sit idle for long periods of time. Therefore, the design and operation of storm water pumping stations provides a most promising opportunity for cost reduction. Potential savings are even more promising in areas where storms are less frequent.

The merits of providng storage to reduce peak pumping rates of pumping stations have long been recognized by engineers. To control the costs of storm water projects, engineers are now examining potential saving much more closely. In order to achieve meaningful cost reductions, savings must be accomplished in both the construction cost, and the maintenance and operations cost areas.

Initial costs can be reduced by providing storage to reduce the peak pumping rate. This will produce savings in the cost of the pump, pump motor, and instrumentation; additional savings can be achieved by reducing the size of piping and valves. Substantial savings can occur if the number of pumps is reduced. These savings will be offset by the cost of providing storage; however, in many cases, a net savings will occur if the storage can be provided at a low cost.

Maintenance and operation costs can be lowered by reducing the fixed electrical charge assessed by most electrical utilities. This charge is basically for the electrical capacity that the utility must maintain to service the pumping station and is usually proportional to the horsepower of the station. Since horsepower is directly proportional to the pumping rate, any reduction in the pumping rate will be reflected in the fixed electrical charge.

Analyzing the effect of storage on reducing the pumping rate using manual calculations is a tedious, time consuming procedure. There are a wide range of storage and pumping rate combinations that will provide an adequate design. Due to time constraints, engineers usually only investigate the more obvious combinations. The purpose of this publication is to provide a collection of programmable calculator programs that will quickly analyze the problem, thus allowing engineers to investigate numerous combinations of storage and pumping rates.

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* Source: "Hydraulic Design of Stormwater Pumping Stations Using Programable Calculators (Texas Instruments TI-59 Calculator Design Series No. 5). U. S. Department of Transportation, Federal Highway Administration, Washington, DC, May 1982, 139 pp.

2.0 Development of a Mass Curve Routing Procedure

The merits of using storage to reduce peak flows have been discussed in the previous section. A generalized case is selected for illustration because the actual pumping station case may be complicated by the varying pumping rates and discontinuities as the pumps turn on and off. This is shown in Figure 1.

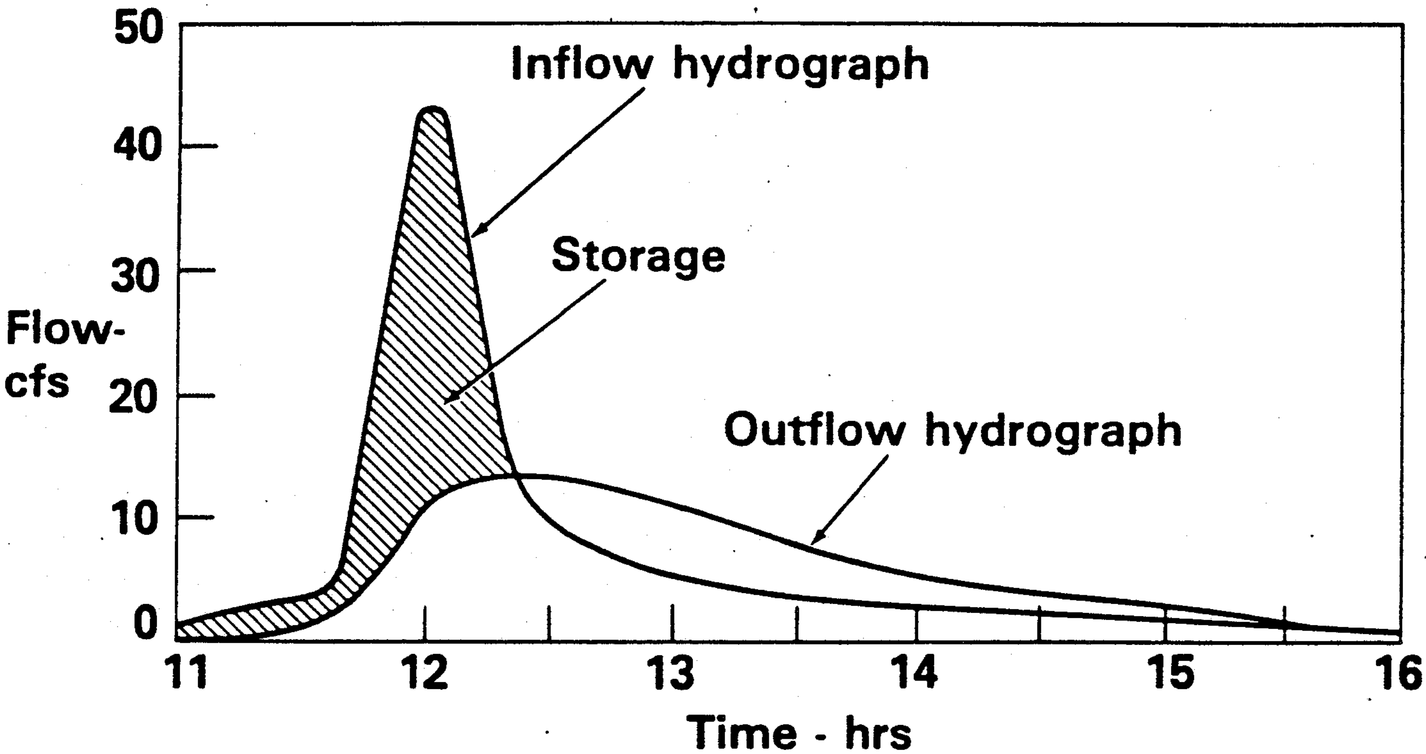

Figure 1. Use of Storage to Reduce Peak Flow Rate

The shaded area between the curves represents the volume of stormwater that must be stored to reduce the peak flow rate. Storage exists in natural channels, storm drain systems, constructed basins or forebays, and in storage boxes. Engineers must be able to identify and analyze the effect of storage on the discharge rates from the pump station.

Designers must establish the interrelationship between three separate components. First, the inflow hydrograph must be determined for the contributing watershed. Second, the volumetric storage capability of the storage facility must be identified. Third, the stage-discharge curve of the pumps must be determined. Once these three components have been established, a mass curve routing procedure can be used to analyze the problem. This routing procedure will be developed in the following sections.

An example problem is utilized to illustrate the development of the routing procedure; the inflow hydrograph used for this example problem is depicted in Figure 2.

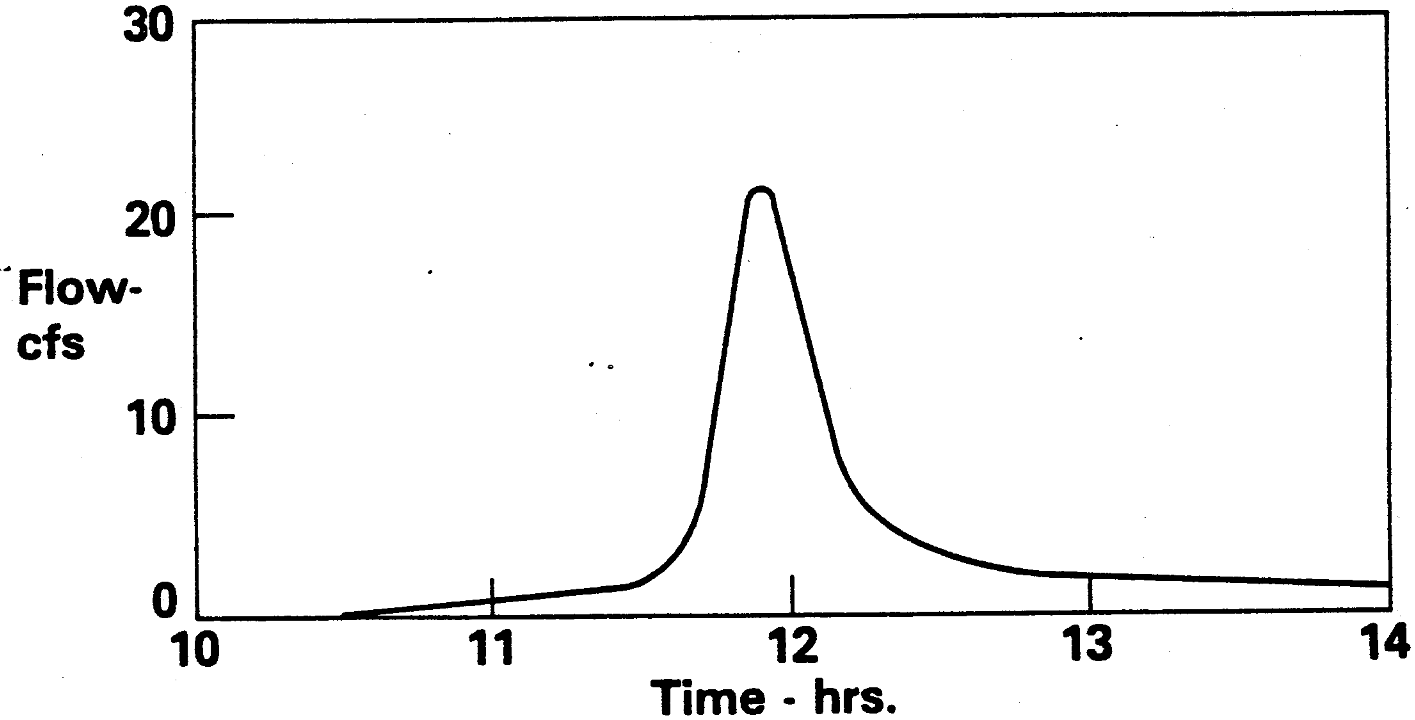

Figure 2. Inflow Hydrograph for Example Problem

3.0 Estimating Required Storage and Pumping Rates

Because of the complex relationship between the variables of pumping rates, storage, and pump on-off settings, a trial and error approach is usually necessary for estimating the pumping rates and storage required for a balanced design. There is a wide range of combinations that will produce an adequate design. A desirable goal is to maximize storage capacity so as to minimize pumping capacity.

Some approximation is necessary to produce the first trial design. One approach is shown in Figure 3.

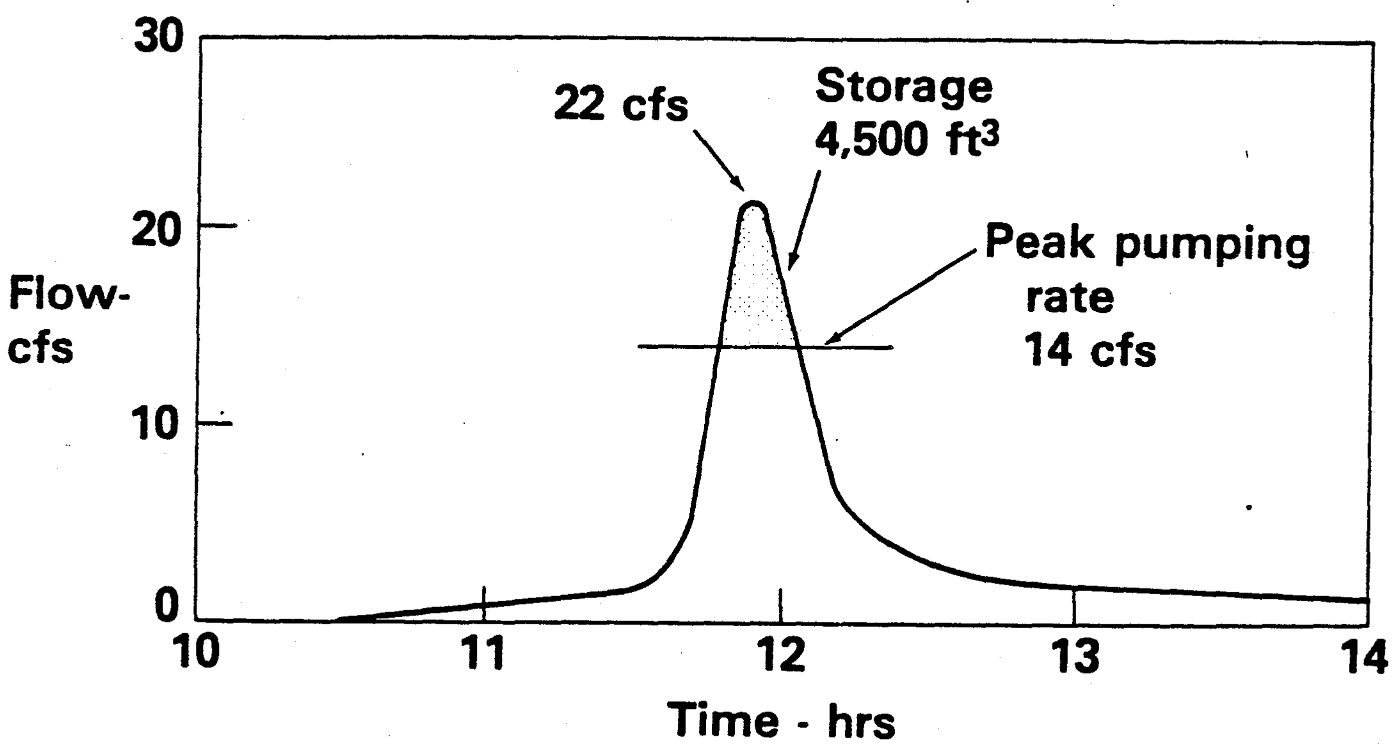

Figure 3. Estimating Required Storage

In this approach, the peak pumping rate is assigned and a horizontal line representing the peak rate is drawn across the top of the hydrograph. THE SHADED AREA ABOVE THE PEAK PUMPING RATE REPRESENTS THE VOLUME OF STORAGE REQUIRED ABOVE THE LAST PUMP-ON ELEVATION.

The number of pumps and their respective pumping rates are selected along with the pump on-off settings, and the storage basin's trial dimensions are assigned to produce the required volume of storage, represented by the shaded area in Figure 3, above the last pump-on elevation.

For the example problem, a peak pumping rate of 14 cfs was assigned; this will be accomplished by two 7 cfs pumps. The pumping rate is plotted as a horizontal line, and the shaded area is measured, determining the required volume (4,500 ft^3) above the last pump-on elevation.

ABSTRACT of paper entitled:

Introduction to

"Manual for Highway Storm Water Pumping Stations"

The Federal Highway Administration has recently published the above Manual, which is a comprehensive display of current practice in the subject, with over 600 pages and more than 200 illustrations. Examples of stations through the nation receive attention and there is some forecast of future trends. The work described was funded entirely by the Federal Highway Administration.

This paper serves to introduce the Manual by making use of some of its basic material, illustrated by visual aids specially drawn in simplified form. Topics covered include collection systems, types of station, pumps, electrical systems and controls. There is a brief description of various alternates and combinations of power sources including emergency generators. The paper concludes with a series of photo slides illustrating various aspects of the Manual content.

The paper is complete in itself and is intended to be fully acceptable and of interest to those who have no need for the more comprehensive and detailed content of the manual but who may have a general interest in the subject of pumping stations. It can serve as an introduction to the subject or as a refresher, or may stimulate interest in the Manual among those more heavily involved with pumping stations.

To be presented on Jan. 17, 1983, at Sheraton Washington Hotel, Washington, DC, during the 62nd Annual Meeting of the Transportation Research Board, National Research Council.

Respectfully Submitted,

William F. Lever

William F. Lever, President
William F. Lever and Associates
Consulting Engineers
A California Corporation

"Manual for Highway Storm Water Pumping Stations", Report No. FHWA-IP-82-17, is available from:

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U.S. Government Printing Office
Washington, DC 20402

Vol. I
S.N. 050-001-00254-0 .... $9.50

Vol. II
S.N. 050-001-00253-1 .... $7.50

CONSTRUCTION COSTS

It has not been possible within the scope of our work to accumulate cost data which would permit meaningful comparisons to be made between the various types of station illustrated. By various types we mean the rectangular wet pit, the circular wet-pit constructed by the caisson method, the submersible pump type, or the dry-pit station with storage box.

The cost of identical construction will vary from one part of the nation to another, and the construction market conditions prevailing at the time of inviting bids may cause significant variations. In fact, due to the economy and current 1983 prices being so depressed, even the 1978 prices we are going to refer to do not seem significantly out-of-date.

The seven preceding photographs showed the Westside Pump Station, Long Beach, Ca., and complete data is available from the Bid Form which was utilized. Bids were taken in January 1978 and a contract was awarded for the entire construction, based on the total price bid by the lowest of five bidders. Construction was completed in April 1979 for substantially the same figure as bid, there being no significant change in plans or extra costs incurred.

The lowest total price bid for the station of 181 cfs Q was $1,100,106.00, representing a unit cost of $6,078 per cfs of pumping capacity. The construction extended from a four-foot length of 75" diameter R.C.P. inlet pipe upstream of the station to the downstream end of the discharge manifold. The station was constructed complete as one unit, with separate contracts being awarded for the collection lines upstream of the station and the discharge line downstream of the station. There was a difference between the low bid and second low bid of $33,387.00 while the high bid of five received was $1,293,261.70.

Some yardstick estimates of the cost of pumping stations can be expressed in dollars per cubic foot per second pumped. A range of $3,000 - $8,000 per cfs in 1983 dollars is suggested. The unit cost for small stations will tend to be higher, while larger stations will benefit from economies of scale, unless elaborate design and complexities nullify this advantage. Costs of forebay or storage box should be included as part of the cost of the station. Costs of collection system upstream or discharge lines or channels downstream of the station are usually accounted for separately from the station. Their cost is not included in the range quoted. A rectangular wet-pit station of 400 cfs Q with 4 engine-driven pumps and 2 electric pumps, also in Long Beach, CA., bid in December 1980 for approximately $2.6 million, or $6,500 per cfs. Equipment and features were generally similar to Westside, but of larger size.

* * * * * * * * *

Mike Zeller - Simons-Li + Assoc -

Design Practices -

Inlets - Similar - Fed Hwy Publication - HEC12 new edition
- normal -10 yr. storm, 14' segment -
- 2 yr. Min -
- 100 yr. with risk -

- Pima County based on LA. Flood -
Constr.

- Arizon Transp. Manual on Drainage -

- Amer. Iron + Steel Institute - Manual

Culverts - Hec 5 + Hec 10 Publications -
Appr - velocitys (Critical or Super Critical)
(get better entrance results than normal -

Outlets Hec. 14; ADOT, + SCS -
culverts + channels

Rip Rap - Basin + Channel lining - no rock
in Tucson - Gabions, Gunite or Soil Cement -
See Pima County -

Large Flows - Bur. Rec. - Hyd design Stilling Basins
+ Energy dissipators - (4-5.00)

Open Channel Flow - Velocities High - For 10 or
greater -

Soil Cement - 10-15% by weight - opt moisture
6/8" lifts - compacted -

Small channels — Peoria City —

— F# 1 or greater — need Bank linings —
— Rip Rap, concrete 6" mesh (1:1 Slope) gabions
most — or conc. Block mattresses —
— (500 - 5000 cfs —)

— 10 sq. mi or less

Hydrology — City Hyd. Manual for Engr. Design
— City simpler procedures & diff previously
in Rainfall intensities — Higher for
— Short duration — limited to 2 sq mi.
or less.

— Hydraulics — mannings — undeveloped & natural channels —

— otherwise Hec # — Backwater effects & changes in water Surface profile —

water & Sediment continuity

— erosion & sedimentation concerns
very imp — must be controlled
— by long term impact on erosion
— must identify ultimate slope — require grade control or check structures to control degradation —

— 100 yr. flood typical design Std for open channel flow — can be 4000 cfs on 2 sq mi. area — 9-5 x other areas.

— Basins to reduce flows —

Problem: How to develop standards for opt. combination of flow - storage —

ADWR - Dam Safety] - 6' + 50 ac or 25' hig
State Jurisdiction] - PMF or 1/2 PMF —

— Sedimentation — 40,-50,000 PPM → 100,000
10% of flow ~~with~~ is Sediment —
Problem - destabilization downstream —

— Pima Cty Guidelines Basins —
Criticism - drainage flood water more nondle, flooding hazards —

— Balanced = now adequate - add will make worse —

—

SCHEDULE

ENGINEERING DESIGN OF URBAN DRAINAGE SYSTEMS

Presented by Stormwater Consultants
Bolingbrook, Illinois

Part A SPEAKER: Dr. Chin Y. Kuo, P.E.*

AM	
7:45	Registration and Literature Viewing
8:00	Course Overview and Introduction
8:15	Inlets--Section 1*
9:30	Refreshment Break & Literature Viewing
9:45	Closed Conduit Systems--Section 2
11:00	Workshop Problem
11:30	Lunch Break

PM	
12:30	Open Channel Flow--Section 3
1:00	Design of Culverts--Section 4, 5
2:00	Outlet Protection--Section 6
2:30	Refreshment Break & Literature Viewing
2:45	Riprap and Channel Linings--Section 7
3:10	Energy Dissipators--Section 8
3:30	Miscellaneous Topics--Section 9, 10
4:30	Adjourn for the day

Note: Course notes for Part A are divided into ten sections

* Professor of Civil Engineering, Virginia Polytechnic Institute and State University Blacksburg, Virginia 24061 Phone: (703) 961-7153

Part B SPEAKER and COURSE LEADER: Herbert G. Poertner, P.E.**

AM	
7:45	Literature Viewing
8:00	[1]Stormwater Detention--Chap. I
8:45	Detention: Planning & Design--Chap. III, pp 1-10
9:30	Refreshment Break & Literature Viewing
9:45	Incorporating Storage into a Drainage System--Chap. III, pp 11-12
10:30	Multiple-purpose Use and Safety Precautions-- Chap.III, pp 15-20
11:15	Improved Inlets for Highway Culverts[2]
11:30	Lunch Break

PM	
12:30	Stormwater Pumping-I p7, III pp 11-12,Apdx.
* 1:00	Design Practices, Criteria, Guidelines --Chap. III pp 13-14; Chap. IV
2:30	Refreshment Break
2:45	Hydraulics of Energy Dissipators (3)
3:15	Construction Materials
3:30	Costs of Drainage Systems--Chap. II
4:00	Miscellaneous Topics
4:15	Course Evaluation; Certificates
4:30	Adjourn

** President, Stormwater Consultants, 3 Westchester Ct. Bolingbrook, IL 60439

NOTES:

(1) Course notes for Part B comprise four chapters and an appendix.

(2), (3) Films produced by the Federal Highway Administration, U. S. Department of Transportation.

* Guest Speaker--see Registration List (roster) for name.

Problem 1

Given: Pavement $n = 0.016$ on a continuous grade with two 12′ lanes draining into a 2-foot width gutter with vertical curb; longitudinal slope $S_0 = 0.03$; pavement cross slope $S_x = 0.03$; permissible spread on pavement $T = (\frac{1}{2}$ of traffic lane$) + ($width of gutter$)$.

Find: Spacing of 2-foot wide efficient grade inlets.

Solution:

1. Spread on pavement $= T = \frac{1}{2}(12) + 2 = 8'$
2. $Z = \frac{1}{3\%} = 33.33$, $\frac{Z}{n} = 2{,}083$
3. Depth at curb $d = \frac{T}{Z} = \frac{8}{33.33} = 0.24'$
4. From page 7, with $d = 0.24'$, $\frac{Z}{n} = 2{,}083$, $S_0 = 0.03$
   $Q = 4.3$ cfs
5. For a 2-foot grate with efficient openings, the flow intercepted is computed as:

   page 7: $X = 2'$

   $$d' = d - \frac{X}{Z} = 0.24 - \frac{2}{33.33} = 0.18'$$

   let $d' = 0.18'$, $\frac{Z}{n} = 2{,}083$, $S_0 = 0.03$, read $Q = 2$ cfs

   $Q_x = 4.3 - 2 = 2.3$ cfs
6. The length of roadway, L, above the first inlet should be sufficient to generate the gutter flow of 4.3 cfs. Take $Q = C \cdot I \cdot A$, say $C = 0.8$, $I = 10.7$ in/hr, then $A = 0.5$ acres or 21,780 ft^2. $L = \frac{21{,}780}{26} = 838'$. Use 830′.
7. The flow bypassing the grate appears at the next grate and only the flow intercepted (2.3 cfs) must be supplied by the area between the grates.

   $$A = \frac{2.30}{(0.8)(10.7)} = 0.27 \text{ acres or } 11{,}800 \text{ ft}^2$$

   $L = 11{,}800/26 = 454'$ use 450′

8. The mean velocity in the 2-foot section over the grate is

$$V = \frac{2.3}{\frac{1}{2}(0.24+0.18)(2)} = 10.95 \text{ fps}$$

9. Minimum length of clear opening (for depth of longitudinal bar of 3")

$$L_b = \frac{V}{2}\sqrt{(d+d_b)} = \frac{10.95}{2}\sqrt{(0.24+0.25)} = 3.83'$$

Problem 2

Given: Q = 2 cfs ; pavement cross slope = 0.03 ; longitudinal slope = 3 % ; 2-foot gutter and pavement ; concrete broom finish with 10-foot curb-opening inlet depressed 2 inches.

Find: Discharge intercepted by inlet

Solution:

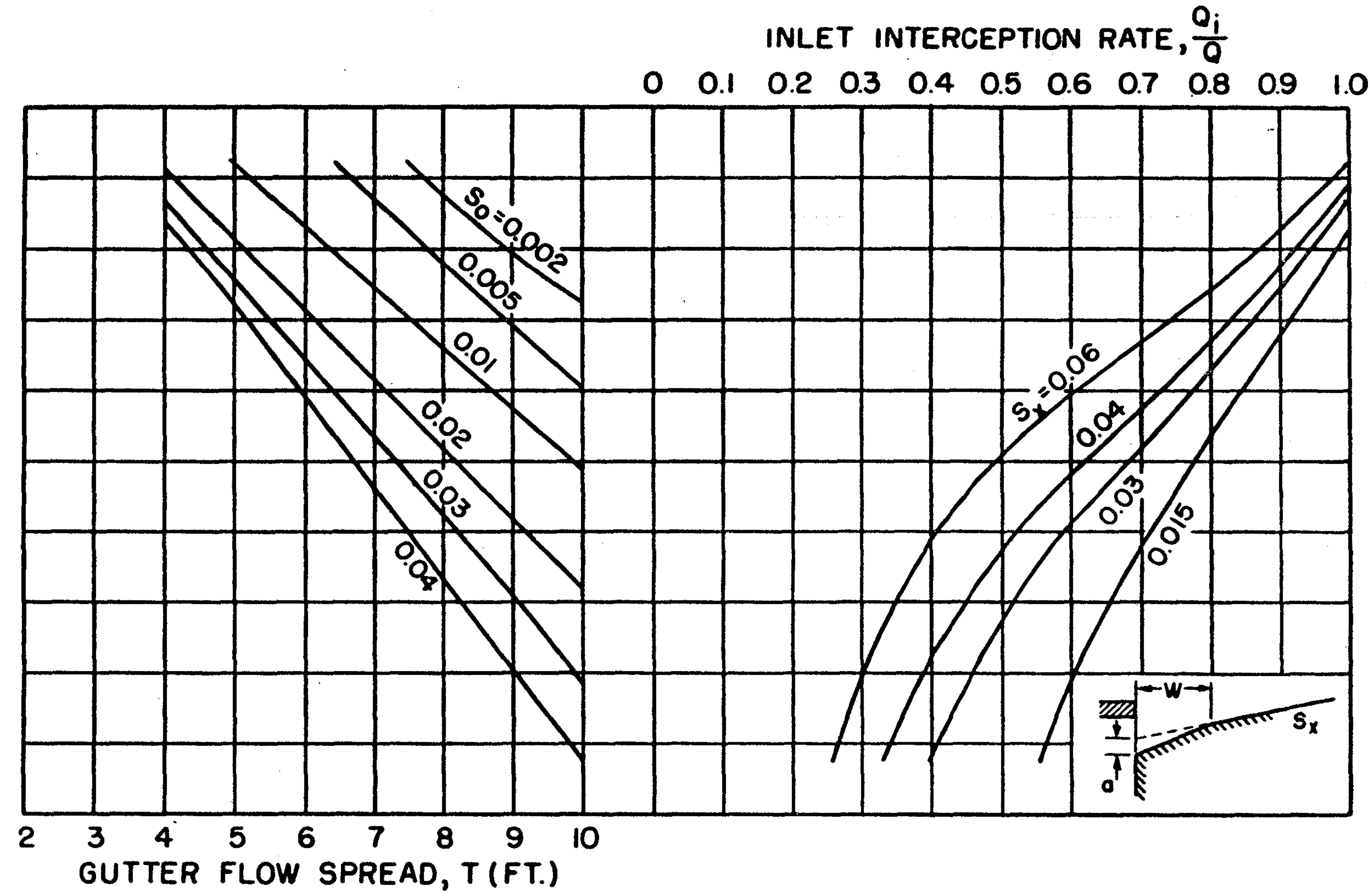
1. From page 8, $n = 0.016$
2. $Z = \frac{1}{3\%} = 33.33$, $\frac{Z}{n} = 2.083$
3. From page 7, with $\frac{Z}{n} = 2.083$, $S = 0.02$, $Q = 2$ cfs, read depth at curb $= 0.18 \pm (y)$
4. Spread on the pavement $= Zd = 33.33\,(0.18) = 6'$
5. From chart attached, with spread = 6', longitudinal slope = 0.03, cross slope = 0.03, read $\frac{Q_i}{Q} = 0.82$
6. Discharge intercepted $= 2\,(0.82) = 1.6$ cfs

pg .75

$\frac{Qa}{La} = .12$; $\frac{2}{.12} = LA = 16.67'$ length $\frac{2/12}{.18} = .9$

$\frac{L}{La} = .6$ $\frac{a}{y} =$

$\frac{Q}{Q_A} = .75$



General condition

W = 2' a ≧ 2"
n = 0.016
Length of opening, L_i = 10'
Minimum height of curb opening, $h_m = T\ S_x$

Source: Searcy, J.K., "Drainage of Highway Pavements", HEC-12, Federal Highway Administration, U.S. D.O.T., March, 1969.

Problem 3

Given: Conditions as in problem 1

Find: Spacing of 10' curb-opening inlet where $W = 2'$, $a = 2''$

Solution:

1. Spread on pavement $T = 8'$
2. Discharge in gutter $Q = 4.3^{cfs}$
3. Length of roadway above the first inlet $= 830'$
4. Same as problem 2, $Q_i = 2.7$ cfs intercepted by the inlet.
5. $A = \dfrac{2.7}{(0.8)(10.7)} = 0.315$ acres or $13{,}720\ ft^2$

   $L = \dfrac{13{,}720}{26} = 528'$ Use $520'$

Problem 4

PROJECT: 142 B

DESIGNER: ______

DATE: ______

HYDROLOGIC AND CHANNEL INFORMATION

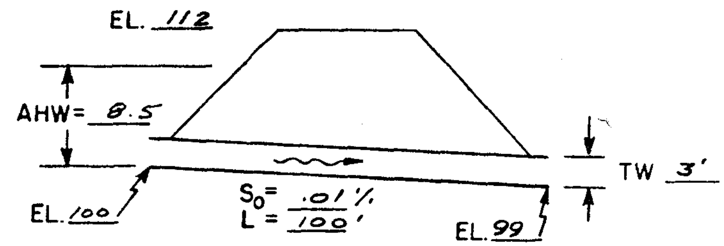
Q_1 = 160 cfs = Q_{50} TW_1 = 3.0'

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 321+14



MEAN STREAM VELOCITY = 8'/sec

MAX. STREAM VELOCITY = 10'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				
CMP Circular Headwall	160	48"	2.6	10.2	0.5	8.7	3.6	3.8	3	3.8 ~~3.8~~	1	11.5 ~~4.3~~				

PROJECT: 142 B

DESIGNER: ____________

DATE: ____________

HYDROLOGIC AND CHANNEL INFORMATION

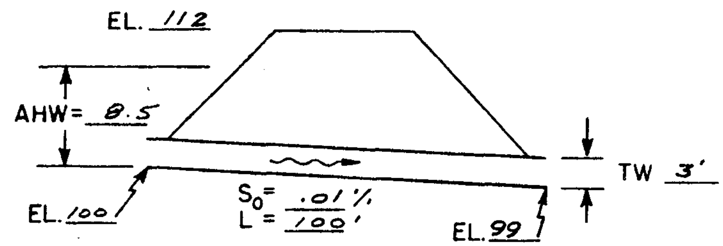
Q_1 = 160 cfs : Q_{50} TW_1 = 3.0'

Q_2 = ________ TW_2 = ________

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100}).

SKETCH

STATION : 321+14



MEAN STREAM VELOCITY = 8'/sec

MAX. STREAM VELOCITY = 10'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_0 - LS_0$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_0	LS_0	HW				
CMP (cir.) Headwall	160	Assume 54"	1.56	7.0												Hw less than 8.5'-Try 48"
"	160	48"	2.25	9.0	.5	8.3	3.7	3.8	3	3.8	1.0	11.1	11.1	13.2'/sec		Hw High Try 54"
"	160	54"	1.56	7.0	.5	4.7	3.6	4.1	3	4.1	1.0	7.8	7.8	11.1'/sec		Velocity at d_c size O.K.
Concrete (cir) Sq. Edge-Hdwl	160	48"	2.35	9.4	.5	4.7	3.7	3.8	3	3.8	1.0	7.5	9.4	14'/sec.		Hw high Try 54"
"	160	54"	1.6	7.2	.5	2.9	3.6	4.1	3	4.1	1.0	6.0	7.2	14.7'/sec		Hw ok. Vel > CMP Try 48 G. Edge
Concrete (cir) Groove end-HDWL	160	48	1.95	7.8	.2	4.0	3.7	3.8	3	3.8	1.0	6.8	7.8	14.0'/sec		Hw ok Vel. high

SUMMARY & RECOMMENDATIONS:

THE SELECTION OF A 54" CMP WITH HEADWALL WILL KEEP THE HEADWATER BELOW THE AHW WITH A MINIMUM OUTLET VELOCITY. A 48" CONCRETE PIPE WITH GROOVE EDGED ENTRANCE GIVES EQUAL HW AND SLIGHTLY HIGHER OUTLET VELOCITY. PROTECTION OF OUTLET CHANNEL MIGHT BE NECESSARY IN SOME LOCATIONS.

John F. Ahrens
PRC Engineering
4131 N. 24th Street, Ste. 110
Phoenix, AZ 85016

Nancy K. Ash
John Carollo Engineers
1314 3rd Street
Phoenix, AZ. 85004

Harold Baldwin
Collar-Williams-White Eng. Inc.
2922 N. 70th Street
Scottsdale, AZ. 85251

Bruce G. Beenken
Fincial and Dombrowski, Inc.
732 N. Stone Avenue
Tucson, AZ. 85705

Kebba Buckley
Flood Control District of
Maricopa County
3335 W. Durango Street
Phoenix, AZ 85009

Stephen Casillas
Dooley-Jones and Associates
P.O. Box 1830
Tucson, AZ. 85702

Michael Chase
Mettee/McGill/Murphy
1450 E. Indian School Road
Phoenix, AZ 85014

Darde de Roulhac
Office of City Engineer
City of Tucson
P.O. Box 27210
Tucson, AZ. 85726

Frederick J. Felix
City of Sierra Vista
2400 East Tacoma Street
Sierra Vista, AZ. 85635

Ronald Ferguson
Dooley-Jones and Associates
P.O. Box 1830
Tucson, AZ. 85702

Chuck Feuquay
Flood Cont. Dist. of Maricopa
County
3335 W. Durango Street
Phoenix, AZ 85009

Terry Hendricks
Pima Cnty Flood Plain
Management Section
1313 So. Mission Road
Tucson, AZ. 85713

Lonnie Frost
Pima Cnty Flood Plain Management Section
1313 So. Mission Road
Tucson, AZ. 85713

Mark Forest
Pima Cnty Flood Plain
Management Section
1313 So. Mission Road
Tucson, AZ 85713

John Hewitt
PRC Engineering
4131 N. 24th St., Ste. 110
Phoenix, AZ. 85016

Peter A. Johnson
City of Phoenix Engineering
125 E. Washington
Phoenix, AZ. 85004

Tom Loomis
Dashney & Associates, Inc.
1107 N. Beeline Hwy., Ste. D
Payson, AZ. 85541

Lisa E. McDaniel
Camp Dresser & McKee Inc.
2701 N. 16th St., Ste. 210
Phoenix, AZ 85006

Keith D. Nath
City of Mesa Engineering
P.O. Box 1466
Mesa, AZ. 85201-0904

Barry Nauseda
Maricopa Cnty. Hwy. Dept.
3325 West Durango Street
Phoenix, AZ. 85009

James Reyes
City of Tucson Engineering
P.O. Box 27210
Tucson, AZ. 85726

Jesus Romero
Flood Cont. Dist. of Maricopa
County
3335 W. Durango Street
Phoenix, AZ. 85009

Michael D. Salmon
Erikson & Salmon, Inc.
2814 N. 36th Street
Phoenix, AZ. 85008

Robert J. Smolinsky
Simons, Li & Associates
120 W. Broadway, Ste. 260
Tucson, AZ. 85702

William Schlesinger
Johnson-Brittain & Associate
630 East 9th Street
Tucson, AZ. 85705

Charles Schwer
City of Safford
717 Main St., City Hall
Safford, AZ. 85546

David M. Summers
Kobuk & Associates
P.O. Box 805
Camp Verde, AZ. 86322

Donald Tharp
Desert Earth Engineering
524 N. 6th Avenue
Tucson, AZ. 85705

Joe Tram
Flood Control District of
Maricopa County
3335 W. Durango Street
Phoenix, AZ. 85009

Kent Whitman
Community Sciences Corp.
P.O. Box 1328
Corrales, NM. 87048

Mike Yost
Community Sciences Corp.
P.O. Box 1328
Corrales, NM. 87048

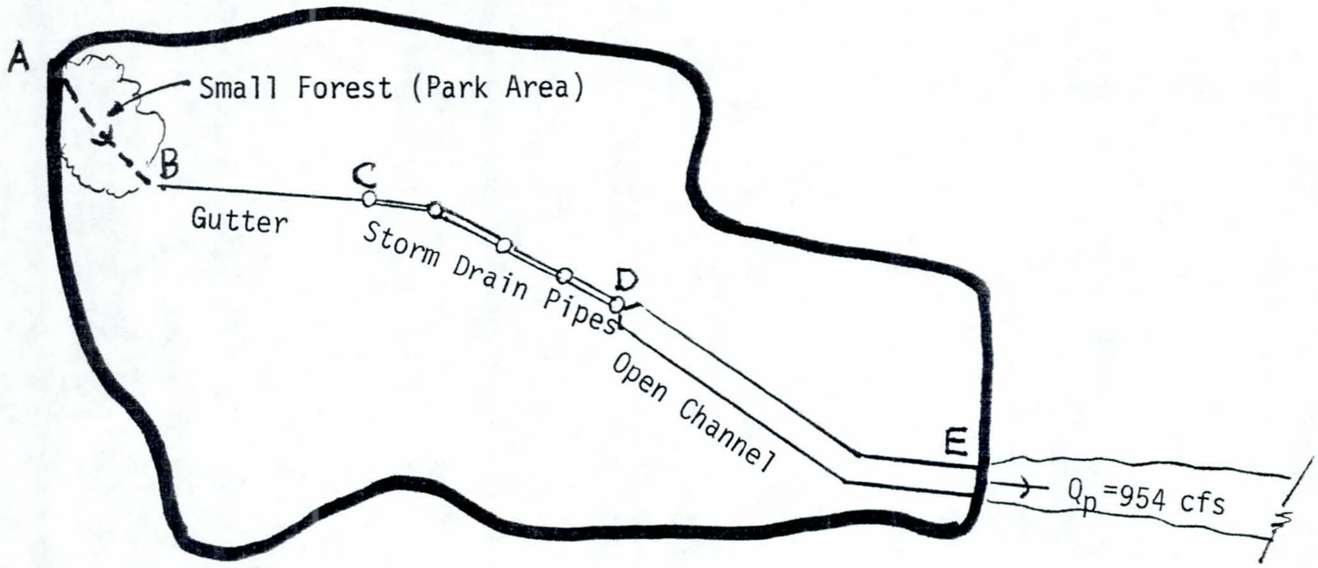
Francis M. Wunderle
3130 N. 7th Ave., Apt. 337
Phoenix, AZ. 85013

SPEAKER
Michael E. Zeller, P.E.
Simons, Li & Associates
P.O. Box 2712
Tucson, AZ. 85702

Herbert G. Poertner
Stormwater Consultants

WORKSHOP PROBLEM

The peak future discharge calculated for an urbanizing watershed at its lower end (E) is shown in the sketch. Also shown is the flow path determined for the runoff traversing the watershed from the most hydraulically remote point (A) to the lower end (E). The bank-full hydraulic capacity of the downstream natural drainageway below point E was calculated.



Reach	Description of Flow	Slope (%)	Length (Ft.)	V
A to B	Overland (forest)	7	500	.16 FT/sec
B to C	Overland (shallow gutter)	2	900	2.7 FT/sec
C to D	Storm drain pipes with manholes, inlets, Etc. (n=0.015; diameter= 3 Ft.)	1.5	2,000	2 FT/sec
D to E	Open Channel: gunite, trapezoidal (b=5'; d=3'; z=1.1; n=0.019); where b= bottom width, d=depth, z=1/side slope.	0.5	3,000	2.3 FT/sec

11.6

The design engineer decides to investigate the feasibility of constructing a detention facility immediately downstream of point E. The outflow structure must be designed to limit the peak discharge from the detention basin to 636 cfs.

An informal meeting is to be held in forty-five minutes (between the design engineer, the developer and the public agency representative) to discuss the feasibility of proceeding with the planned land development. You are the design engineer and will be attending the meeting.

The main item of discussion is to be the approximate storage capacity required. Prior to that will be a discussion of the calculations you made to determine the post-development peak discharge from the watershed.

Prepare a brief set of calculations (to distribute at the meeting) for the following:

(a) Time of Concentration (Tc) at point E.
(Suggestion: Refer to Figure 3.1 in the Soil Conservation Service publication "Urban Hydrology for Small Watersheds", Technical Release No. 55)

(b) Approximate Storage Volume needed (S) expressed in acre-feet.
(Suggestion: Assume inflow and outflow hydrographs to be triangular).

Also be prepared to explain the "procedure" you used to calculate the peak discharge (Q_p) at point E from the planned development.

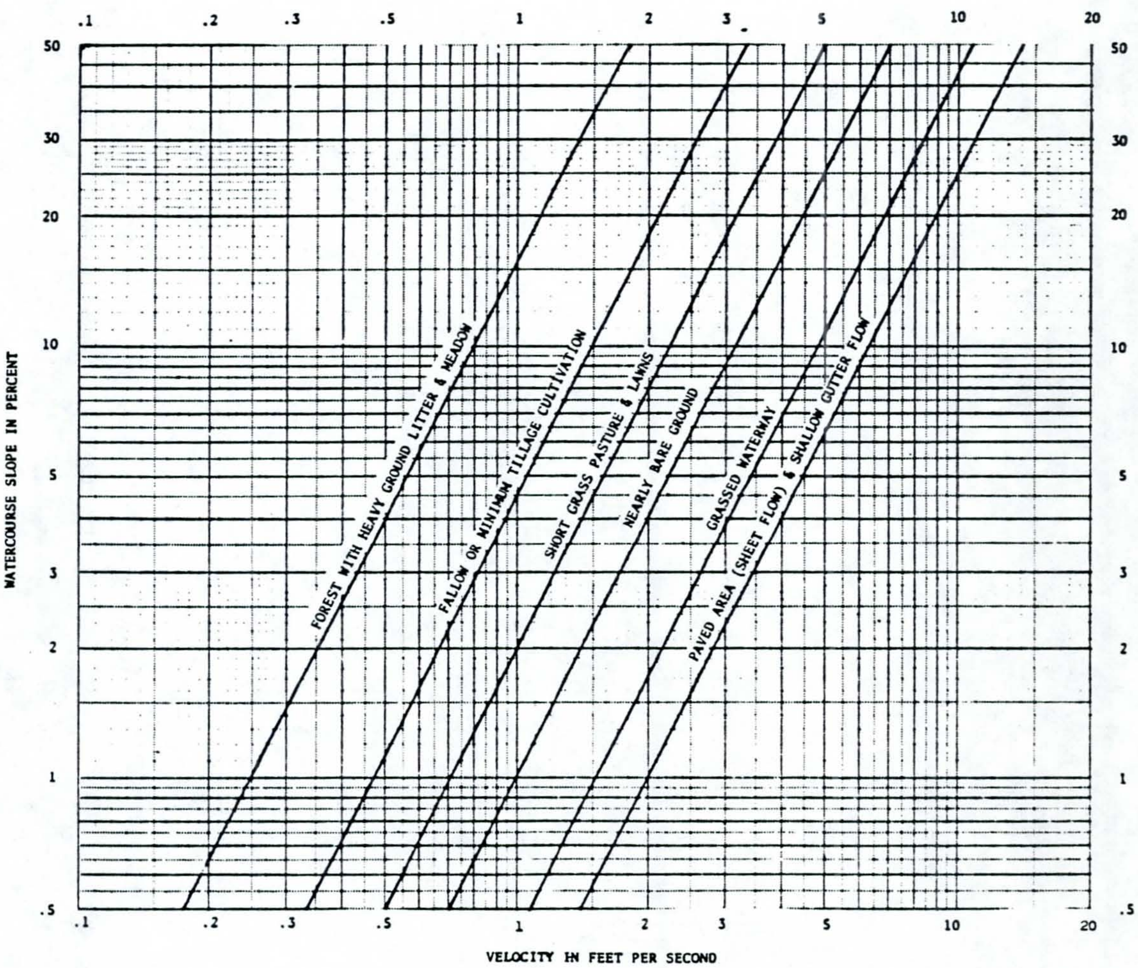


Figure 3-1.--Average velocities for estimating travel time for overland flow.

Soil Conservation Service, U. S. Department of Agriculture, Urban Hydrology for Small Watersheds, Technical Release No. 55, January 1975 (revised 1981).

$R = \frac{A}{P}$

$Q = AV$ $V = \frac{1.49}{n} R^{2/3} S^{1/2}$

$= \frac{1.49}{.015}$

$Q = AV$

$t_p = T_c$

	$T_1 V$	Time Sec	Sec
A B	.7	500/.7	714
BC	2.8	900/2.8	321
C-D	~~20~~ 10	2,000/10	200
D-E	~~2.3~~ 8.2	3000/8.3	364

Zeller Solution

$V_S = \frac{R}{12}\left(1 - \frac{Q_O}{Q_I}\right)A.$

$R = \overset{SCS}{\text{Runoff}} \times .70$

$Q_O =$

$Q_I =$

TC 1601 Sec

$S = .5(\cancel{2.67 \times 1600})($

4272 Sec

$= \frac{(2136)(954-636)}{43,560} =$

636
318

679,700 or 680,000 cf.

43,560

= 15.6 AC FT