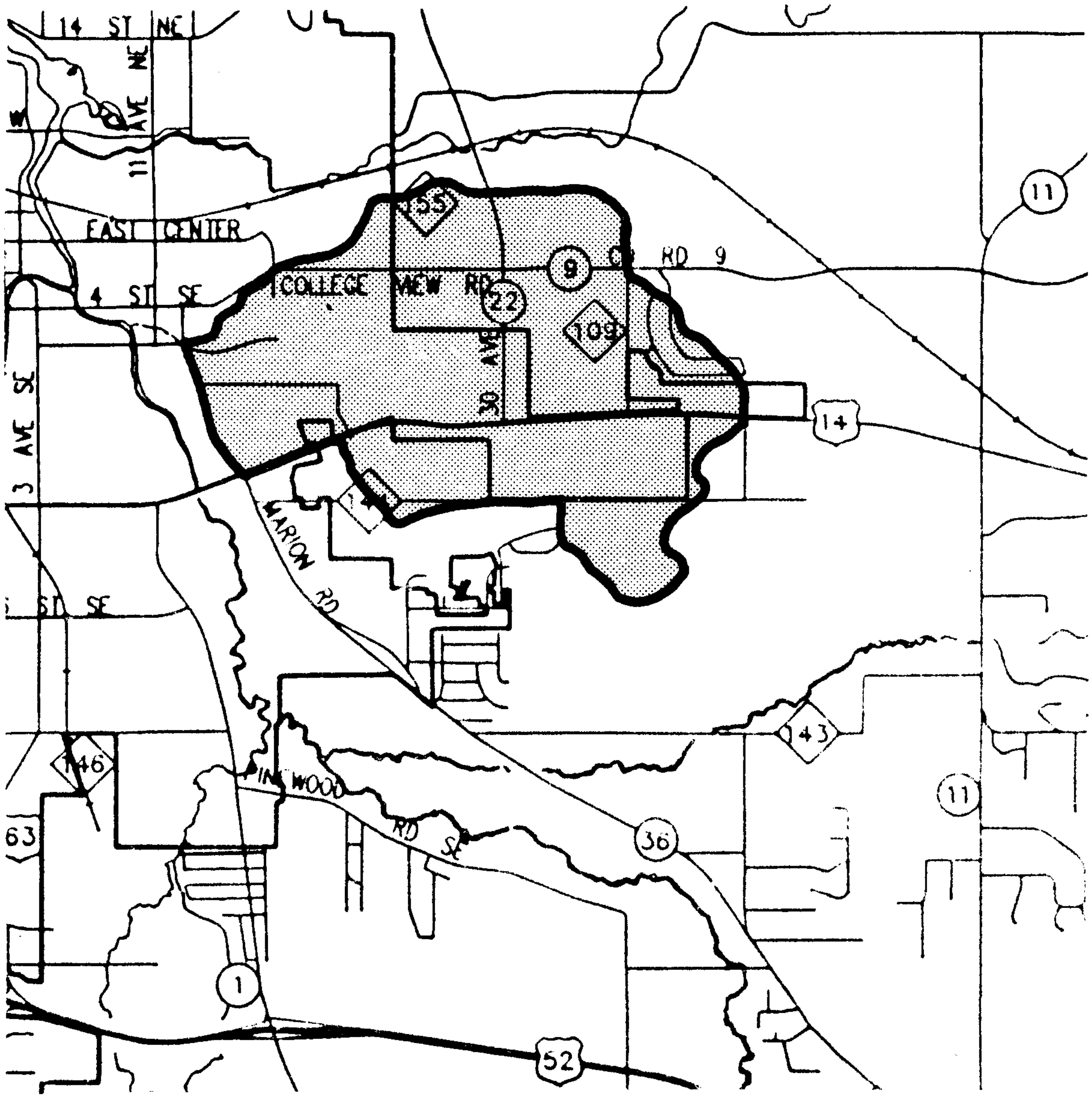

STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA
MAY, 1991

Bonestroo
Rosene
Anderlik &
Associates
Engineers & Architects
St. Paul, Minnesota

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I. INTRODUCTION

The purpose of this report is to provide the City of Rochester with a Comprehensive Storm Water Management Plan for the Mayo Run Watershed (Figure 1) that will serve as a guide for the development of the storm drainage system.

The Comprehensive Plan includes a layout of the storm sewer trunk system and ponding areas with major and minor drainage districts defined. Pond high water levels and the amount of storage required in each pond have been established. The size and capacity of the proposed and existing trunk storm sewers are shown. Cost estimates have been prepared to estimate the total cost to construct the trunk storm sewer as shown in Figure 8.

Storm water drainage facilities are an essential part of the development of any municipality. As an area develops from rural uses to urban uses, culverts and drainageways which were adequate for rural runoff become overloaded and flooding occurs; frequently resulting in property damage. The primary function of an urban storm drainage system is to minimize economic loss and inconvenience due to periodic flooding of streets, basements and other low lying areas. The desirable economic end point is reached when the cost of damage attributable to storm flooding plus the cost of storm sewer installation reaches a minimum.

On the other hand, economy is not the only consideration, since well designed storm drainage facilities provide flood control, minimize hazards, and inconvenience associated with flooding and protect and/or enhance water quality. During the preparation of this plan, the water quality aspects related to runoff have been addressed as part of the storm drainage analysis. Ponds are classified from a Water Quality perspective and initial water quality standards are being proposed. In the context of this plan, water that is retained below the normal water level is referred to as "wet volume". This volume of water helps to neutralize the action of phosphorus which is one of the main nutrients affecting water quality. Also, the aesthetic consideratoins of storm water management involving water quality and management of ponding areas have been considered in this plan.

Frequently, the downstream reaches of a drainage basin develop earlier than the remainder of the basin. When this occurs, drainage structures that are installed as part of the early development may be sized for only the present runoff without consideration for the increased rate and quantity of storm water runoff that will result as land in the upper reaches of the watershed is developed. A proper storm drainage plan takes the entire drainage basin with future saturation development into consideration. Therefore, costly mistakes such as replacement of undersized lines in developed areas, can be avoided.

If a planned program of storm drainage construction is established in the early development stages of a drainage basin and if the plan is carried through with all phases of development, the most economical system will be achieved and the great cost of duplication and waste arising from storm sewer construction after the area is developed is avoided. Storm sewers and ponding areas can then be incorporated into a developer's plan. By providing storm water storage in the system, ponding areas reduce the required capacity of the trunk sewers downstream. The decreased discharge rates result in smaller size sewers, reduce overland flow, and a more cost effective system. Ponding areas can also be planned and integrated into a park and a trailway system. The cost of acquiring these areas will be low at the present time compared to the cost involved after development has substantially increased land values.

An outline of the steps involved in the preparation of the plan is presented below:

1. Determine drainage district and subdistrict boundaries for use in sizing storm sewers, open channels and ponding areas.
2. Relate existing land usage and future land usage as presented in the Land Use Guide Plan to the probable amount of storm water runoff anticipated.
3. Establish routing of storm water conveyance facilities.
4. Establish location, size and flood elevations of storm water ponding or storage areas.
5. Investigate alternates which might affect the feasibility or economy of segments of the system.
6. Estimate the cost of storm drainage facilities in order to provide a guide for development of a sound and equitable financing program.
7. Consider the potential for using ponding facilities to meet desirable water quality standards.

The main goals of the City concerning the Mayor Run Watershed storm drainage is to preserve and use natural water storage and retention systems in order to:

a) Reduce to the greatest practical extent the public capital expenditures necessary to control excessive volumes and rates of runoff.

b) Improve and preserve water quality.

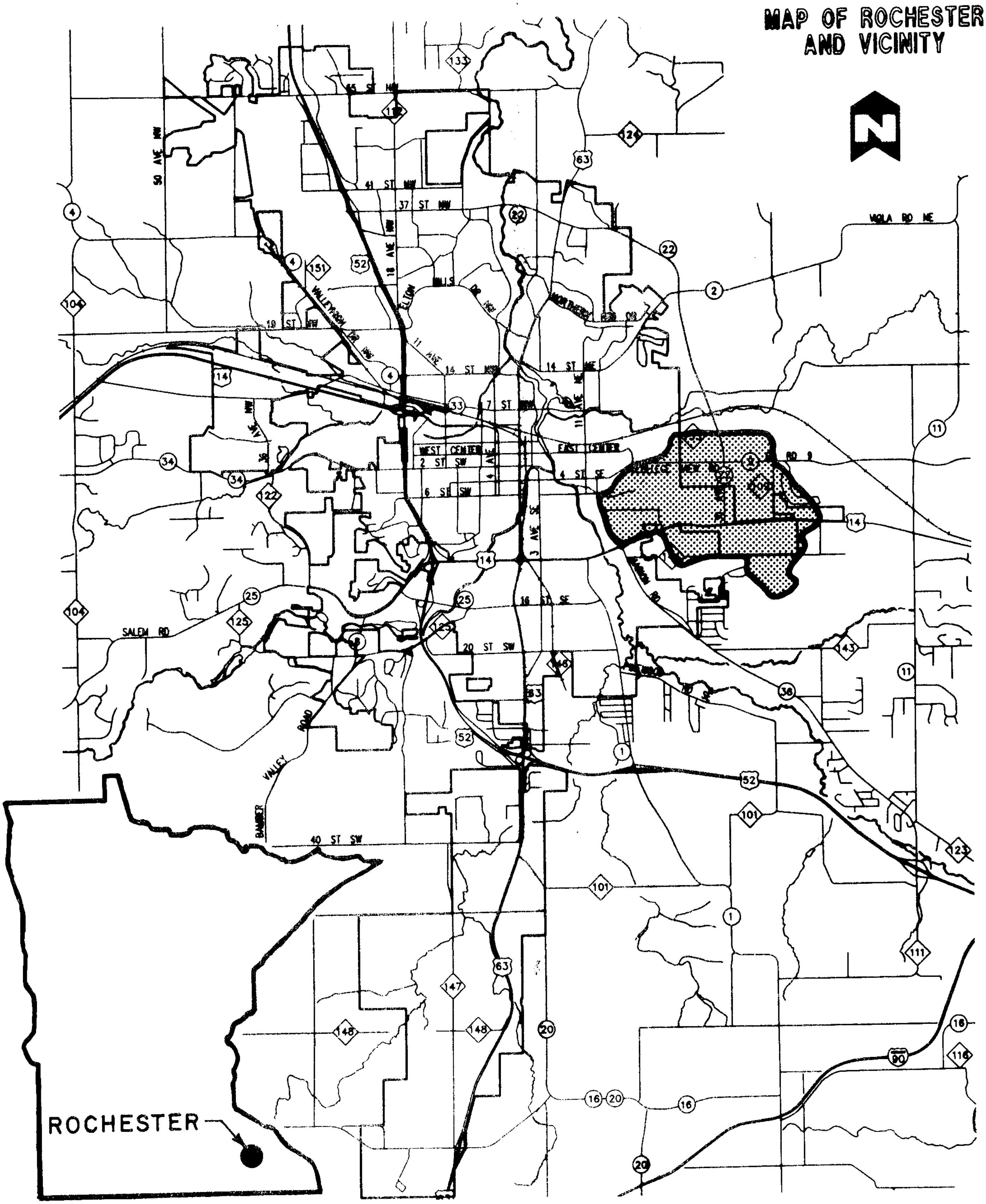
c) Prevent flooding and erosion from surface flows.

d) Protect and enhance wildlife habitat and park facilities, and

e) Secure the other benefits associated with proper management of surface water.

f) Maintain outflows from the watershed at a lesser rate than the capacities available in the culverts located between 13th Street and the Bear Creek.

An ultimate storm drainage system for the entire City is presented in this plan. The major storm water facilities that are required to control runoff from a completely urbanized watershed as shown in the Land Use Plan, Figure 6 are described. The ponding areas and pond outlets are designed for a 100 year storm (6.2 inches of rain in 24 hours). Overland drainage routes to ponding areas are required for complete protection from 100 year storms. Design data for the proposed storm drainage system is presented in Appendices A, B, and C. Estimated costs to complete the City's trunk storm sewer system are presented in Appendix D for each segment. Appendix E represents the well monitoring data. Pond water levels for 10, 50, 100 and 500 year storm are presented in Appendix F. Information pertaining to "wet" and "dry" ponds and their wet volume requirements, as well as water quality parameters, are presented in Appendix G.



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA

FIGURE NO. 1

LOCATION PLAN

9/19/90 COMM. 36302

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II. STORM DRAINAGE SYSTEM DESIGN

GENERAL

Since the middle 70's, storm water management authorities have emphasized the full utilization of storm water ponding and impoundment. In the ideal design solution, the storm water falling on a given drainage area should be absorbed or retained on site to the extent that, after development, the quantity and rate of water leaving the site would not be significantly different than if the site had remained in an undeveloped rural use. The City of Rochester should implement this principal in its policy for this and future Storm Water Management Plans.

The design of a storm drainage system involves the following aspects:

- The determination of the amount of runoff anticipated
- The selection of a method of conveying that runoff
- The delineation of ponding areas and floodplains for storage, sediment trapping and nutrient assimilation.

In addition, decisions must be made as to the degree of flood protection that is economically justified for an area. Once these factors have been determined, the system can be designed based on an economical combination of storm water storage volume, storm conduit capacity, and water quality improvements.

The Mayo Run Watershed is located in the eastern portion of the City, east of Bear Creek and between U.S. Highway 14 and 2nd Street, S.E.

The topography of the approximately 1,600 acres of land within the watershed boundaries varies from nearly flat at the center to fairly steep slopes north of 2nd Street S.E. and South U.S. Highway 14. Due to topographic features and for practical purposes, the major watershed was divided into 3 subdistricts (East, Central and West) to facilitate its hydrogical analysis. The drainage is generally toward the central portion of the watershed and has the main outlet at Mayo Run into Bear Creek. The storm water drainage plan contemplates outlets for most of these ponds and directs storm water runoff to Bear Creek.

Land surface elevations inside the watershed vary from a low of 1002 feet near the northwest corner of the watershed to a high of 1200 feet at the southeast section of the watershed.

In March 1980, the United States Department of Agriculture Soil Conservation Service published a Soil Survey for Olmsted County. This report provides information on the surface layers of soil within the Mayo Watershed. These soils have been grouped into a variety of classifications based on similar characteristics. The soils are also classified into nine hydrological groups by the Soil Conservation Service. These groupings classify the soils according to the infiltration rate and the transmission rate. The infiltration rate is the rate at which the water enters the soil at the surface and the transmission rate is the rate at which water moves through the soil.

Three hydrologically different types of soils appear in the Mayo Run Watershed:

a) Northeast Section: Dickinson-Plainfield association. Somewhat poorly drained soil with moderate to rapid permeability. Runoff is slow or medium.

b) Northcentral and East Sections: Racine-Floyd-Maxfield association. Moderately drained soil with moderate permeability. Medium runoff.

c) South and southwest sections: Rockton-Channehon-Adkinson association. Well drained soil with rapid permeability. Medium to high runoff.

Detailed information on the various soil types and location is contained in the Soil Survey Report available from the Olmsted County office of the Soil and Water Conservation District.

RUNOFF

Storm water runoff is defined as that portion of precipitation which flows over the ground surface during, and for a short time after, a storm. The quantity of runoff is dependent on the intensity of the storm, the amount of antecedent rainfall, the length of storm, the type of surface the rain falls on and the slope of the surface.

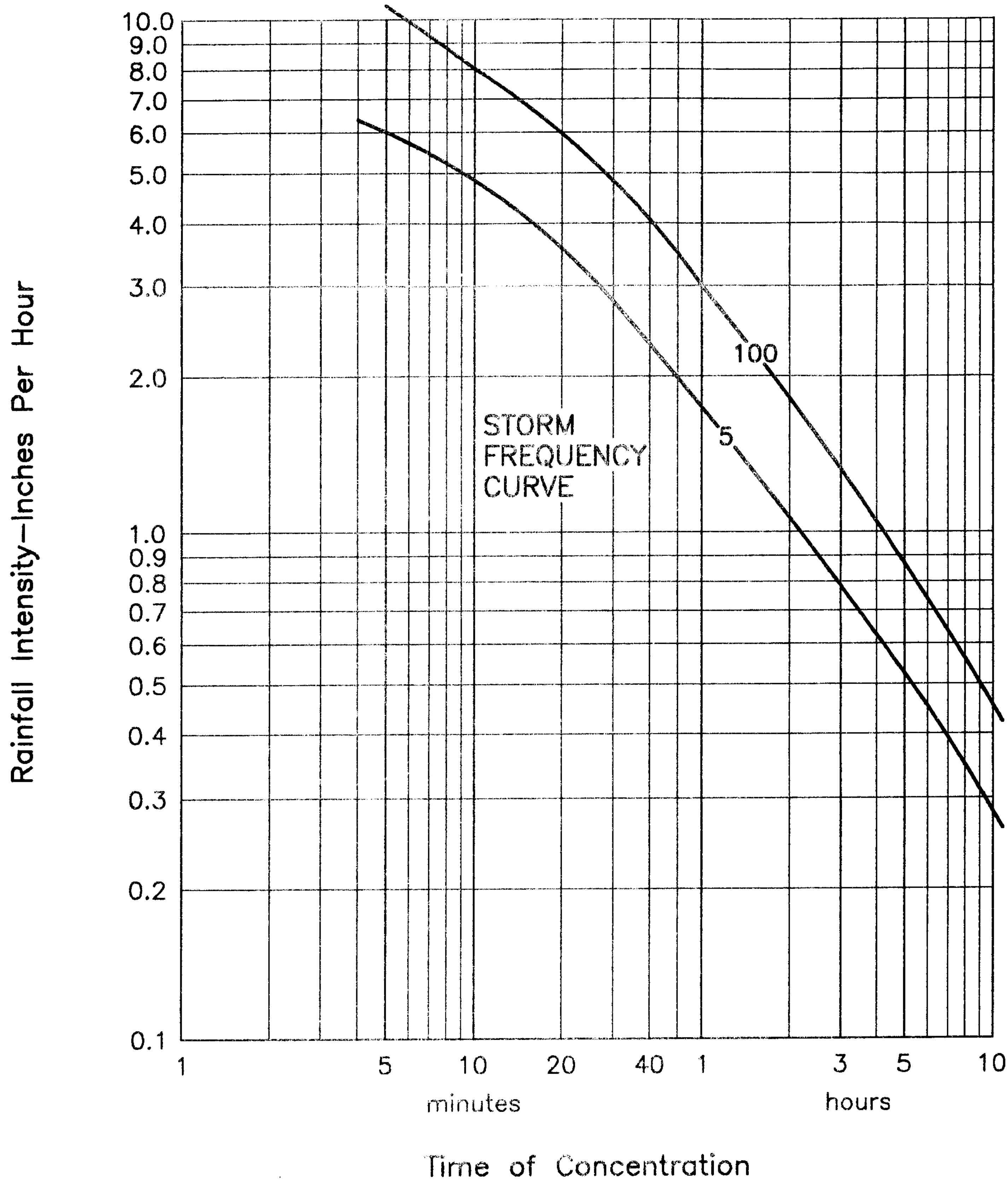
The intensity of a storm is described by a return storm interval which designates the average period of years during which a storm of a certain magnitude is expected to occur one time. Thus, the degree of protection is determined by selecting a return storm interval to be used as a basis for design. Based on historical data as presented in U.S. Weather Bureau Technical Paper No. 40, the rainfall events for storm sewer design are estimated to have a five or ten year frequency, and those for overland drainage and ponding are estimated to have a one hundred year frequency. A five year frequency storm has a 20% chance of occurring or being exceeded in any given year whereas a one hundred year frequency storm has a 1% chance of occurring or being exceeded in any given year.

Experience in areas similar to Rochester has shown that the runoff from storms similar to one having 1.8 inches of rainfall in one hour (5 year frequency storm), is adequate for the design of lateral storm sewers. Ponding areas are evaluated based on design storms similar to one having 6.2 inches of rainfall in 24 hours (also a 100 year storm).

Figures 3 & 4 reproduce charts for 24 hour rainfalls of 5 and 100 year frequencies of occurrence as presented in the Soil Conservation Service (SCS) Hydrology Guide for Minnesota.

The excess runoff caused by storms greater than that used for design will be accommodated by providing outflow through overland drainage routes. This short term overland drainage will minimize much of the damage to property which would occur if those facilities were not provided. Provisions should be made to provide or preserve overland drainage routes for emergency overflows and for snowmelt conditions.

A minimum concentration time of 25 minutes was selected for design of the trunk storm sewer system ponds. Shorter times may be utilized in lateral system design. As the storm water runoff enters the system, the flow time in the storm sewer is then added to the concentration time resulting in a longer concentration time and a lower average rainfall intensity as the flow moves downstream from the initial design point. The average rainfall intensity for the Design Storm Frequency of Occurrence can be computed or taken from a graph such as Figure 2.



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA

FIGURE NO. 2

RAINFALL CHART

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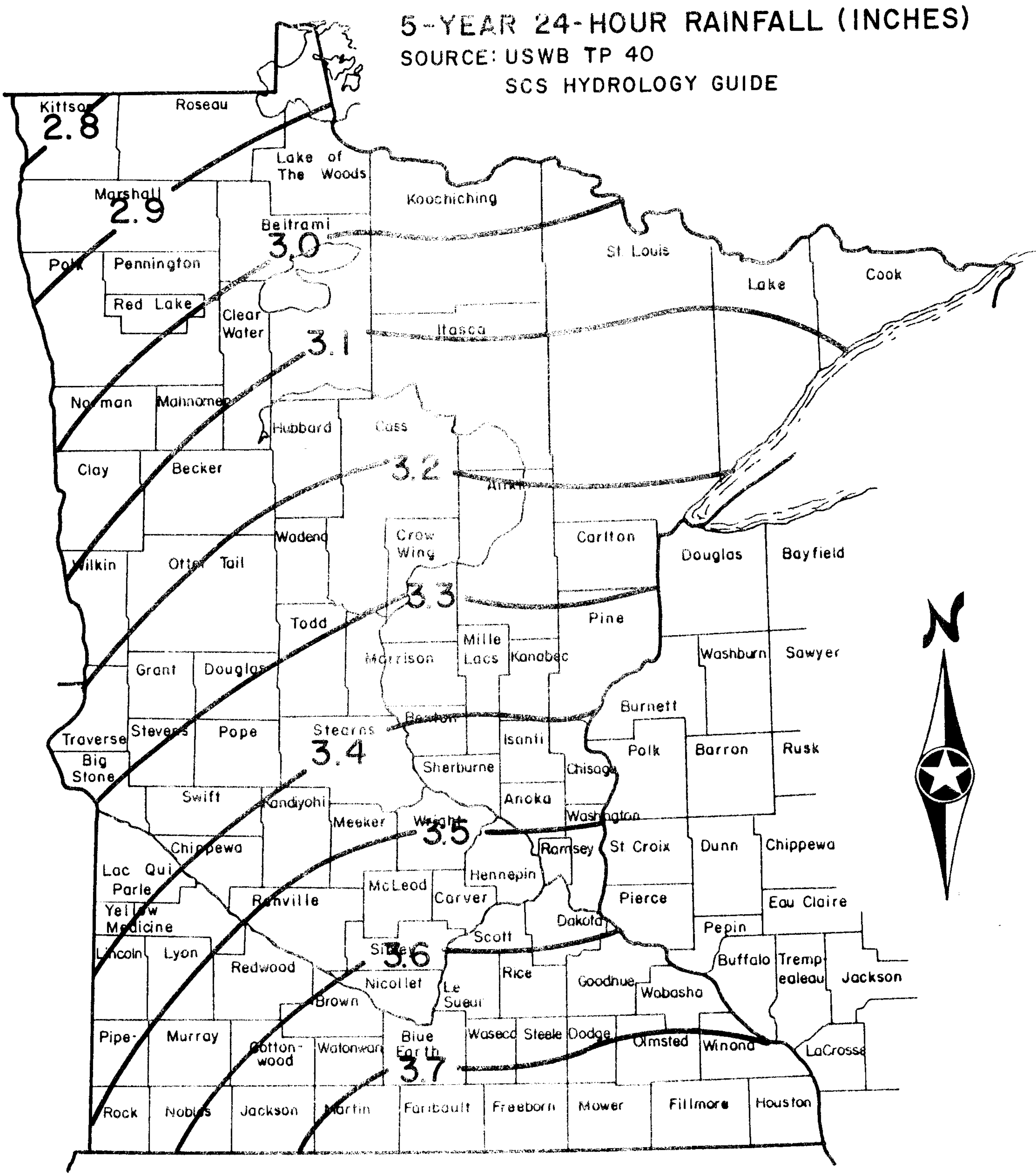

STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA FIGURE NO. 3

RAINFALL INTENSITY CURVES 5 YEAR FREQUENCY

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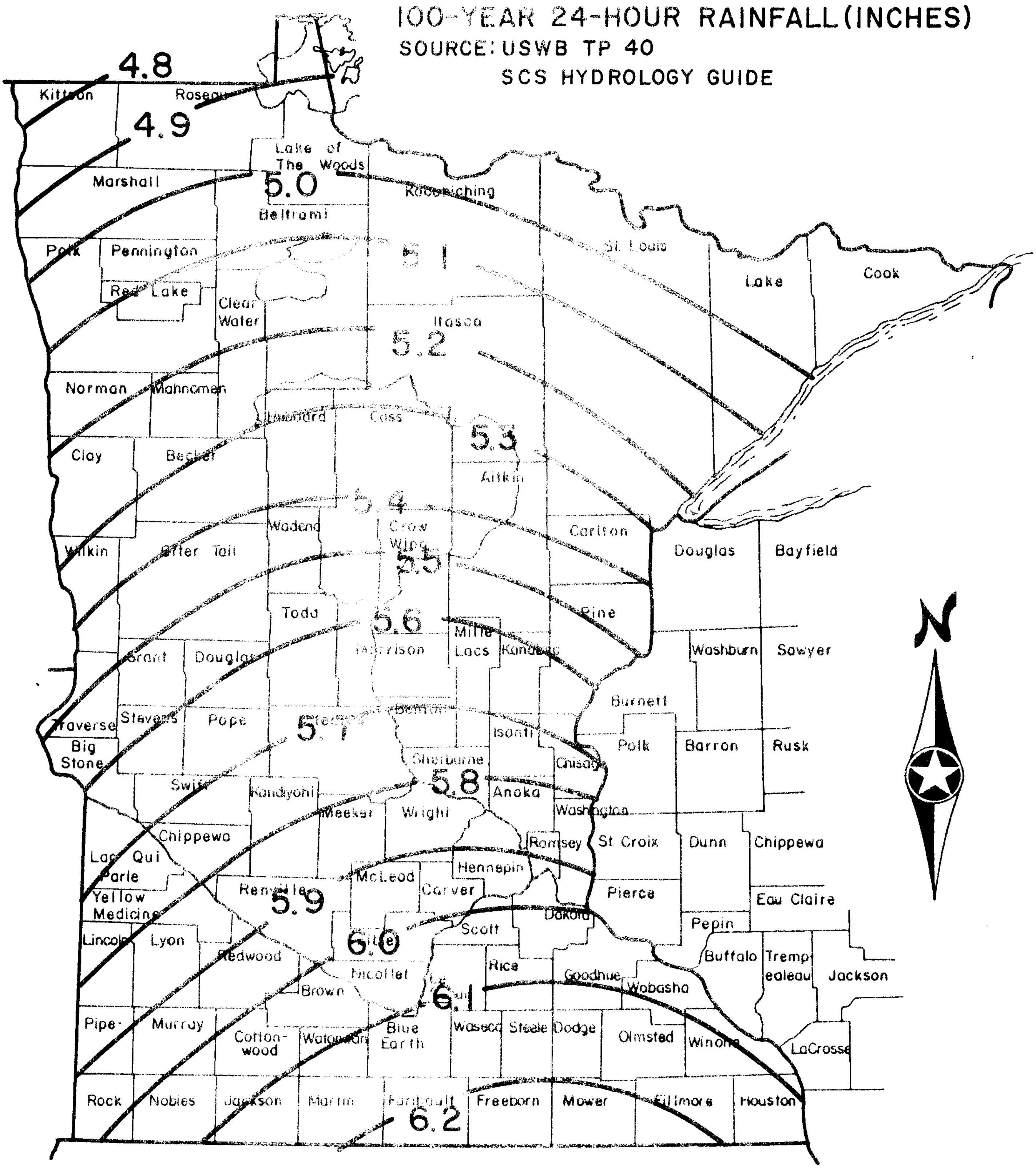
St. Paul, Minnesota



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA

FIGURE NO. 4

RAINFALL INTENSITY CURVES 100 YEAR FREQUENCY

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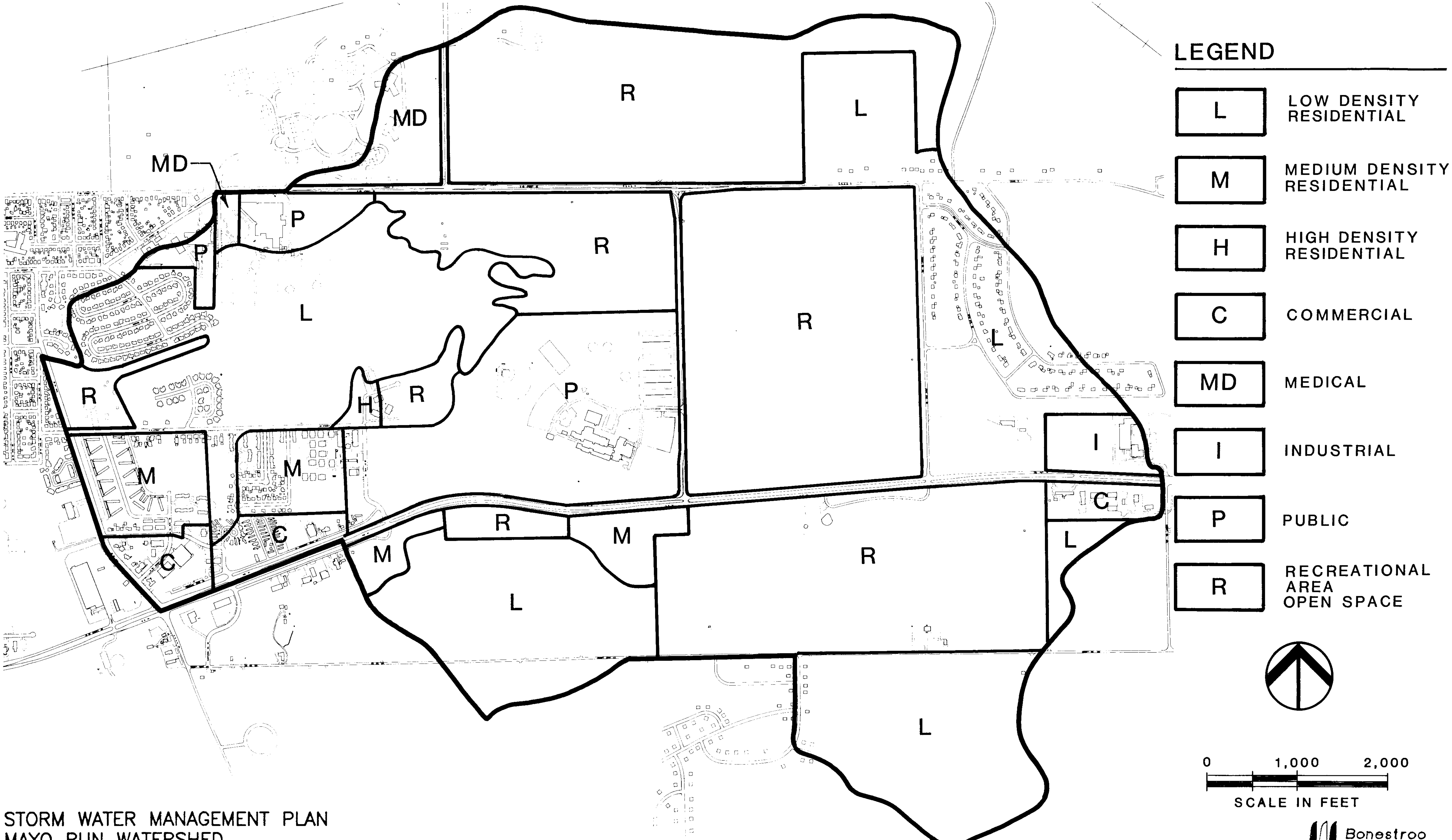
St. Paul, Minnesota

The percentage of rainfall falling on an area that must be collected by a pond or a storm sewer facility is dependent on the watershed variables such as soil perviousness, ground slope, vegetation, surface depressions, type of development and antecedent rainfall. These factors are taken into consideration when selecting a runoff curve number (CN) in the SCS TR-20 method for a certain area. The runoff curve number varies from 61 for parks to 98 for asphalt and concrete surfaces. CN values depend on the type of soil, cover type and hydrologic condition. Soils in the City of Rochester are typically type B which consist of shallow loess and sandy loams and have minimum infiltration rates from 4 to 8 mm/hr. This means that the peak rate of runoff experienced over a given watershed will vary from 20% to 95% of the rainfall rate. The present runoff coefficient throughout undeveloped areas is less than it will be when total urbanization is reached. The values of the coefficient will increase with the decrease in the amount of pervious surface, caused by street surfacing, lawn development, building construction, and grading.

The 1979 Land Use Guide Plan adopted for the Mayo Run Watershed is shown on Figure 5. The proposed Land Use Map used in estimating CN values for the sole purpose of indicating runoff coefficients expected when the land develops is presented in Figure 6. Average CN values for each land use type were used in the design of the storm drainage facilities of undeveloped area. For the modeling of existing facilities, CN values were determined for each type of development in each subdistrict. CN values are representative of degree of development and percentage of impervious surface in a drainage area. The values do not need to be changed for different storm events. Runoff coefficients



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA
EXISTING LAND USE PLAN
NOV., 1990 COMM. 36302

FIGURE NO. 5

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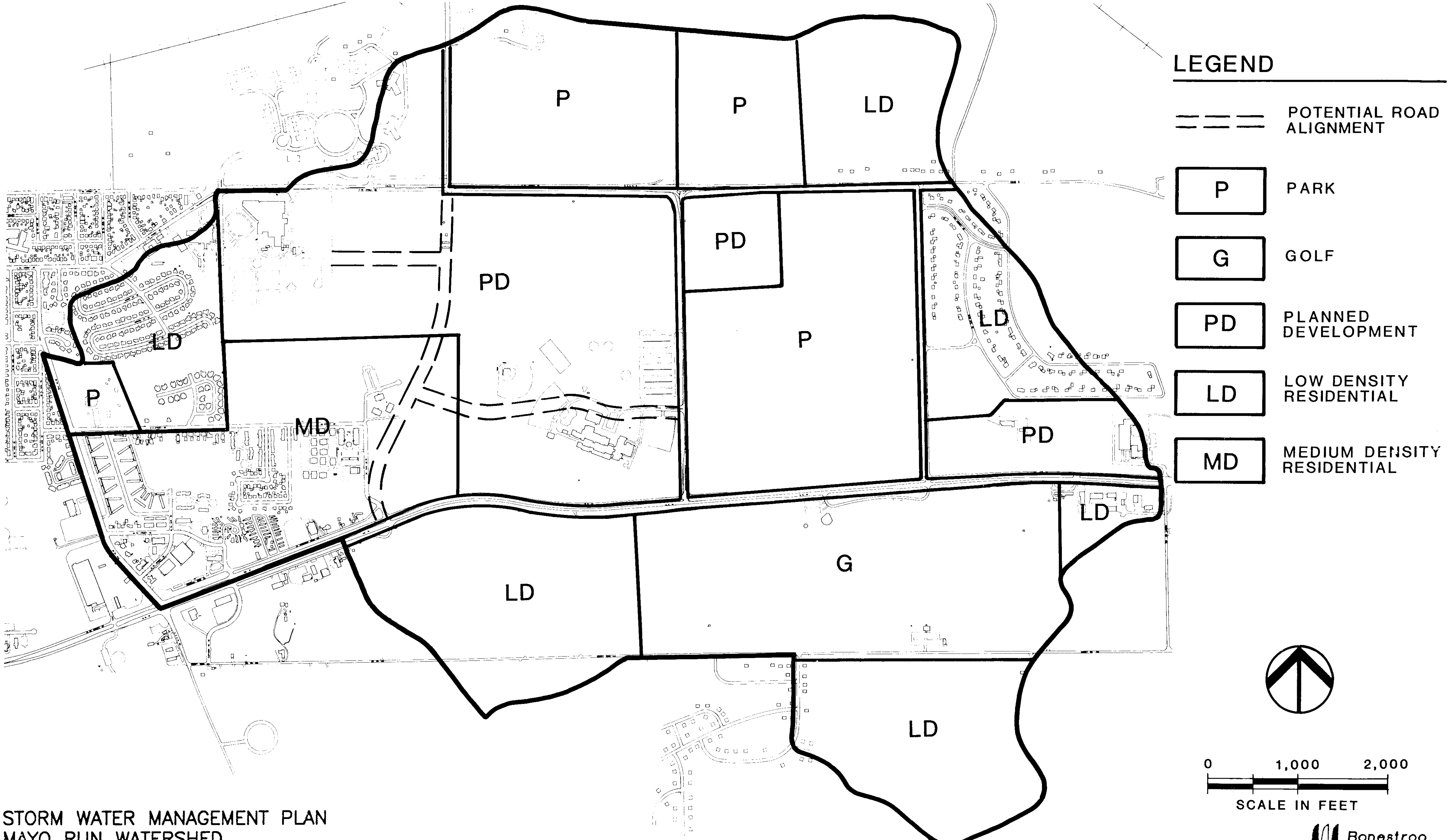
St. Paul, Minnesota



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA
PROJECTED LAND USE

NOV., 1990 COMM. 36302

FIGURE NO. 6

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traditionally used in the rational method approach and equivalent CN values for antecedent moisture conditions Type II (AMC II), are presented in Table 1.

A CN value of 58 for a Type B soil would be equivalent to a runoff coefficient (C) of 0.3 during a storm event of 6.2" of rain in 24 hours. A comparison of curve number (CN) values to runoff coefficient for a 100 year storm event (6.2" of rain in 24 hours) is presented in Table 2.

TABLE 1

RUNOFF COEFFICIENTS

LAND USE TYPE	C VALUE 5 Year	C VALUE 100-Year	CN Value (AMCII)
Parks and Public Land	0.2	0.3	58
Rural and estate Residential	0.3	0.4	66
Low Density & Single Family Residential	0.4	0.5	72
Medium Density Residential	0.5	0.6	78
High Density Residential	0.6	0.7	84
Commercial, Industrial	0.6	0.7	84
Planned Development	0.6	0.78	88
Ponds	1.0	1.0	99
Special	As required		

TABLE 2

EQUIVALENT CURVE NUMBERS TO RUNOFF COEFFICIENTS FOR A 100 YEAR STORM*

C	CN	RUNOFF
0.08	40	0.57
0.13	45	0.89
0.19	50	1.25
0.25	55	1.64
0.30	58	1.91
0.32	60	2.06
0.39	65	2.5
0.40	66	2.58
0.47	70	2.97
0.50	72	3.16
0.55	75	3.45
0.60	78	3.78
0.63	80	3.96
0.70	84	4.39
0.72	85	4.49
0.81	90	5.05
0.90	95	5.61
0.96	98	5.96

* 6.2 inches of rainfall in 24 hours and antecedent soil moisture conditions Type II

As can be seen in Table 1, the runoff coefficient for a 100 year storm is greater than that used for a 5 year storm. This is based on the assumption that wetter conditions during a 100 year storm occurrence result in greater runoff quantities. Data from Table 1 can be used to calculate overland drainage routes capacities. The determination of overland capacities will result in a more realistic and safer design of flow patterns likely to occur in the system due to a 100 year storm.

PONDING AREAS

Storm water ponding areas are an essential part of a storm drainage system. These areas provide locations where ponding caused by restricted flow can be allowed thereby minimizing flood damage by detaining, controlling, and delaying storm peak flows. The Storm Drainage Plan tries to incorporate the natural depressions found throughout the Mayo Run Watershed as ponding areas. The effective use of ponding areas enables the installation of outflow sewers with reduced capacities since the design storm duration is effectively increased over the total time required to fill and empty the ponding reservoirs.

Equally as important as the cost considerations is the use of ponding areas to 1) improve water quality, 2) stabilize the groundwater table, and 3) increase water amenities in developments for aesthetic, recreational and wildlife purposes. Storm water quality is improved by allowing nutrients and sediments carried by runoff to settle below the pond normal water level. Stabilization of groundwater is obtained by designing normal water levels and the outflow rate from the ponds. Amenity aspects are maximized by careful planning in the initial development of any area and by integrating the ponding system into the park development program wherever possible.

The outlet for the Mayo Run Watershed is the 4' by 10' box culvert under 13th Avenue S.E. The invert elevation of this culvert is 996.8 while the lowest street elevation is 1001.4. The difference in elevation allows the culvert to operate with 4' of ponded water and a free board of 0.6 feet before the road starts to flood. Under existing conditions there is a flow capacity of approximately 225 cfs which provides a level of protection for a 2 year storm without flooding the road (assuming the culvert operates without debris or obstructions). Ponding areas for the Mayo Run Watershed are proposed based on the amount of runoff expected to occur during a 100 year storm event (6.2" in 24 hours). Runoff is calculated considering the estimated future projected development in the watershed as shown in Figure 6 and its final outlet capacity. In the proposed Plan, ponding areas were designed to control runoff from a 100 year storm allowing a water elevation of 1000.5 at 13th Avenue S.E.

Ponding areas are shown on Figure 8 in the back of this report and data on the ponds is provided in Appendix C. The ponds are identified by a letter for the drainage district in which they are located (East, Central or West), the letter "P" to designate a ponding area, and a number to differentiate between ponds in the same district. Thus, the first pond in the East District would be numbered EP-1.

Pond CP-13, in the Central Drainage District, is a natural pond. In a natural pond, a long term balance exists between runoff from precipitation and water loss from evaporation and seepage. A normal water level in this pond has usually been established due to this balance. Over the years, organic material and silt have been carried into this pond by runoff. This material along with the decomposing of aquatic vegetation and algae on the bottom forms a layer of impervious materials sealing the bottom and limiting seepage. During wet periods, the water level rises and greater seepage occurs in the band of flooded land around the pond above the normal water level. Therefore, monitoring of water levels following storm events exceeding 2 inches in 24 hours is recommended for Pond CP-13 to determine empty times and possible adjustments of the pond outlet. Water level monitoring for Pond CP-13 from May 10, 1990 to June 22, 1990, was used to determine the pond normal water level. Appendix E shows the water table elevations for Wells 1 to 6 and the intensity and timing for the different storm events.

Most of the ponding areas in the system collect water from large drainage areas. To provide proper protection for adjacent property, the design storm interval for ponding areas is a 100 year storm. To provide an additional safety factor, the lowest exposed elevation of a structure in a development should be at least 2 feet above the calculated High Water Level of the pond. The lowest exposed elevations of structures that are adjacent to ponds should be certified by the builder during basement construction to ensure adequate elevation differential.

Runoff determinations for pond design vary from storm sewer calculations. The critical storm for storm sewer design is the short, high intensity storm, whereas the critical storm for pond design is of longer duration, since water is being stored for longer periods of time and released at a slower rate. Since runoff is directly related to rainfall intensity, a graph has been presented in Figure 2 that relates rainfall intensity to storm duration for a five year and one hundred year storm characteristic of the Rochester area. The rapid decrease in average rainfall intensity for the short duration of four or five hours emphasizes the advantage of ponding water from short duration, high intensity storms. Computations of pond inflow is based on the TR-20 method which uses a runoff curve number for a drainage area, a time of concentration, rainfall intensity of 6.2" and rainfall Distribution Type II which is given in Table 3. A 0.4 hour time increment is used for the Type II rainfall distribution.

TABLE 3

TR-20 METHOD
TYPE II RAINFALL DISTRIBUTION
CUMULATIVE % OF RAINFALL

TIME HOURS	0.00	0.25	0.50	0.75
0.00	.0000	.0020	.0050	.0080
1.00	.0110	.0140	.0170	.0200
2.00	.0230	.0260	.0290	.0320
3.00	.0350	.0380	.0410	.0440
4.00	.0480	.0520	.0560	.0600
5.00	.0640	.0680	.0720	.0760
6.00	.0800	.0850	.0900	.0950
7.00	.1000	.1050	.1100	.1150
8.00	.1200	.1260	.1330	.1400
9.00	.1470	.1550	.1630	.1720
10.00	.1810	.1910	.2030	.2180
11.00	.2360	.2570	.2830	.3870
12.00	.6630	.7070	.7350	.7580
13.00	.7760	.7910	.8040	.8150
14.00	.8250	.8340	.8420	.8490
15.00	.8560	.8630	.8690	.8750
16.00	.8810	.8870	.8930	.8980
17.00	.9030	.9080	.9130	.9180
18.00	.9220	.9260	.9300	.9340
19.00	.9380	.9420	.9460	.9500
20.00	.9530	.9560	.9590	.9620
21.00	.9650	.9680	.9710	.9740
22.00	.9770	.9800	.9830	.9860
23.00	.9870	.9920	.9950	.9989
24.00	1.000	1.000	1.000	1.000

The use of the TR-20 computer model in the analysis of the Mayo Run ponding system has enabled the efficient review of several routing patterns through several ponds. The pond storage and outflow rates were determined by computer for all of the ponds identified in this report.

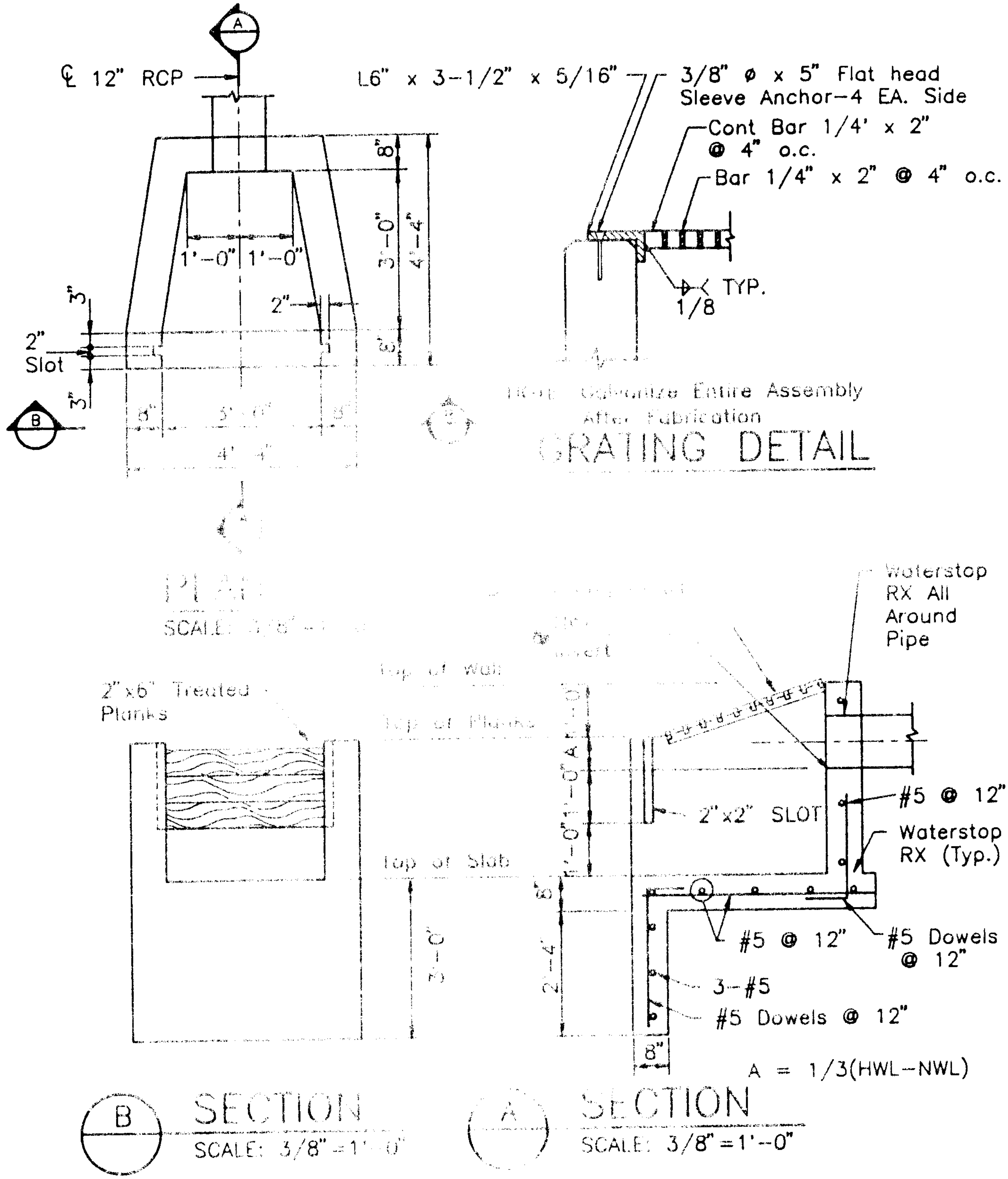
Theoretically, as a pond elevation changes while it fills and empties, the rate of discharge will vary but not be significant in cases where pond elevations change only a few feet. However, pond outlets should be designed carefully since an under-designed outlet will create local flooding by retaining more than the designed volume of storm water and an over-designed outlet will reduce ponding efficiency by decreasing detention time.

Special attention has been given in this plan to ponds with outlet capacities of 2 cfs or less since they are often provided with a 12" pipe outlet without flow restrictions which result in outflow rates greater than desired. Ponds with oversized outlets reduce the available flow capacity in downstream pipes and tend to empty sooner than anticipated. This can be resolved with the construction of outlet control structures and are recommended for ponds listed in Appendix C.

Pond outlet devices that will control outflows larger than 2 cfs should be designed with capacities that will allow a certain amount of outlet variation. As a matter of policy, a pond outlet structure should be designed to provide a low outflow rate to a predetermined pond elevation (i.e., V-notch weir, orifices, inverted pipes, etc.). This will allow the downstream excess pipe capacity normally used by the pond outflow to be used by flows from short duration, high intensity storms. Upon reaching the overflow elevation, ponded water will then be allowed to pass through the outlet at the design rate. This will provide an additional safety factor in the downstream reaches of the drainage system since the design overflow will occur during longer duration storms which do not produce the critical flows downstream.

Where feasible and in areas prone to spills that can contaminate the environment, pond outlet structures should be designed to "skim" the flow. "Skimming" the flow can allow a significant amount of time for cleaning or removal of undesirable floating debris and substances. A typical skimming structure for outflows under 3 cfs is shown in Figure 7.

An ideal storm water pond is one which has a large surface area with gradually sloping sides combined to provide substantial storage volume for storm water and water quality purposes. For water quality purposes, the ideal storm water pond is one which has a wet storage volume greater than or equal to the volume of runoff from a 2.5 inch rainstorm under full projected watershed development conditions. This volume will satisfy the recommendations of the National Urban Runoff Program (NURP), and the "Walker" phosphorus removal model. The side slopes of any pond should not be steeper than 4' horizontal to 1' vertical (4:1) and, where possible, should not be over 10' horizontal to 1' vertical (10:1).



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA

FIGURE NO. 7

FREE FLOW

9/19/90

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St. Paul, Minnesota

STORM SEWERS AND OPEN CHANNELS

Storm sewers are the actual conduits used to transport storm water runoff. The capacity of the storm sewer conduit is dependent on the pipe slope, pipe diameter, and the roughness of the inner surface of the pipe. The capacity is measured in volume per unit of time, or cubic feet per second (cfs) as determined by the Rational Formula. Computations for storm sewer conduit capacity are based on the following Manning's Formula:

$$Q = \frac{1.49}{n} (A/P)^{2/3} S^{1/2} A$$

Where:

Q = Storm sewer conduit capacity in cubic feet per second (cfs)

n = Roughness coefficient

A = Area of conduit

P = Circumference of conduit

S = Slope of conduit

A roughness coefficient (n) of 0.013 is used for concrete storm sewer pipe which takes into account losses due to bends and manholes in the system as well as the roughness of the inner pipe surface.

Open channels for the Mayo Run Watershed are shown in Figure 8. A trapezoidal cross-section with 3:1 maximum side slopes (3 feet horizontal to 1 foot vertical) should be designed wherever open channels are utilized. The same Manning's formula should be used to determine channel capacity with the roughness coefficient (n) increased to 0.030. For open channels, P in the equation becomes the wetted perimeter of the channel.

Only major storm sewer trunks and related facilities have been considered in this study. A complete working system consists of trunks, manholes, lateral lines, overland drainageways, catch basin leads, catch basins, pond inlets and outlets and all correlated items.

The ultimate storm drainage system alignment with channels and approximate pipe locations is shown on Figure 8 at the back of this report. Design flows for proposed facilities are given in Appendix B. The alignments shown on the plan are general in nature since future development will determine the exact location of channels or storm sewers. The lines shown as future alignment follow natural drainageways and the existing slope of the terrain wherever possible, therefore, variations from proposed alignments should be kept to a minimum. Pipe sizes and channel widths are also general since they are based on an assumed slope. Final design slopes will be dictated by grades established when an area develops.

The design capacities given in Appendix B are the controlling criteria for future design. It is extremely important that each area be re-evaluated at the time of final design to confirm the criteria used in this study and to make any changes that a proposed development may dictate. Special consideration must be given to areas that develop differently than shown in the Land Use Guide Plan, especially when a higher runoff coefficient will result from a development.

Although lateral systems are designed for five year storm events, their performance must be analyzed for storms exceeding the design storm. It should be anticipated that surcharging of the system will occur when the design storm is exceeded. During surcharging, the system works as a closed conduit and the pipe network becomes pressurized with different pressure heads throughout the system. Low areas that are commonly provided with catch basins become small detention ponds often performing like pressure relievers (water "gushing out" in some locations). For this reason, it is extremely important to ensure that these low areas have an overland drainage route and are not landlocked.

All storm sewer facilities, especially those conveying large quantities of water at high velocities should be designed with efficient hydraulic characteristics. Special attention should be given during final design to those lines which have extreme slopes and create high hydraulic heads. These lines should be designed to provide adequate energy dissipation which will reduce the risk of back pressure effects and reduce the risk of downstream erosion.

It is very important that consideration be given to each intake to ensure adequate inlet capacity. This is especially true where steep grades allow excessive carry over where conventional grates are used. The desired inlet structure is one which will provide the required inlet capacity and will not produce a hazard. In some cases, specially designed intakes may be necessary. These intakes will require an increased inlet area and should be oriented more normal to the direction of the water flow. All of these details should be considered during final design.

Open channels are shown on Figure 9 where flows and small grade differences prohibit the economical construction of an underground conduit and in areas where an open channel type drainage will enhance the aesthetic qualities of a development. Slopes indicated in Figure 8 for open channels should be maintained. Slopes of less than 1.0% are difficult to construct and maintain and can create problems with pocketing of water. Although due to the elevation of the water table, channels might carry flows throughout the summer. Side slopes should be a maximum of 3:1 (horizontal to vertical) with lesser slopes being very desirable. Where space permits, slopes should be cut back to the existing ground level. Proper riprap should be provided or other erosion protection measures at all points of juncture between two open channels and where storm sewer pipes discharge into a channel. Riprap or concrete liners should be provided in areas where high velocities cannot be avoided. Periodic cleaning of an open channel is required to insure that the design capacity is maintained. Therefore, all channels should be designed to allow easy access for equipment.

Both storm drainage facilities and sanitary sewer lines are designed to take advantage of natural draws and usually follow a ravine, creek or gully. As more area develops in the watershed, the total runoff in natural drainageways will increase. This is especially true for the areas south of the golf course. As these areas develop, the provision of a pipe outlet connecting ponds EP-1 and EP-2 to EP-8 might be necessary.

III. STORM WATER QUALITY

GENERAL

Maintaining the highest quality of water practical in the storm water system is an essential element to any Storm Water Management Plan. The only completely effective way to keep a high quality of water in ponds and lakes is to prevent undesirable sediments, nutrients and other materials from entering the storm drainage system. Presently, complete interception for water treatment at the point of discharge is neither practical nor economically feasible.

The four main reasons for degradation of water quality are 1) solids from erosion or street sanding, 2) calcium chloride or salt from street sanding, 3) composted decay around ponds, and 4) fertilizers and other chemicals from lawn care, impervious surfaces, or farming practices. The recognition of the problems and the implementation of reasonable control measures can minimize the degradation of the water quality in the Mayo Run Watershed ponds.

In areas such as Mayo Run Watershed where substantial development will take place, the storm water frequently will contain substantial quantities of solids. This is a typical situation when large amount of site grading is being done and inadequate erosion control management practices are being followed.

Erosion control measures during construction cannot be overemphasized. The City of Rochester enforces the practices recommended in the SWCS Erosion and Sediment Control Planning Handbook.

Since many solids enter defined ponding areas during construction, developers could be required to excavate temporary settling ponds in low areas of their development.

Even when precautions are taken during construction, a large amount of solids may enter the drainage system. For this reason, it is desirable to minimize the quantity of these solids in the storm water before they enter the streams and ponds. Provision for siltation basins or structures located at the point of discharge into a body of water are recommended.

In certain cases, settling chamber type catch basins or manholes can be provided in storm sewers prior to discharging into ponds. These can effectively provide removal of sand and gravel which may be flushed down the storm sewer from streets or highways, but are not effective in the removal of finer particles such as silts and clays. Use of this type of catch basin or manhole should be limited to those areas where they can be regularly maintained and can realistically be expected to intercept the sand from winter sanding operations and gravel from driveways or development construction. A solids removal structure must be regularly maintained if it is to remain effective. Maintenance would be greatly reduced and more likely completed with only a few solids removal systems. In all cases, the location, number, and type of solids removal systems to be utilized with ponding areas must be established at the time of final design of that portion of the storm sewer system.

Even with the best and most expensive solids removal system, contamination of the ponds and lakes will occur unless careful attention is given during development and continually thereafter in the use of the land. Developers must utilize the best practices to minimize erosion during the grading of the land and home construction. Property owners must use care in the development of their yards and sodding of bare areas. Debris is frequently raked from lawn areas before and after sodding and left in the street gutters, which if not cleaned up will be washed into the storm sewer, eventually reaching ponding areas. After development is complete, streets must be kept clean by conscientious effort from citizens to avoid littering and by frequent street sweeping to remove sand, dirt, and litter before it washes into the storm sewer system. Chemicals such as calcium chloride must be minimized in ice control programs on streets and highways. Residents must also make judicious use of fertilizers, especially those using phosphorus and other chemicals which wash into the ponds and cause degradation of the water quality.

For water quality purposes only, the systems discharging to Ponds EP-16 & CP-13 are modeled since any others will discharge directly to Bear Creek. Ponds discharging to Bear Creek located in erosion prone areas are classified as sediment traps.

This Stormwater Management Plan classifies each body of water within the Mayo Run Watershed Drainage Trunk according to its intended use as a highly aesthetical body of water or treatment basin.

Highly aesthetical water bodies are in the following two classes:

Class I: Scenic recreation (high water quality standards).

Class II: Wild life habitat (appropriate water quality standards).

Treatment basins are in the following three classes:

Class III: Nutrient trap (nutrient removal efficiency greater than 50%).

Class IV: Sediment Basin (sediment removal and nutrient removal efficiency between 20 & 50%)

Class V: Storm water storage (nutrient removal is negligible).

Ponds EP-12 and CP-13 are classified as highly aesthetical and have a high and appropriate water quality standard respectively and are considered a high priority. This plan considered both water bodies in order to review storage volumes and nutrient treatment capacities available at upstream facilities. The provision of wet volumes at upstream ponds increases nutrient trapping and improves water quality in the designated water bodies where high water quality standards are desired. Appendix G contains the water quality parameters for ponding facilities located upstream of Ponds DP-12 and CP-13.

Modeling

The Pond Net model developed by William Walker, Jr., was selected for use in the Mayo Run Watershed because it meets the following conditions:

a) Predicts the phosphorus concentration in rainstorm runoff in water flowing through a large number of ponds;

b) Predicts phosphorus and runoff volumes from different types of developments;

c) Predicts phosphorus runoff rates before and after development occurs;

d) Estimates total treatment efficiencies;

e) Is available on a user friendly software.

The Walker Model is available on IBM compatible Lotus 1,2,3 software and is user friendly. It is an Empirical model (based on experimentation) and was developed from data collected through the Environmental Protection Agency (EPA), National Urban Runoff Program (NURP) and calibrated for development conditions of the cities of Vadnais Heights and Eagan, Minnesota.

The Pond Net Model estimates the average annual water quality conditions in ponds and shallow lakes. The model can also estimate the phosphorus removal efficiency of a large number of hydrologically connected ponds. The model predicts phosphorus removal in a pond as a function of wetpond volume, depth, and configuration of the pond.

A limitation of the Pond Net model is its inability to predict phosphorus concentrations in large, deep waterbodies. In general, the pond net model is not effective for waterbodies larger than 20 acres or with mean depths greater than 10 feet, which is not the case of any of the ponds located in the Mayo Run Watershed.

Modeling was performed utilizing the following phosphorus concentrations based on our experience and on studies performed for different types of developments in the State of Minnesota.

TABLE A

LAND USE PHOSPHORUS CONCENTRATIONS

Land Use	P-Concentration (ppb)	Model Parameters Summer Runoff Coefficient * (%)	P-Export Coefficient (lbs/ac.)
Agriculture	450	.12	0.62
Industrial/Commercial	600	.75	2.80
Single Family Residential	450	.22	0.65
Multi-Family Residential	500	.32	1.05
Open/Undeveloped	200	.09	0.09

* 2 year storm frequency (2.8" of precipitation)

ppb = Parts per billion/Micrograms per liter

The phosphorus concentration and the runoff efficiency for the various land use types will have to be calibrated in the future using the water quality characteristics of Pond CP-13.

IV. STORM DRAINAGE SYSTEM DESCRIPTION

GENERAL

The Mayo Run Watershed has been divided into three major drainage systems (East, Central and West) as shown in Figure 8 at the back of this report.

The major districts are further subdivided as shown on Figure 9 at the back of this report. Each subdistrict is labeled with the associated district letter and a number to differentiate it from the other subdistricts. The areas in acres of each subdistrict are listed in Appendix A.

Ponding areas, storm sewer locations and open channels are also indicated on Figure 8. The storm water conveyance system is broken into segments between ponds. Capacities of the proposed storm drainage facilities are presented in Appendix B. The proposed system is shown in solid black lines. The pipe sizes that are shown are based on the required capacity of each line at assumed grade. While the pipe size and grade can change at the time of final design, the pipe capacity of each line should only be changed as a result of additional engineering analysis.

Ponding areas are identified by the letter of the major district in which they are located followed by a P and a number to identify the ponding area. Pond data involving tributary area, pond area, storage volume, normal water level, high water level and pond outflow are presented in Appendix C.

The storage volume of a pond is as important to an adequately maintained storm drainage system as the peak outflow. The area and depth of a pond can vary from those presented in Appendix C as final development occurs, but the storage volume must be provided to insure that downstream flow capacities of storm drainage facilities are not exceeded.

Peak pond outflows that are given in Appendix C are based on discharges through either a pipe or an orifice with the pond level at the high water level (HWL). In the case of two-staged outlets (normally an orifice and weir) or any controlled outflow, proper computer modeling of all conditions should be a requirement.

Appendix F contains the calculated water levels during 10, 50, 100, and 500 year storm events with fully developed conditions of the watershed.

CENTRAL DISTRICT

Drainage Area: 683 Acres

System Description: The district is served by two trunk lines converging at the end of the district. One trunk line services the area south of T.H. 14 and discharges into an existing 2600 foot channel. The second trunk line runs through the middle of the district east to west and consists of an existing channel with very mild slopes.

Number of Ponds: 14

Ponds Controlled By Two Stage Outlets: CP.15

Water Quality Ponds: CP-10, 13, 14, 15

Water Quality: One of the main goals of this plan is to preserve, maintain and enhance the water quality of Pond CP-13 which has the highest water quality standards in the system. Ponds CP-10 and 15 will reduce the amount of nutrients reaching Pond CP-13.

Pond CP-15 is a nutrient trap and will help to maintain the quality of water in downstream areas, especially when development occurs in subwatershed C-16.

Pond CP-10 is connected to Pond CP-13 which increases the volume of flow to Pond CP-13 and reduces the impact to water quality associated with future development.

Future monitoring of water quality parameters in Pond CP-13 can be used to determine the overall performance of the system. The water quality parameters can be used to evaluate any possible deterioration of the waterbody.

District Priorities: Priorities in this district are related to aesthetics and depend on the amount and proximity of development occurring south of T.H. 14. Most of the ponds in the lower portion of the watershed will take a minimum of 3 years to become well defined and for the surrounding vegetation to become established. As development occurs south of T.H. 14, the proper ponding areas should be incorporated. Figure 8 shows the ponding facilities necessary to avoid undesirable flooding.

Pond CP-14 will be defined by the future road alignment shown in Figure 6. If ponds EP-11 and EP-16 are not built in the near future, the construction of Pond CP-14 becomes a high priority.

Comments: Ponding in this district is directly related to future development. Ponds located in this district should be constructed concurrent with any development in drainage subdistricts tributary to each pond.

Ponds CP-2, 5, 6 and 7 are the result of local flooding from storms exceeding the discharge capacities of the existing culverts under T.H. 14. The drainage areas tributary to these ponds are characterized by steep slopes and are prone to erosion. Excavation below the existing culvert invert elevations is recommended to provide some sediment trapping capacity.

Pond CP-14 can be classified as Class III, IV, or V, depending on the desired aesthetics of the surrounding area, or classified as part of a park corridor. The influence of groundwater, located only 3 feet below the existing ground elevation allows this pond to either be a wet pond or a wetland. Either option can be implemented in the future, proper modeling for storm water parameters is recommended.

Pond CP-11 does not have to be built until development in Subdistrict C-17 causes runoff peaks from this subdistrict to increase above 45 cfs during a five-year storm.

EAST DISTRICT

Drainage Area: 706 Acres

System Description: The district is served by two main trunks that merge at Pond EP-16 which controls and regulates the final outflows from this district to a peak discharge of 36 cfs.

Number of Ponds: 15

Ponds Controlled By Two Stage Outlets: EP-1, 2, 9, 10, 11, and 13.

Water Quality Ponds: Ponds EP-1, 2, 14, and 15 are designed to trap nutrients from upstream residential developments.

Water Quality: All ponds except EP-4, 9 & 13.

Ponds EP-3, 5, 6, 7, and 8 are located in the golf course and are intended to treat nutrients associated with the golf courses maintenance. Monitoring of nutrients in Ponds EP-4 and DP-9 can be implemented in the future and will show the treatment efficiencies of the ponds in the golf course. Ponds EP-10 and 11 are designed to trap nutrients from highly impervious areas. Pond EP-11 will play a very important role in the characteristics and conditions of Pond EP-12, classified as a wildlife habitat pond.

Pond EP-16 is a large waterbody servicing 162 acres of park land dedicated to grow corn to feed the geese.

District Priorities: Pond EP-11 will protect downstream areas from nutrient overloading and will control runoff peak flows from destroying and disturbing the wetland vegetation of Pond EP-12.

Pond EP-16 will control peak flows during severe storms whereby minimizing downstream flooding. The construction of a pond outlet with the capabilities to adjust and regulate outflows as development occurs or when upstream ponds are integrated in the system is highly recommended.

Comments: Ponds EP-4, 9, and 15 can be created by excavating below the invert elevation and by controlling the flow capacities of the existing culverts.

Pond EP-12 is a wildlife pond that will require the construction of a hydraulic structure to produce sheet flow. Agricultural Management Practices and erosion control practices should be observed in the district to minimize sediment transport and avoid excessive nutrient loads associated with sediment and chemicals such as fertilizers and pesticides. Water quality parameters in Pond EP-16 are difficult to estimate due to groundwater inflows. Monitoring of water quality parameters is recommended on an annual basis. Pond EP-13 occurs due to the existing 5-year storm capacities of the culverts under 36 Avenue S.E. Maintenance of the culvert inlets is recommended as well as the provision of oversize trash racks to avoid potential plugging during severe storms which could result in undesirable flooding of the homes located near by.

Ponds located in the golf course will not only help to control runoff but will also trap nutrients associated with golf course care. The need for implementation of this type of ponds can be determined as water quality in Ponds EP-11, EP-12, and EP-16 deteriorates.

WEST DISTRICT

Drainage Area: 179 Acres

System Description: The district is served by two channels, one being an existing channel running south to north and also by the final section of the Mayo Run trunk channel running east to west.

Number of Ponds: 2

Ponds Controlled By Two Stage Outlets: None

Water Quality Ponds: None

Water Quality: There is no need for water quality treatment in this district.

District Priorities: The main priority in this district is to improve the channel's slope which will alleviate any excessive flooding in the final section of the main trunk. Pond WP-2 is the result of high water levels in the main trunk channel created by the limited flow capacity of the 4'x10' existing culvert under 13th Avenue.

Only proper grading of Pond WP-2 is necessary to meet the parameters recommended in this report. Pond WP-1 controls and regulates the outflows from 41 acres of drainage. The priority of this pond depends on the construction of Ponds EP-16 and CP-14. The construction of WP-1 can be postponed until development in Subdistrict C-17 occurs and assuming that CP-14 and EP-16 are built. We recommend the provision of the proper channelization along 10th Street S.E. to the Bear Creek to avoid runoff from Subdistricts W-1 and W-2 to Pond in Marion Street.

Comments: This district is mostly developed and local ponding occurs when the transport capacities of the existing storm facilities are exceeded.

Ponds WP-1 and WP-2 are located in the existing floodplain and are intended to control local flooding by defining appropriate areas for ponding.

Mayo Run - Bear Creek Floodplain

As part of this study a review and flood analysis of the 1987 Flood Insurance Study was performed in order to determine the flow/hydraulic characteristics at the location where the Mayo Run meets the Bear Creek.

During the process of setting up an HEC-2 model for the lower portion of the Mayo Run, it was determined that the final flow and flood conditions are influenced strongly by the hydraulic characteristics of Bear Creek and the local topography during a 100 year and 500 year events.

It is important to differentiate between the two different types of flooding that can occur at the outlet of the Mayo Run Watershed, flooding due to a 100 year storm (short-duration, high-intensity) and flooding due to a 100 year Spring runoff event (Snowmelt event).

Flooding due to a 100 year storm event.

Flooding due to a 100 year short-duration, high intensity event is typically the result of a Summer storm. The proposed improvements contained in this management plan will minimized this type of flooding at the existing 4' by 10' culvert under 13th Avenue by allowing the expected water level to peak at 1000.5.

Flooding due to a 100 year snowmelt event.

Flooding due to a 100 year runoff event is mainly associated with a Snowmelt event. The proposed facilities in this management plan are designed considering the snowmelt phenomena. The expected flooding conditions due only to the Mayo Watershed drainage area will be less than those expected under existing development conditions. The flooding occuring in the Mayo Run Outlet is due to water levels occuring in the Bear Creek during Spring runoff events. The flooding is not caused directly by a backwater effect into the Mayo Run Outlet, but caused by the high water levels along the Bear Creek in the sections between 6th Street S.E. and 12th Street S.E.

The depths of flow in the Bear Creek that occur during 100 and 500 year flood events force the water to "overtop" the (not very well define) creek's east-bank. The overtopping occurs mainly through two corridors intercepting 15th Avenue;

1) at 10th Street SE

2) at 8 1/2 Street SE

1) Flood corridor at 15th Avenue and 10th Street SE

The overtopping occurring during a 100 year event at 10th Street SE can be expected to have low depths above elevation 1007.0 (road centerline) . The flows reaching this point are channelized to the north by 15th Avenue. The tendency of these flows would be to pond in the lowest point of 15th Avenue which corresponds to the outlet point of the Mayo Run Watershed.

2) Flood Corridor at 8 1/2 Street SE

The overtopping expected to occur during a 100 year flood event at 8 1/2 Street SE is 2.2 feet above the elevation 1003.1 (road centerline). At this location the flows will merge with the discharges conveying at 10th Street to become a large street/interception pond.

The size and length of the pond created during a 100 year flood event is expanded during a 500 year flood event since the expected water levels are aproximately 2.5 feet above those occurring during a 100 year flood event.

Existing Conditions

A simplified model was developed to address the ponding priorities in the Mayo Run Watershed. The purpose of this model was to estimate the ponding required to increase the level of protection against the flooding of properties located east and west of 15th Avenue.

Flood conditions resulting from 10, 50, and 100 year storms were calculated at three different locations considering existing conditions in the watershed. The three locations are Ponds EP-16, CP-13 and WP-2. Pond EP-16 is the result of high peak flows that exceed the capacity of the three arch culverts under 30th Avenue S.E. Pond CP-13 is an existing pond without a well defined outlet and currently serves mostly undeveloped land. Pond WP-2 can be identified as the final outlet of the watershed since its high water level correlates to the elevation east of 15th Avenue.

After modeling existing conditions, the model was modified to determine ponding priorities that will create a higher level of protection at 15th Avenue. The ponds proposed in this report have the highest storage efficiencies and were added to the existing model. Ponds CP-12, EP-15, EP-12 and EP-16 have the largest storage volumes of the proposed ponds. Pond EP-16 was assumed to provide 36.2 AF of storage for the purpose of defining the priority ponding. Table 4 summarizes the results of existing conditions and the provision of Ponds CP-12, EP-15, EP-16, and EP-12.

TABLE 4

ESTIMATED WATER LEVELS

	Existing Conditions Storm Event (Years)			With Priority Ponding Storm Event (Years)		
Location	10	50	100	10	50	100
EP-16	1023.2	1025.0	1026.0	1022.0	1024.0	1025.0
CP-13	1010.7	1011.5	1012.0	1010.7	1011.5	1012.0
WP-2	1002.0	1003.0	1004.0	1000.0	1001.0	1002.5

The conditions in CP-13 remain the same since no upstream ponding was added.

The construction of Ponds CP-12, EP-15, EP-16 and EP-12 protects the lower reach of the watershed against a 50 year storm and results in minor flooding during a 100 year storm. The level of protection is increased as other ponds are constructed in the upper parts of the watershed.

The following list ranks the priority ponds necessary to control flooding in the watershed. The high priority ponds are contained in Group A. The ranking is arranged also in Groups B and C. Group B contains ponds to be developed in the future as a long-term goal. Group C contains ponds that will be needed as development occurs throughout the watershed.

GROUP A

Rank	Pond	Comment
1	EP-16	Reduces downstream flooding.
2	CP-12	Controls flows from the Central District. (Pond CP-14 is an alternative to this pond)
3	EP-12	Supplements EP-16. This pond will be a wildlife habitat and requires time to become established.
4	EP-15	Supplements EP-16. Also helps to reduce road flooding.

GROUP B

Rank	Pond	Comment
5	EP-11	Improves water quality in EP-12 and protects it from large increases in flow.
6	CP-14	Increases level of protection to downstream areas.
7	WP-1	Confines flooding between Points L and O.
8	WP-2	Adds protection to 15th Avenue and reduces park flooding.

GROUP C

Rank	Pond	Comment
9	EP-1 to 9	Controls flow from severe storms and improves downstream water quality.
10	CP-1 to 9	Reduces local flooding in developed area.

V. COST ANALYSIS

The primary purpose of this study was to create a Comprehensive Storm Sewer Plan for the Mayo Run Watershed. A part of that objective is the determination of the total cost of the system. Cost estimates presented in this report are based on 1990 construction costs and can be related to the value of the Construction Cost Index of 4701. Future changes in this index are expected and will result in future cost changes in the proposed facilities. The general locations shown on Figure 8 of the storm drainage facilities served as a basis for the cost estimate.

The cost of trunk storm sewers, pond acquisition, and development costs are included in the cost estimates. Some ponds will have to be acquired prior to development. Lateral systems are not included. All lateral sewers will be installed as development occurs and will be totally assessed to the developing property. Trunk facilities are listed in Appendix C.

The cost of a storm sewer system can be divided between trunk costs and lateral costs. Storm sewer required to convey drainage from outside a development boundaries or to interconnect storm water ponds identified in this comprehensive plan is considered trunk storm sewer. Storm sewer required to convey drainage within the development is considered lateral storm sewer.

When development occurs in an area, it is anticipated that all ponding area easements required within the area will be dedicated as part of the development. Occasionally, this includes not only the ponding areas within the area being developed, but also downstream ponding areas in undeveloped land which must be acquired. This acquisition must be carried out if the system is to function effectively and prevent future flooding.

In some cases, development inside the Mayo Run Watershed will occur in areas where the trunk storm sewer system has not yet been constructed. In these cases, the portion of the trunk system necessary in the development should be constructed at the time of development. In all cases, the City will charge assessments to the full trunk storm sewer area benefited by the improvements. The approximate cost for each of the storm sewer segments is listed in Appendix D and summarized in Table 5.

Storm drainage system total costs and costs per square foot are presented below in Table 6. Costs per square foot vary for each land use type as a function of the ratio of the runoff coefficient for that land use type and average runoff coefficient of 0.4, which represents single family residential land use. In this manner, a variation in assessment rate according to benefit is made directly proportional to the rate of runoff expected, as shown below in Table 6.

The trunk storm sewer system and the cost of constructing storm sewer should be reviewed each year. New rates should be established to reflect the change in cost of the storm sewer construction each year. At intervals no greater than five years, the entire storm sewer system should be re-evaluated and the cost of the remaining trunk system construction refigured and compared to the areas still available for assessment.

An alternative method of expressing the same concept is by using only one assessment rate with a standardized runoff condition. The term equivalent square foot is used in this report to represent the standardized condition. An equivalent square foot represents one square foot of area having a 0.4 runoff factor. Thus, again making the assessment directly proportional to the runoff, one square foot of area having a 0.6 runoff factor would be represented by 1.5 equivalent square feet. Assessment rates for each drainage district using equivalent square feet are presented in Table 6.

TABLE 5

FUTURE TRUNK STORM SEWER COSTS

Area	Future Construction + Overhead Design, Management
EAST	$421,920
CENTRAL	376,370
WEST	28,360
Total	$826,650
Pond Acquisition & Development	--
Pond Outlets (Control Structures)	150,000
	$976,650
Channel Improvements	421,300
10% Contingencies	139,850
Grand Total	$1,537,800

TABLE 6

STORM DRAINAGE COST ALLOCATION

	C Runoff Coefficient	Area Ac.	Equiv. Area Ac.	Assessment Rate $/Sq.Ft.	Total Cost
1. Park/Woods/Golf	0.3	713	535	0.014	$434,800
2. Low Density Residential	0.4	196	196	0.019	162,200
3. Medium Density Residential	0.5	246	307	0.024	257,200
4. Industrial/Commercial	0.8	38	76	0.038	62,900
5. Planned Development	0.8	375	750	0.038	620,700
TOTAL			1864		$1,537,800

VI. SUMMARY AND RECOMMENDATIONS

This report provides the City of Rochester with a Stormwater Management Plan for the Mayo Run Watershed that will serve as a guide for the initial construction and future expansion of the storm drainage system. The storm drainage system is shown on Figures 8 and 9 at the back of this report. The following issues have been incorporated into this plan.

1. Division of the Watershed into major drainage districts and subdistricts. Based on existing grading plans and natural topography.
2. Relation of storm water runoff to anticipated land usage.
3. General routing of storm sewers and open channels.
4. Design flows and sizing for all proposed facilities.
5. Type, tributary area, storage volume and water levels of all required ponding areas.
6. Classification of water bodies by desired use considering existing physical characterization for water quality purposes.
7. Define water quality parameters for each water quality pond.
8. Estimated construction costs of the storm drainage system as shown.

The importance of incorporating ponding areas as recommended in the Stormwater Management Plan cannot be overemphasized. The ponding areas will provide the necessary storage required to retain high intensity storm water runoff peaks and maintain the watershed's outflow below the capacity of downstream facilities. The storage requirements established for each pond must be maintained to prevent flooding of property. The discharge flow rates computed for each ponding area must also be maintained to insure the storage volume provided is used and downstream flows are not exceeded.

Site grading and drainage plans must be required for all developments in the Watershed. These plans will provide the detail necessary to design an adequate lateral storm sewer system and insure drainage of each parcel within the development. Lots should be carefully examined to insure that construction near the lowest areas will not become susceptible to flooding during peak storms. Site grading and drainage plans should show the required site grading to eliminate side and backyard drainage problems.

Verification of compliance with the grading plans should be noted by both engineering and building inspectors during construction of storm sewers and buildings.

Pond types, elevations, and surface areas of proposed ponds are not intended to be rigid and can be adjusted in final design if desirable. The ponds have been selected to generally take advantage of natural depressions and therefore require little or no excavation to develop. Many of the pond normal water levels should provide proper wet volumes for water quality purposes. Properly designed ponds will improve water quality through nutrient removal. Also, low lands with water tables near the surface were considered to be upgraded to perform as wetlands. Submerged outlets for deeper ponds or skimmer structures are recommended to prevent floating solids from being transferred from one pond to another. This will also facilitate the cleanup of an oil spill or other floating material by confining it to only one pond in a series of ponds.

The pipe sizes shown may vary due to grades and lateral requirements in the final design. The pipe routes shown should be followed as closely as practical since they take advantage of existing ravines and lowlands. Major route changes will add to the depth and cost of the storm drainage system.

The following recommendations are presented for the City Council's consideration based upon the data compiled in this report.

1. The Stormwater Management Plan as presented herein be adopted by the City of Rochester as a basic guideline for the future Mayo Run Watershed development.
2. Ponding areas be established as shown on Figure 8 and made a part of the storm drainage system with the storage volumes required as presented in Appendix C. Encourage additional ponds in new developments. Require a detailed analysis before changing any ponding elevation.
3. Final highwater levels governing building elevations adjacent to ponding areas and floodplains be established as an area develops or when drainage facilities are constructed for an area. A freeboard of 2 feet is highly recommended.
4. Establish and maintain overflow routes to provide relief during extreme storm conditions which exceed design conditions.
5. Establish and maintain a maintenance program to ensure the successful operation of the storm drainage system.

6. Develop a Water Quality Monitoring Program for selected waterbodies as recommended in this report. This will provide long term records of water quality to assist in design improvements where possible.
7. Establish, maintain, and enforce erosion control criteria for new developments.
8. Skimmers, or other outlet controls and siltation basins be provided and maintained in ponds located upstream of direct contact waterbodies. This will prevent floating or settleable solids from reaching these important waterbodies.
9. Implement a recording rain gauge within the Mayo Run Watershed. This will provide valuable information for final design of the proposed facilities when development occurs.

APPENDIX A

DRAINAGE AREA

Area Designation	Area (Acres)
EAST DISTRICT	
E-1	44
E-2	55
E-3	16
E-4	22
E-5	31
E-6	12
E-7	13
E-8	6
E-9	16
E-10	31
E-11	12
E-12	9
E-13	20
E-14	30
E-15	66
E-16	33
E-17	15
E-18	13
E-19	100
E-20	162
Total East	706
CENTRAL DISTRICT	
C-1	34
C-2	54
C-3	26
C-4	14
C-5	32
C-6	11
C-7	9
C-8	25
C-9	21
C-10	17
C-11	21
C-12	108
C-13	35
C-14	72
C-15	68
C-16	28
C-17	36
C-18	32
C-19	90
Total Central	683
WEST DISTRICT	
W-1	20
W-2	19
W-3	34
W-4	38
W-5	9
W-6	6
W-7	5
W-8	11
W-9	23
W-10	14
Total West	179

Total East	706
Total Central	683
Total West	179
TOTAL	1,568 Ac.

APPENDIX B

PIPE FLOWS

Flow From	Flow To	Tributary Areas (Acres) Direct	Ponded	Total	Design Capacity (cfs)
East District					
EP-1	EP-5	44	0	44	3.4
EP-5	EP-6	31	44	75	4.0
EP-6	EP-4	12	75	87	4.5
EP-3	EP-4	16	0	16	2.0
EP-2	EP-8	55	0	55	3.6
EP-7	EP-8	16	0	16	1.0
EP-8	EP-9	31	71	102	5.8
EP-9	EP-11	27	102	129	34.6
EP-10	EP-11	44	0	44	32.5
EP-11	B	66	173	239	70.0
A	B	13	0	13	12.0
B	EP-12	0	239	239	82.0
EP-12	EP-16	25	239	267	16.5
EP-4	EP-16	28	103	131	7.2
EP-13	EP-14	33	0	33	66.0
EP-14	EP-16	13	33	46	3.5
EP-15	EP-16	100	0	100	3.1
EP-16	C	162	544	706	36.6
West and Central Districts					
CP-7	C	25	0	25	3.7
C	D	0	731	731	40.3
D	CP-14	19	731	750	59.3
CP-1	CP-2	34	0	34	3.0
CP-3	CP-5	14	0	14	3.3
CP-4	CP-6	32	0	32	2.8

APPENDIX B

PIPE FLOWS

Flow From	Flow To	Tributary Areas (Acres) Direct	Ponded	Total	Design Capacity (cfs)
West and Central Districts (Cont'd)					
CP-6	CP-5	9	32	41	3.2
CP-5	CP-9	11	55	66	5.6
CP-9	F	21	66	87	4.9
CP-2	F	30	34	64	12.7
F	G	0	151	151	17.6
G	H	17	151	168	39.3
CP-8	H	21	0	121	7.7
H	I	0	189	189	47.0
J	I	53	0	53	62.4
I	WP-1	9	242	251	117.2
WP-1	N	41	251	292	59.0
CP-10	CP-13	108	0	108	4.3
CP-15	CP-13	28	0	28	4.0
CP-13	E	68	136	204	15.7
CP-14	E	71	750	821	45.7
CP-12	K	104	0	104	34.9
E	K	0	1025	1025	63.7
K	L	0	1129	1129	85.8
CP-11	L	49	0	49	13.3
L	M	0	1178	1178	95.3
M	N	23	1470	1493	131.5
N	O	17	1493	1510	146.1
WP-2	O	38	0	38	9.3
O	O1	0	1548	1548	175.0
P	Q	20	0	20	30.0

APPENDIX C

POND DATA

Pond #	Tributary Area (Acres) Direct	Ponded	Total	Pond Area @ NWL (Acres)	Pond Area @ HWL (Acres)	Storage Volume (Ac-Ft)	Normal Water Level	High Water Level	Pond Outflow (cfs)	Two-Stage Outlet Required
EAST DISTRICT										
EP-[illegible]	44	0	[illegible]	[illegible]	3.0	10.3	1130	1136	3.4	Yes
EP-[illegible]	[illegible]	0	[illegible]	[illegible]	2.5	11.9	1123	1128	3.6	Yes
EP-[illegible]	[illegible]	0	[illegible]	[illegible]	0.7	2.4	1094	1099	2.0	No
EP-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.5	5.6	1051	1055	7.2	No
EP-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.2	5.0	1082	1088	4.0	No
EP-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.7	2.5	1066	1071	4.5	No
EP-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.7	2.5	1094	1099	1.0	No
EP-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.1	4.6	1078	1085	5.8	No
EP-[illegible]	27	[illegible]	[illegible]	[illegible]	0.9	2.4	1062	1066	34.6	Yes
EP-10	[illegible]	[illegible]	[illegible]	[illegible]	1.3	5.5	1070	1075	32.0	Yes
EP-[illegible]	66	[illegible]	[illegible]	[illegible]	3.4	13.9	1051	1055.5	70.3	Yes
EP-[illegible]	28	[illegible]	[illegible]	[illegible]	5.0	20.7	1044	1048.5	16.5	No
EP-[illegible]	33	0	[illegible]	[illegible]	0.3	1.0	1067	1070	66.1	Yes
EP-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.0	8.3	1055	1060	3.5	No
EP-15	100	[illegible]	[illegible]	[illegible]	4.2	23.8	1048	1054	3.1	No
EP-16	[illegible]	[illegible]	[illegible]	[illegible]	8.0	36.2	1019	1024	36.6	No

Pond #	Tributary Area (Acres) Direct	Ponded	Total	Pond Area @ NWL (Acres)	Pond Area @ HWL (Acres)	Storage Volume (Ac-Ft)	Normal Water Level	High Water Level	Pond Outflow (cfs)	Two-Stage Outlet Required
CENTRAL DISTRICT										
CP-1	34	0	34	0.4	0.7	3.0	1054	1060	3.0	No
CP-2	30	34	64	0.6	0.8	2.4	1033	1037	12.7	No
CP-3	14	0	14	0.4	0.6	1.8	1079	1084	3.3	No
CP-4	32	0	32	0.9	1.2	5.0	1080	1086	2.8	No
CP-5	11	55	66	0.6	0.8	3.2	1055	1060	5.6	No
CP-6	9	32	41	0.4	0.6	1.8	1070	1074	3.2	No
CP-7	25	0	25	0.7	1.0	3.1	1062	1068	3.7	No
CP-8	21	0	21	0.8	1.0	3.5	1003	1007	7.7	No
CP-9	21	66	87	1.4	1.7	6.1	1036	1040	4.9	No
CP-10	108	0	108	4.5	5.0	24.0	1011	1016	4.3	No
CP-11	49	0	49	1.7	2.1	9.5	1000	1005	13.3	No
CP-12	104	0	104	8.0	9.0	25.4	1003.5	1006.5	34.9	No
CP-13	68	136	204	3.6	9.1	20.4	1008	1011.5	15.7	No
CP-14	71	750	821	7.5	9.0	28.5	1008	1011.5	45.7	Yes
CP-15	28	0	28	1.8	2.1	8.7	1010	1014	4.0	No
WEST DISTRICT										
WP-1	41	251	292	2.8	3.2	17.6	999	1005	59.0	No
WP-2	38	0	38	2.6	3.0	4.2	999	1000.5	64.0	No

APPENDIX D

DESIGNED TRUNK COST *

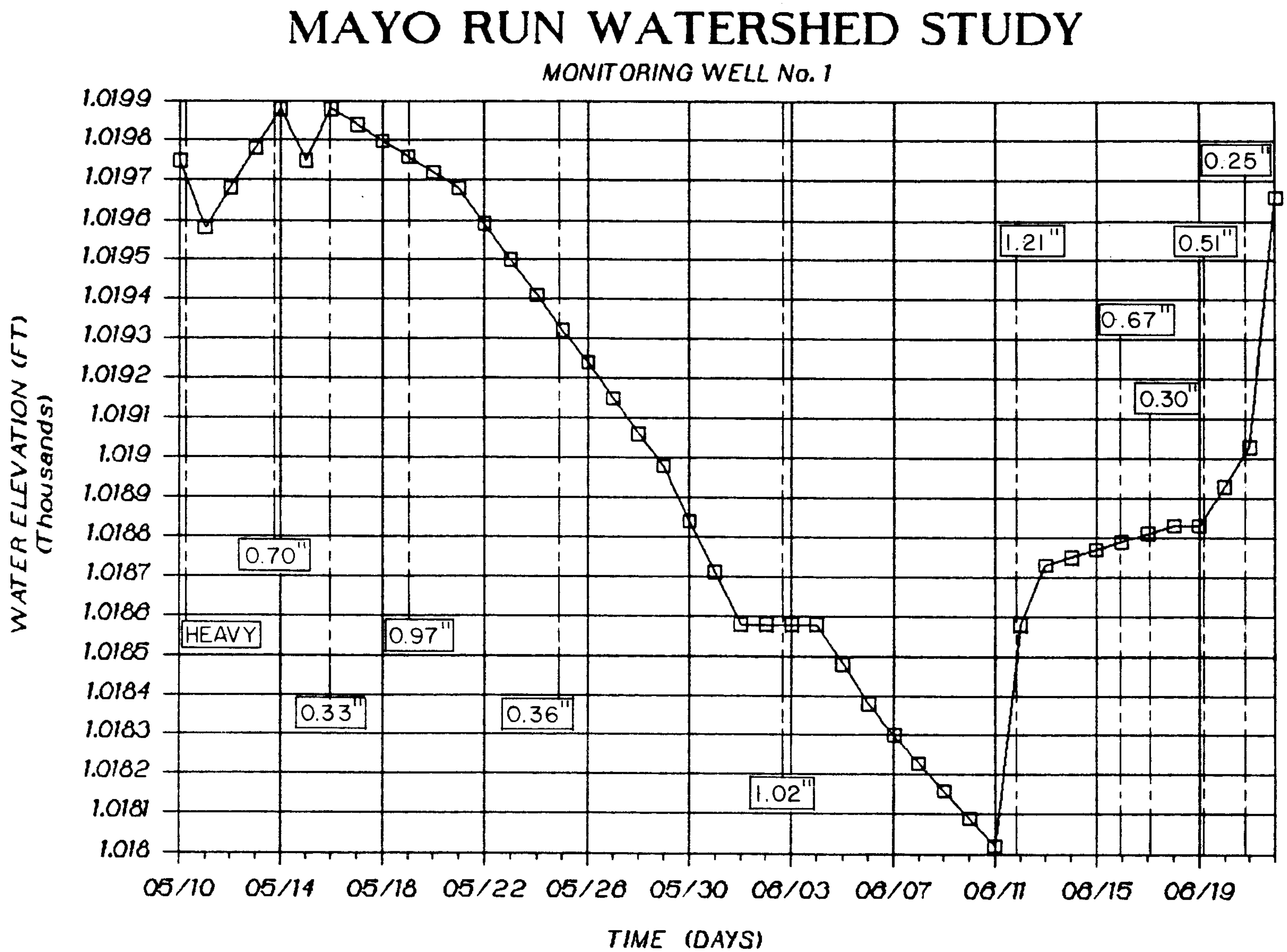
District	Flow From	Flow To	Design Cap.(cfs)	Pipe Size (in)	Length (Ft.)	Cost per Foot	Total
East	EP-1	EP-5	3.4	12	1,100	$ 38.9	$ 42,790
	EP-5	EP-6	4.0	12	250	38.9	9,725
	EP-6	EP-4	4.5	12	400	38.9	15,560
	EP-3	EP-4	2.0	12	850	38.9	33,065
	EP-2	EP-8	3.6	12	1,200	38.9	46,680
	EP-7	EP-8	1.0	12	250	38.9	9,725
	EP-8	EP-9	5.8	12	450	38.9	17,505
	EP-9	EP-11	34.6	24	350	54.3	19,005
	EP-10	EP-11	32.0	24	650	54.3	35,795
	EP-4	EP-16	7.2	15	1,800	42.15	75,870
	EP-14	EP-16	3.5	12	1,900	38.9	73,910
	EP-15	EP-16	3.1	12	1,100	38.9	42,790
							$421,920
Central	CP07	C	3.7	12	2,000	38.9	77,800
	CP-1	CP-2	3.0	12	400	38.9	15,560
	CP-3	CP-5	3.3	12	450	38.9	17,505
	CP-4	CP-6	2.8	12	300	38.9	11,670
	CP-6	CP-5	3.2	12	350	38.9	13,615
	CP-5	CP-9	5.6	12	250	38.9	9,725
	CP-9	F	4.9	15	800	42.15	33,720
	CP-2	F	12.7	18	750	46.1	34,575
	F	G	17.6	24	400	54.3	21,720
	CP-8	H	7.7	18	200	46.1	9,220
	CP-10	CP-13	4.3	15	1,000	42.15	42,150
	CP-15	CP-13	4.0	15	650	42.15	27,400
	CP-13	E	15.7	24	200	54.3	10,860

* Cost of Open Channel Improvements are not included

APPENDIX D (CONT'D)

District	Flow From	Flow To	Design Cap.(cfs)	Pipe Size (in)	Length (Ft.)	Cost per Foot	Total
Central	CP-14	E	45.7	36	200	91.8	18,360
(cont'd)	CP-12	K	34.9	42	200	124.95	24,990
	CP-11	L	13.3	21	150	50.0	7,500
							$376,370
West	WP-1	M	59.0	36	200	91.8	18,360
	WP-2	O	9.3	21	200	50.0	10,000
							$ 28,360
	GRAND TOTAL						$ 826,650

R36302AP



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

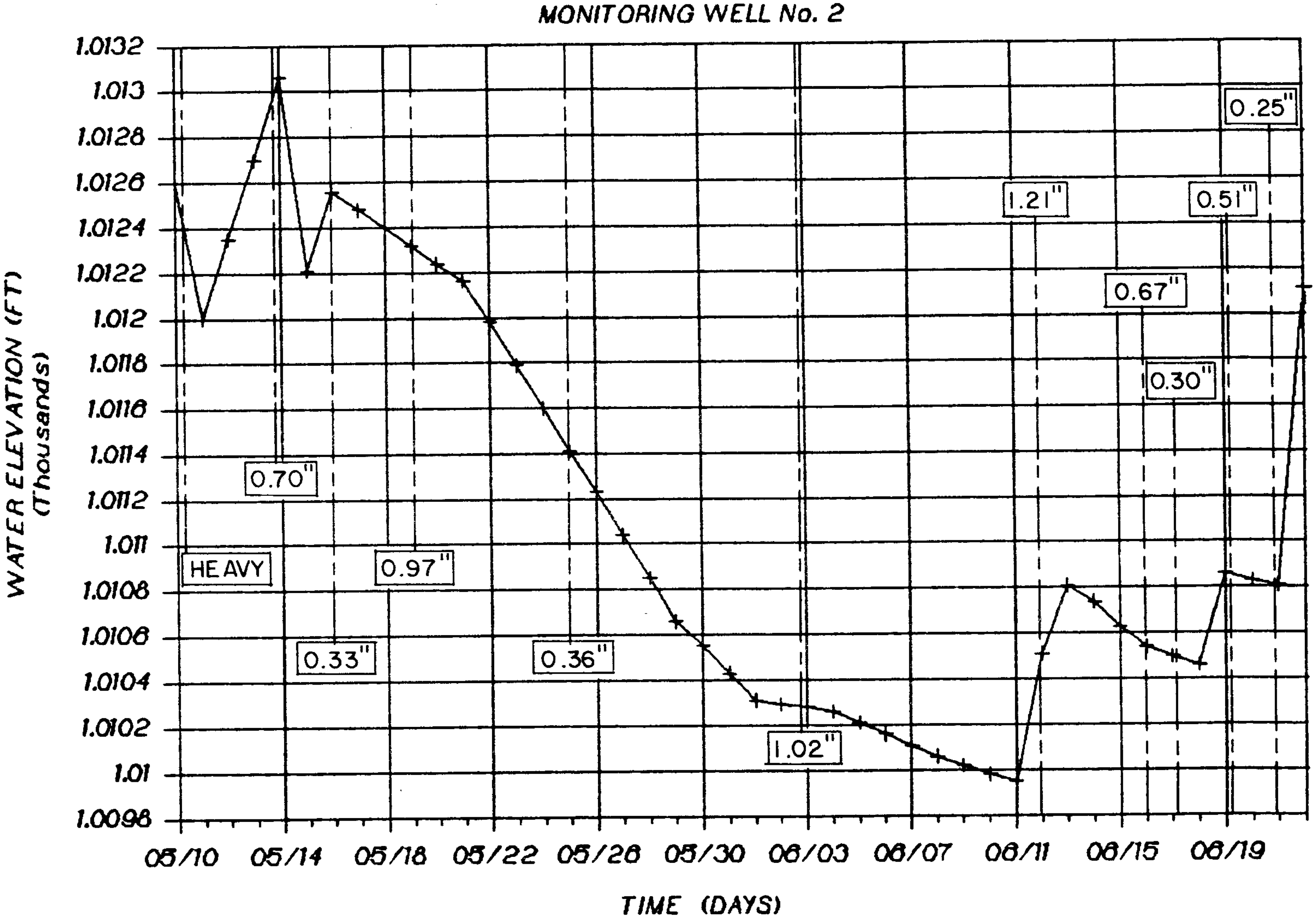
ROCHESTER, MINNESOTA

WELL NUMBER 1: WATER TABLE MONITORING AND STORM INTENSITIES (1990)

9/19/90 COMM. 36302

Bonestroo Rosene Anderlik & Associates
Engineers & Architects
St. Paul, Minnesota

MAYO RUN WATERSHED STUDY



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA

WELL NUMBER 2: WATER TABLE MONITORING AND STORM INTENSITIES (1990)

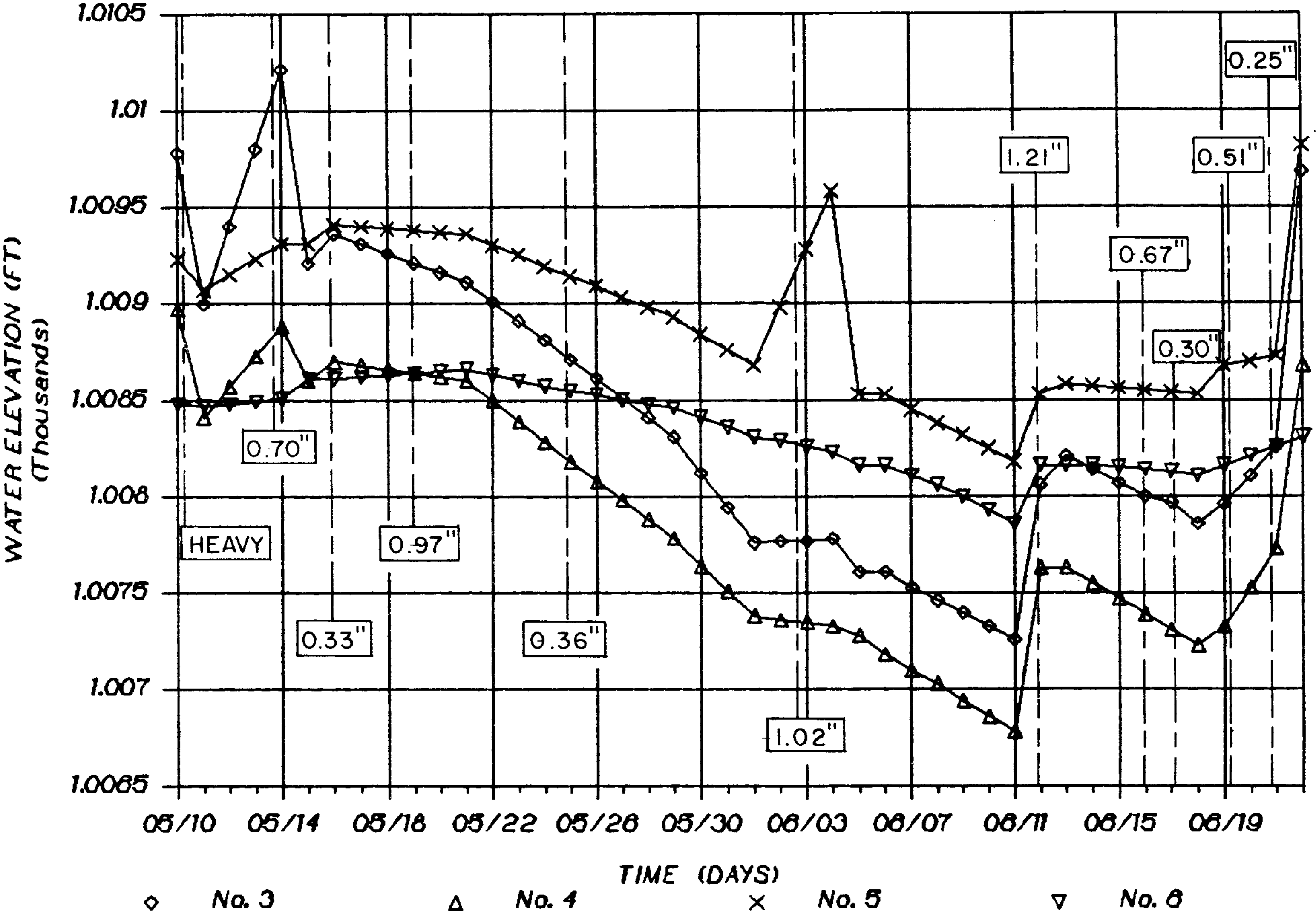
9/19/90 COMM. 36302

Bonestroo
Rosene
Anderlik &
Associates

Engineers & Architects
St. Paul, Minnesota

MAYO RUN WATERSHED STUDY

MONITORING WELLS No.3, 4, 5 AND 6



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

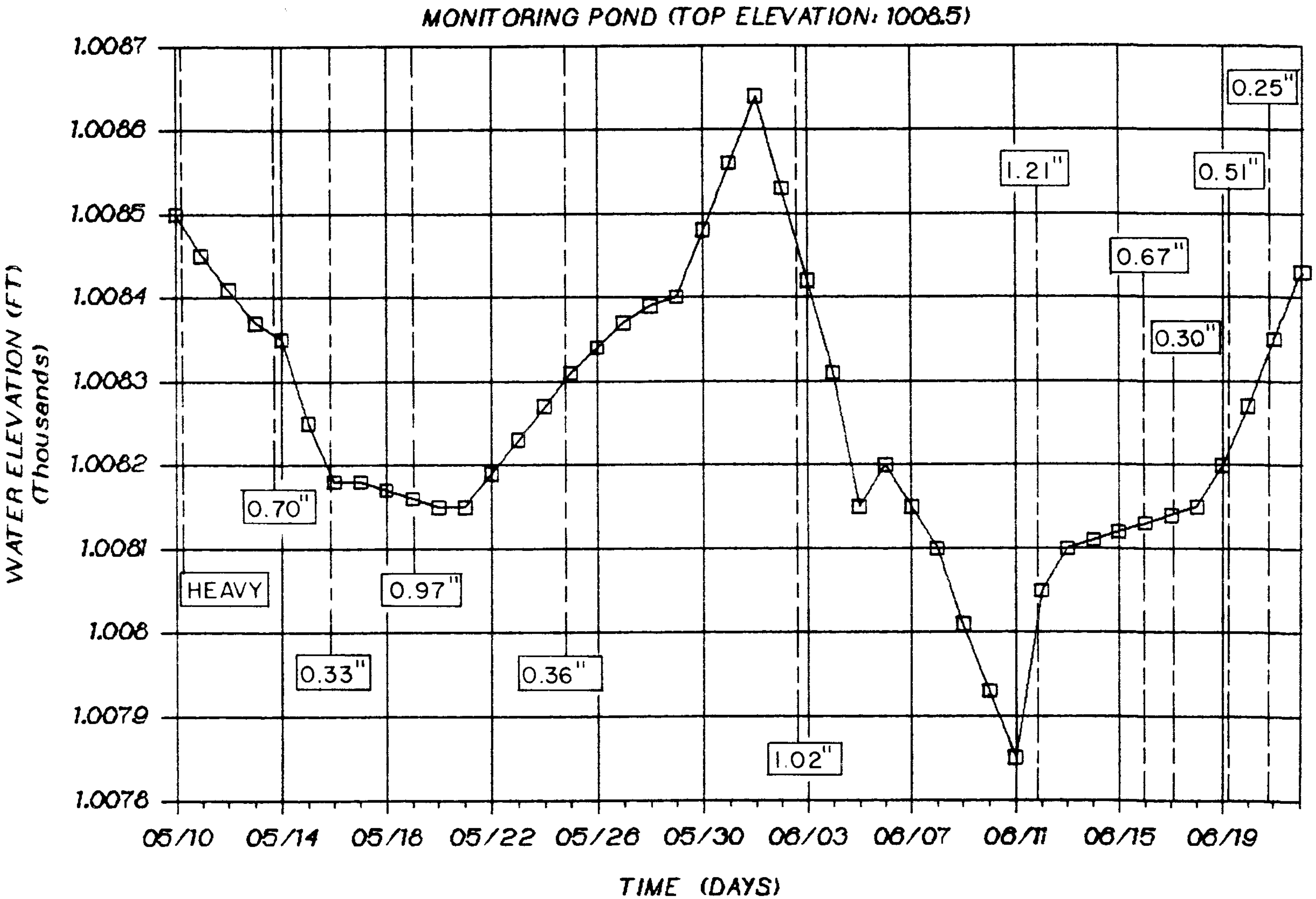
ROCHESTER, MINNESOTA

WELL NUMBERS 3,4,5 AND 6: WATER TABLE MONITORING AND STORM INTENSITIES (1990)

9/19/90 COMM. 36302

Bonestroo Rosene Anderlik & Associates
Engineers & Architects
St. Paul, Minnesota

MAYO RUN WATERSHED STUDY



STORM WATER MANAGEMENT PLAN
MAYO RUN WATERSHED

ROCHESTER, MINNESOTA

HIGH WATER LEVEL MONITORING FOR THE CENTRAL DISTRICT NATURAL POND
(MAY 10, 1990 to JUNE 22, 1990) AND STORMS INTENSITIES

9/19/90 COMM. 36302

Bonestroo
Rosene
Anderlik &
Associates

Engineers & Architects
St. Paul, Minnesota

APPENDIX F
POND WATER LEVELS
10, 50, 100 & 500 YEAR STORMS

POND HIGH WATER LEVEL

Pond #	Normal Water Level	10-YR	50-YR.	100-YR.	500-YR.
EAST DISTRICT					
EP-1	1130	1134.5	1135.5	1136.0	1136.5
EP-2	1123	1125.8	1127.2	1128.0	1128.7
EP-3	1094	1096.3	1097.6	1099.0	1099.9
EP-4	1051	1053.7	1055.2	1056.0	1056.9
EP-5	1082	1083.8	1085.1	1088.0	1090.7
EP-6	1066	1068.1	1069.5	1071.0	1072.9
EP-7	1094	1096.2	1097.7	1099.0	1100.8
EP-8	1078	1080.1	1081.3	1085.0	1086.5
EP-9	1062	1064.7	1065.4	1066.0	1066.3
EP-10	1070	1074.4	1075.1	1075.5	1076.0
EP-11	1051	1054.4	1055.1	1055.5	1056.0
EP-12	1044	1045.9	1047.2	1048.5	1050.3
EP-13	1067	1068.6	1069.1	1070.0	1070.3
EP-14	1055	1057.5	1058.7	1060.0	1061.0
EP-15	1048	1051.0	1052.6	1054.0	1055.9
EP-16	1019	1021.5	1022.8	1024.0	1025.0

APPENDIX F (CONT'D)
POND WATER LEVELS
10, 50, 100 & 500 YEAR STORMS

Pond #	Normal Water Level	10-YR	50-YR.	100-YR.	500-YR.
		POND HIGH WATER LEVEL			
CENTRAL DISTRICT					
CP-1	1054.0	1056.6	1058.2	1060.0	1062.2
CP-2	1033.0	1034.7	1035.8	1037.0	1038.8
CP-3	1079.0	1080.3	1081.8	1084.0	1084.2
CP-4	1080.0	1086.3	1083.8	1086.0	1086.9
CP-5	1055.0	1057.4	1058.8	1060.0	1061.5
CP-6	1070.0	1071.9	1073	1074.0	1075.1
CP-7	1062.0	1064.3	1065.9	1068.0	1069.0
CP-8	1003.0	1005.3	1006.2	1007.0	1008.2
CP-9	1036.0	1038.0	1039.0	1040.0	1041.0
CP-1	1011.0	1013.4	1014.8	1016.0	1017.7
CP-11	1000.0	1002.9	1004.0	1005.0	1006.3
CP-12	1003.5	1005.7	1006.1	1006.5	1007.0
CP-13	1008.0	1010.6	1011.1	1011.5	1011.9
CP-14	1008.0	1010.6	1011.3	1011.5	1011.7
CP-15	1010.0	1012.4	1013.2	1014.0	1014.8
WEST DISTRICT					
WP-1	999.0	1002.2	1003.7	1005.0	1006.6
WP-2	999.0	999.9	1000.2	1000.5	1000.8

APPENDIX G

WATER QUALITY POND DATA

Pond #	Class	Tributary Area Direct	Wet Volume Ac.	Upstream Pconc ppb	Outflow Pconc ppb	Pond Efficiency %	Minimum Depth
EAST DISTRICT							
EP-1	III	44	4.8	--	194	57	2.5
EP-2	III	55	2.4	--	200	56	2.5
EP-3	III	16	0.7	--	193	57	2.5
EP-4	IV	28	0.8	193	177	61	--
EP-5	III	31	1.4	194	176	61	2.5
EP-6	III	12	0.5	176	173	62	2.5
EP-7	III	16	0.7	--	200	56	2.5
EP-8	III	31	1.4	200	174	61	2.5
EP-9	IV	27	1.0	174	170	62	--
* EP-10	III	44	4.1	--	200	67	5.5
EP-11	III	66	7.8	200	158	64	3.0
EP-12	II	28	Wetland	158	146	67	--
EP-13	IV	33	0.2	--	321	24	--
EP-14	III	13	3.0	321	169	62	2.5
EP-15	III	100	7.4	--	167	52	2.5
EP-16	III	162	12.3	177	161	67	2.5

* This pond was modeled as a two cell pond

Pconc = Phosphorus Concentration (ppb) parts per billion

APPENDIX G (CONT'D)

WATER QUALITY POND DATA

Pond #	Class	Tributary Area Direct	Wet Volume Ac.	Upstream Pconc ppb	Outflow Pconc ppb	Pond Efficiency %	Minimum Depth
CENTRAL DISTRICT							
CP-1	IV	34	--	--	--	--	--
CP-2	IV	30	--	--	--	--	--
CP-3	IV	14	--	--	--	--	--
CP-4	IV	32	--	--	--	--	--
CP-5	IV	11	--	--	--	--	--
CP-6	IV	9	--	--	--	--	--
CP-7	IV	25	--	--	--	--	--
CP-8	IV	21	--	--	--	--	--
CP-9	IV	21	--	--	--	--	--
CP-10	IV	108	6.2	--	190.0	58	2.5
CP-11	V	49	--	--	--	--	--
CP-12	IV/V	104	--	--	--	--	--
CP-13	I	68	10.5	190.0	170.0	66	3.0
CP-14	*III/V	71	*	161.0	*	*	*
CP-15	III	28	5.1	--	190*	60	3.0

* See information in district description

Pond #	Class	Tributary Area Direct	Wet Volume Ac.	Upstream Pconc ppb	Outflow Pconc ppb	Pond Efficiency %	Minimum Depth
WEST DISTRICT							
WP-1	V	41	--	--	--	--	
WP-2	V	38	--	--	--	--	