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* DETERMINATION OF RUNOFF FOR URBAN STORM WATER DRAINAGE SYSTEM DESIGN

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

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THE TECHNICAL RECORD

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by Kurt W. Bauer, Executive Director, SEWRPC

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DETERMINATION OF RUNOFF FOR URBAN STORM WATER DRAINAGE SYSTEM DESIGN

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INTRODUCTION

Urban storm water drainage systems are among the most expensive of public works, requiring large public expenditures for continuing operation and maintenance, as well as for initial construction. Urban storm water drainage systems, moreover, directly affect the public health, safety, and welfare. Improperly designed storm water drainage facilities may not only result in great economic loss, due to the possible damage of both public and private property through flooding, but may result in grave hazard to human health and safety. The design of these systems, therefore, warrants the most careful attention of the municipal engineer.

One of the more difficult problems encountered by the municipal engineer in the design of urban storm water drainage systems is the determination of storm water runoff; that is, determination of the quantity of water, or "hydraulic loading," which must be carried by the drainage system. The amount of storm water runoff, although a critical factor in the successful design of the drainage system, is not susceptible to precise determination and, therefore, calls for the exercise of great judgment on the part of the design engineer. Various methods of calculating storm water runoff have been devised. Application of the various methods, however, may lead to quite different values for the amount of water to be carried by the drainage system under design. Application of only one method of calculation may even produce quite different results because of the varying design criteria possible.

Wherever storm water drainage problems transcend municipal boundary lines and more than one agency of government becomes involved, the methods and criteria to be used in storm water drainage system design must be agreed upon by all parties concerned. Only if such agreement is achieved, and common design methods and criteria are used, can system and facility plans be evolved which are amenable to cooperative adoption and joint implementation. The adoption of common design methods and criteria for storm water drainage system design becomes particularly important in the preparation of comprehensive watershed plans wherein the storm water drainage proposals of a considerable number of local units of government must be related to the receiving major drainage ways and stream channels on a common, areawide basis. A study of design criteria for urban storm water drainage systems was, therefore, incorporated in the first comprehensive watershed planning program undertaken by the SEWRPC.[1] While this study was prepared as a part of the Root River water-

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[1] *The first comprehensive watershed planning program undertaken by the SEWRPC is for the Root River basin. This study is the joint effort of the SEWRPC staff and Harza Engineering Company, Chicago, Illinois. Harza is responsible to the Commission for the performance of all of the hydrologic and hydraulic investigations necessary to the preparation of the comprehensive watershed plan for this basin, including basinwide proposals for land and water use, as well as specific proposals for necessary water control facilities.*

shed planning program, it is believed that its findings with respect to urban storm water drainage system design criteria will be of interest and use to all municipal engineers within the Region.

THE RATIONAL METHOD

One of the more common design methods used in the calculation of storm water runoff is known as the rational method. First introduced in the United States in 1889, the method is presently used by most governmental agencies within the Region to calculate the rate of storm water runoff for storm sewer design. The formula used in the rational method recognizes that a direct relationship exists between rainfall and runoff and is expressed as:

$$Q = CiA$$

Where: Q is the maximum rate of storm water runoff, expressed in cfs;

C is a dimensionless coefficient of runoff representing the ratio between the maximum rate of runoff from the area under consideration and the average rate of rainfall on the area during the time of concentration;

i is the average rainfall intensity expressed in inches per hour during the time of concentration; and

A is the drainage area, expressed in acres, tributary to the point in the drainage system under consideration.

The rainfall intensity, i, is taken as the highest average intensity which can be expected to occur for a specified time of duration on the average of once during a selected recurrence interval.[2] The time of duration is ordinarily selected as equal to the time of concentration which is defined as the time required for runoff to flow from the remotest part of the tributary drainage area to the point in the drainage system under consideration.

The rational method has certain inherent limitations; and, generally, its application tends to result in "over design." Partially because of these limitations, intensive research is presently being devoted, both in the United States and in the United Kingdom, to the entire subject of urban storm water runoff; and it is probable that improved storm water drainage design methods will ultimately result from this research. As yet, however, no practical design methods have either been evolved from the research underway or brought to a level of general acceptance by practicing design engineers. For this reason it is recommended that the rational method continue to be used for the determination of storm water runoff for urban storm water drainage design within the Region in the immediate future. The rational method, properly understood and applied, can produce satisfactory results for urban storm

[2] *The recurrence interval, T, is defined as the average interval of time within which the magnitude, y, of an event will be equaled or exceeded once.*

sewer design. It should be stressed, however, that good design practice limits its application to small drainage areas not exceeding five square miles in areal extent. Development of data for application of more accurate hydrograph methods, as in the Root River watershed planning effort, is usually warranted when considering larger drainage areas.

Determination of Parameters

1. Area

   The tributary drainage area is the only element of the rational formula subject to precise determination and may be most efficiently delineated and measured on a good topographic map of the area to be served. It is important to note, however, that the delineation of the tributary drainage area for urban storm sewer design requires not only careful consideration of the natural topography but of existing and proposed street grades as well, since these may significantly alter the natural topography and drainage pattern. The best practice, therefore, dictates that tributary drainage areas be delineated on the basis of a master land subdivision plan encompassing the entire drainage area involved and showing all existing and proposed streets, established and proposed street grades, and existing hypsometry by contours.

   Where a permanent "rural" street cross section, utilizing road ditches to facilitate drainage, as opposed to the standard urban street cross section, utilizing curb and gutter together with storm sewers for drainage, is proposed, it is particularly important that the street grades and drainage system be established and designed within the context of such an areawide subdivision and street grade study. Consideration of existing and proposed street grades becomes far more critical in areas to be developed with a permanent rural street cross section because the economical use of such a section generally dictates that, to the maximum extent possible, storm water be carried in, and disposed of, by means of surface drainage channels.

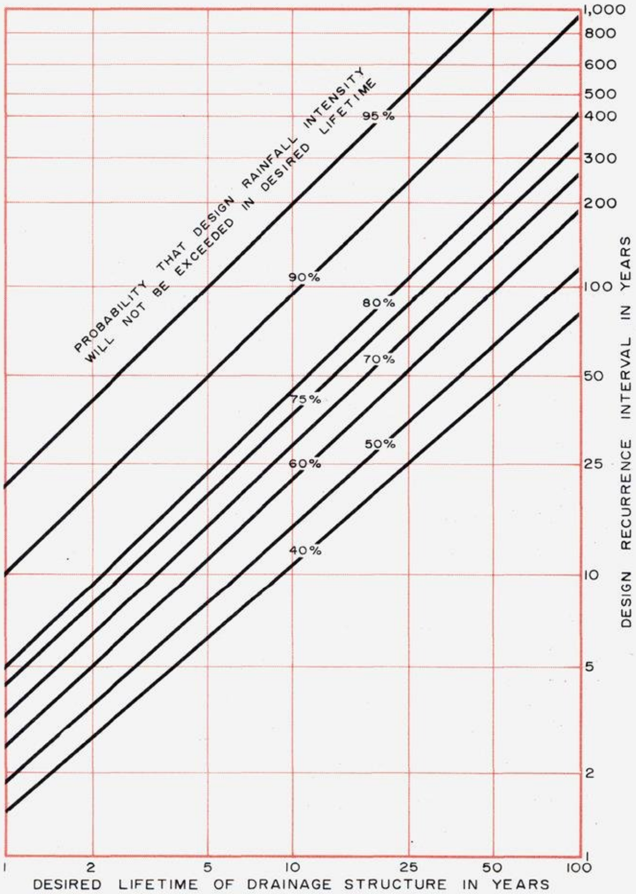
   Topographic maps for the necessary areawide grade and drainage studies, minimally, should have a scale of not less than 200 feet to the inch with a vertical contour interval of 5 feet and, desirably, a scale of not less than 100 feet to the inch with a vertical contour interval of 2 feet. Topographic maps prepared in accordance with SEWRPC Planning Guide No. 2, Official Mapping Guide, and based upon the monumented survey control system outlined in that guide provide ideal base maps for the necessary subdivision, street grade, and drainage studies (see map 1, page 15).

   The total drainage area under consideration must be subdivided into component parts tributary to each point of inlet to the proposed drainage system. This requires a delineation of the geographic location and arrangement of the proposed sewers or drainage ways and their inlet points in relation to the existing and proposed street system.

   Three additional items of information must also be obtained about the tributary drainage area and its component parts: 1) land use, both present and probable future; 2) hydrologic characteristics of the soils; and 3) the general degree of slope of the terrain. The first of these items affects both the degree of protection to be provided by the proposed drainage system and the amount of runoff to be carried. The last two of these characteristics affect the amount of runoff.

Figure I

CALCULATED RISK DIAGRAM



2. <u>Rainfall</u>

Any determination of rainfall intensity for urban storm drainage system design involves determination of three factors: 1) average frequency of recurrence, 2) intensity-duration characteristics of the rainfall, and 3) time of concentration.

The average frequency of recurrence used in the design determines the degree of the protection afforded by the drainage system. Its proper selection requires knowledge of probable future as well as of existing land use in the area to be served so that possible flood damages can be considered in relation to system construction and maintenance costs. Economy is always a consideration in design. Both overdesign and underdesign involve excessive costs over a long period of time, and one of the design objectives should be to achieve the lowest annual cost. In the selection of the recurrence interval to be used, it should be recognized that the cost of storm sewers is not directly proportional to the design frequency.

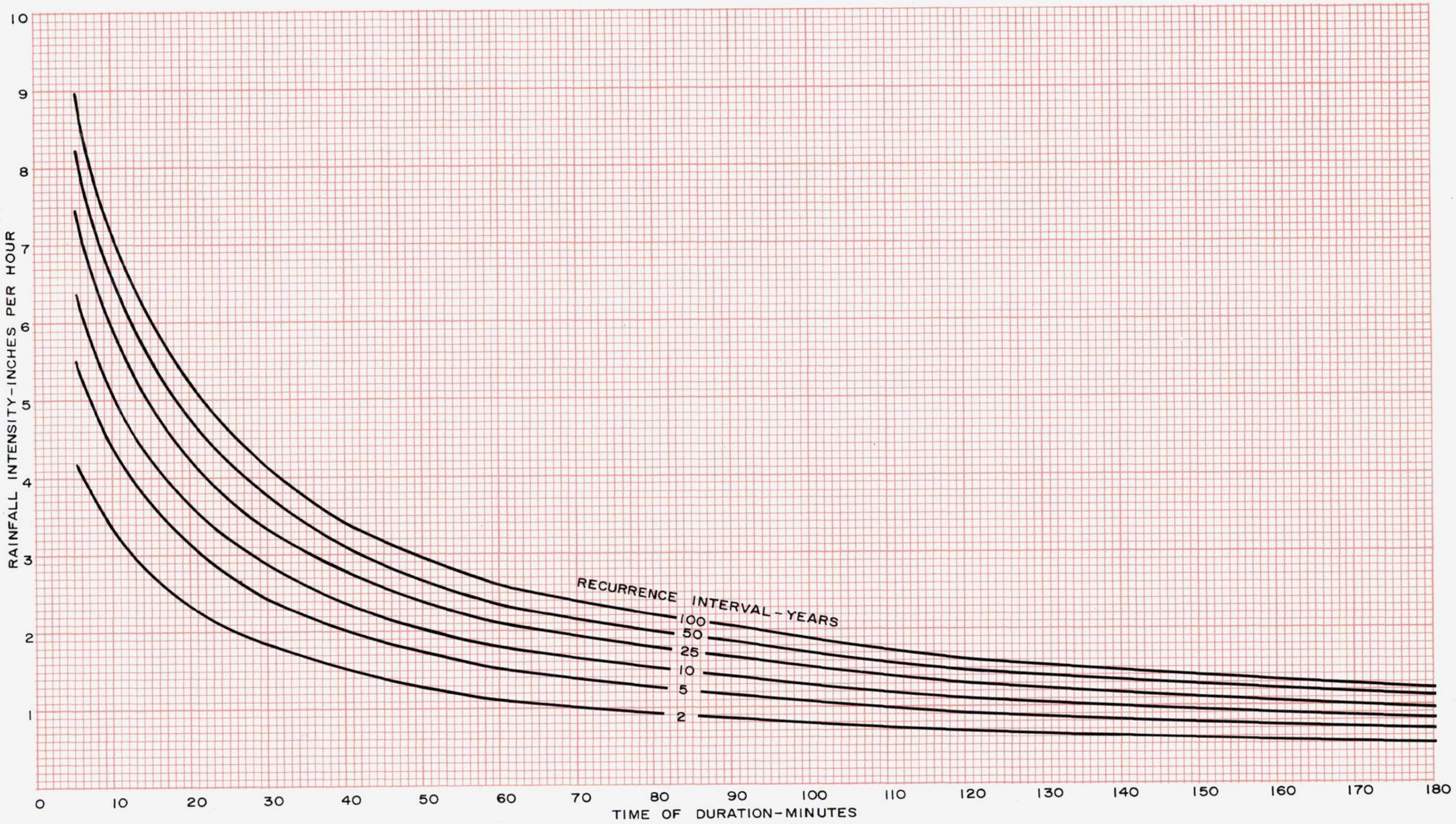
It should also be recognized that an average frequency of recurrence does not imply recurrence at even approximately uniform or constant intervals. Rather, an average frequency of recurrence implies that the given rainfall will probably occur a given number of times over a long period of years, always with the possibility,

Table I

THEORETICAL DISTRIBUTION OF RETURN PERIOD FOR CALCULATION OF RISK OF DESIGN RECURRENCE INTERVAL BEING EQUALED OR EXCEEDED

Average Recurrence Interval	Probability That Interval Between Events Will Not Be Exceeded in Period of N Years								
	1%	5%	10%	25%	50%	75%	90%	95%	99%
100 years	457.00 years	298.00 years	229.00 years	138.00 years	68.90 years	28.60 years	10.50 years	5.10 years	1.00 years
50	228.00	148.00	114.00	68.60	34.30	14.20	5.22	2.54	0.50
25	113.00	73.40	56.30	33.90	16.90	7.04	2.58	1.25	0.25
10	43.70	28.40	21.80	13.20	6.58	2.74	1.00	0.49	0.10
5	20.60	13.40	10.30	6.21	3.11	12.90	4.68	0.23	0.04
2	6.64	4.32	3.32	2.00	1.00	0.42	0.15	0.07	0.02

Figure 2
POINT RAINFALL
INTENSITY - DURATION - FREQUENCY
MILWAUKEE, WISCONSIN
1903 TO 1951



however, of two or more of the given rainfall events occurring within a single year. It should also be noted that the recurrence interval is equal to the reciprocal of the probability of occurrence in any one year. For example, a rainfall of such an intensity that it occurred on an average of once in 100 years would have a recurrence interval of 100 years and a probability or risk of happening in any year of 1 percent or 1 chance in 100. The theoretical distribution of possible actual return periods is illustrated in Table 1 and in Figure 1, which may be used to calculate the risk of a given recurrence interval being equaled or exceeded over a given period of years.

From this table it can be seen that over a long period of time 25 percent of the intervals between events equal to, or greater than, the 100-year event would have a span equal to, or less than, 29 years, while an equal number would have a span equal to, or in excess of, 138 years. From this table it is also evident that to achieve a 75 percent assurance that the design rainfall will not be equaled or exceeded within the next 29 years, a 100-year recurrence interval rain must be used. The rainfall, however, has an equal risk of occurring in any year or in successive years.

Rainfall intensity-duration-frequency curves for use with the rational method within the Region have been prepared by the SEWRPC from U. S. Weather Bureau data. These curves are shown in Figure 2. Each curve represents the highest average rainfall intensity, expressed in inches per hour, which may be expected to occur during a given duration on the average of once during a given recurrence interval. These curves represent point rainfall and should not be applied to areas larger than 10 square miles in extent. The curves are based on the 48-year rainfall intensity record of the first order weather station maintained at Milwaukee by the U. S. Weather Bureau. This record constitutes the longest intensity record available within the Region.

The curves shown in Figure 2 are based upon an annual-series analysis which considers only the maximum rainfall of each year and ignores other rainfalls of lesser intensity during the year. Some of these lesser rainfalls in one year may exceed the maximum rainfalls of other years. A partial-duration series analysis which considers all of the "excessive" or very intense rainfalls regardless of the number occurring within a particular year may be applied to overcome this limitation, but the differences between the two series are negligible for design purposes when considering recurrence intervals of greater than 10 years. For recurrence intervals of 10 years or less, the partial-duration series analysis is probably more appropriate. To adjust data derived from the curves based on the annual series, as presented in Figure 2, to a partial-duration

Table 2

ADJUSTMENT FACTORS FOR CONVERSION OF ANNUAL SERIES DATA TO PARTIAL-DURATION SERIES DATA

Recurrence Interval (Years)	Adjustment Factor
2	1.13
5	1.04
10	1.01
25 or more	1.00

series basis, the values derived from the curves should be multiplied by the factors set forth in Table 2.

Climatological characteristics relating to intense rainfall indicate that the curves presented should be applicable throughout the Region. Comparison of rainfall intensity-duration-frequency curves for the Milwaukee, Madison, and Chicago Weather Stations reveal negligible differences in point rainfall, as shown in Table 3. Since the Milwaukee Station is reasonably well centered within the Region with respect to latitude, and since differences between the Milwaukee and Madison Station records, which represent different longitudes, appear negligible, it appears reasonable to conclude that the Milwaukee data are the best available representation of the Region. U. S. Weather Bureau isohyetal maps indicate a general isohyetal pattern having lines running in a generally northwesterly-southeasterly direction through the Region with higher rainfalls to the southwest. Therefore, the potential rainfall in Walworth County might be considered to be slightly higher than that of Ozaukee County. The amount and variation is extremely small, however, and would not appear to warrant the application of individual county criteria.

Table 3

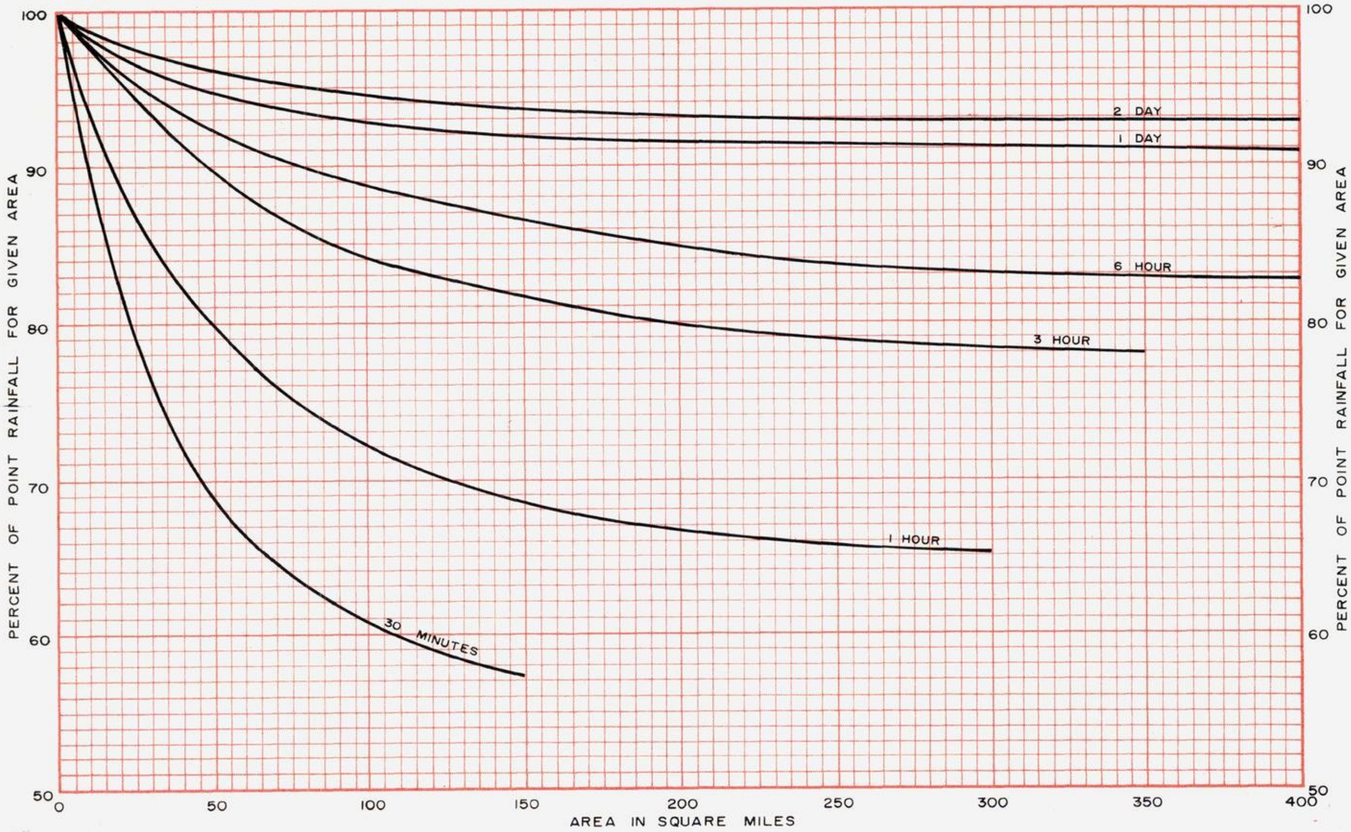
COMPARISON OF CHICAGO, MILWAUKEE, AND MADISON U. S. WEATHER BUREAU RAINFALL INTENSITY-DURATION-FREQUENCY DATA

Recurrence Interval	City	Time						
		5 min.	15 min.	30 min.	1 hour	3 hours	12 hours	24 hours
100-year	Chicago	9.6	6.6	4.6	3.0	1.4	0.42	0.22
	Milwaukee	9.0	6.0	4.2	2.8	1.2	0.40	0.23
	Madison	8.2	6.1	4.4	3.0	1.4	0.46	0.25
50-year	Chicago	8.9	6.0	4.2	2.7	1.3	0.37	0.20
	Milwaukee	8.2	5.5	3.7	2.3	1.1	0.36	0.22
	Madison	7.5	5.5	4.0	2.7	1.2	0.41	0.22
25-year	Chicago	8.0	5.3	3.7	2.4	1.1	0.34	0.18
	Milwaukee	7.5	4.8	3.3	2.1	0.92	0.33	0.20
	Madison	6.8	5.0	3.7	2.4	0.98	0.37	0.21
10-year	Chicago	7.0	4.6	3.2	2.1	0.92	0.28	0.16
	Milwaukee	6.4	4.2	2.8	1.9	0.80	0.27	0.16
	Madison	6.0	4.3	3.0	2.0	0.91	0.30	0.17
5-year	Chicago	6.0	4.0	2.7	1.7	0.78	0.25	0.13
	Milwaukee	5.6	3.6	2.4	1.5	0.65	0.23	0.13
	Madison	5.3	3.7	2.6	1.7	0.76	0.25	0.15
2-year	Chicago	4.6	3.1	2.1	1.4	0.58	0.18	0.10
	Milwaukee	4.2	2.7	1.8	1.2	0.45	0.17	0.10
	Madison	4.2	2.8	1.8	1.2	0.51	0.18	0.11

If consideration of an area larger than 10 square miles is necessary, the rainfall intensities derived from Figure 2 may be reduced through application of the area reduction curves shown in Figure 3. The average depth of rainfall of a given frequency and duration over a large area may be obtained by multiplying the corresponding point rainfall of that frequency and duration by the percentage indicated for the area and duration in Figure 3.

Figure 3

RAIN FALL DEPTH - AREA RELATIONS



SOURCE: U.S. Weather Bureau Technical Paper No. 40 & 49

The curves presented in Figure 2 represent annual maximum rainfall events. Within the Region, the probability of a very intense rainfall event occurring is greater, however, in the summer months than in the remainder of the year. Variations of rainfall probability with season are, therefore, shown in Figure 4 for selected recurrence intervals. These seasonal curves are presented to permit the design engineer to better analyze the flood risks associated with a given recurrence interval and are not intended to be used to reduce rainfall intensity data derived from Figure 2 for seasonal variations. When applying the seasonal curves, the recurrence interval for a given rainfall depth during a particular month may be interpolated between the curves. For example, a one-hour rain of 1.5 inches may be expected to occur in September on the average of once in about 18 years, while the same rainfall may be expected to occur in July on the average of once in every 10 years. It should be noted that the variation in other factors which affect storm water runoff, such as temperature, frost, and snowmelt, follows different seasonal patterns from rainfall.

The series of rainfall curves presented herein will permit the municipal engineer to readily vary the design frequency or recurrence interval. In this respect less frequent, more intense rainfall may be used for the design of those parts of the system not economically susceptible to future relief or for the design of special water control facilities, such as highway underpass drainage pumping stations, where actual runoff exceeding the design capacity of the facilities could seriously disrupt the operation of an important facility or endanger human health and safety. Similarly, more frequent, less intense rainfall may be used for the design of those parts of the system serving areas not highly susceptible to flood damage or presenting little hazard to human health and safety in order to provide a system commensurate with available funds.

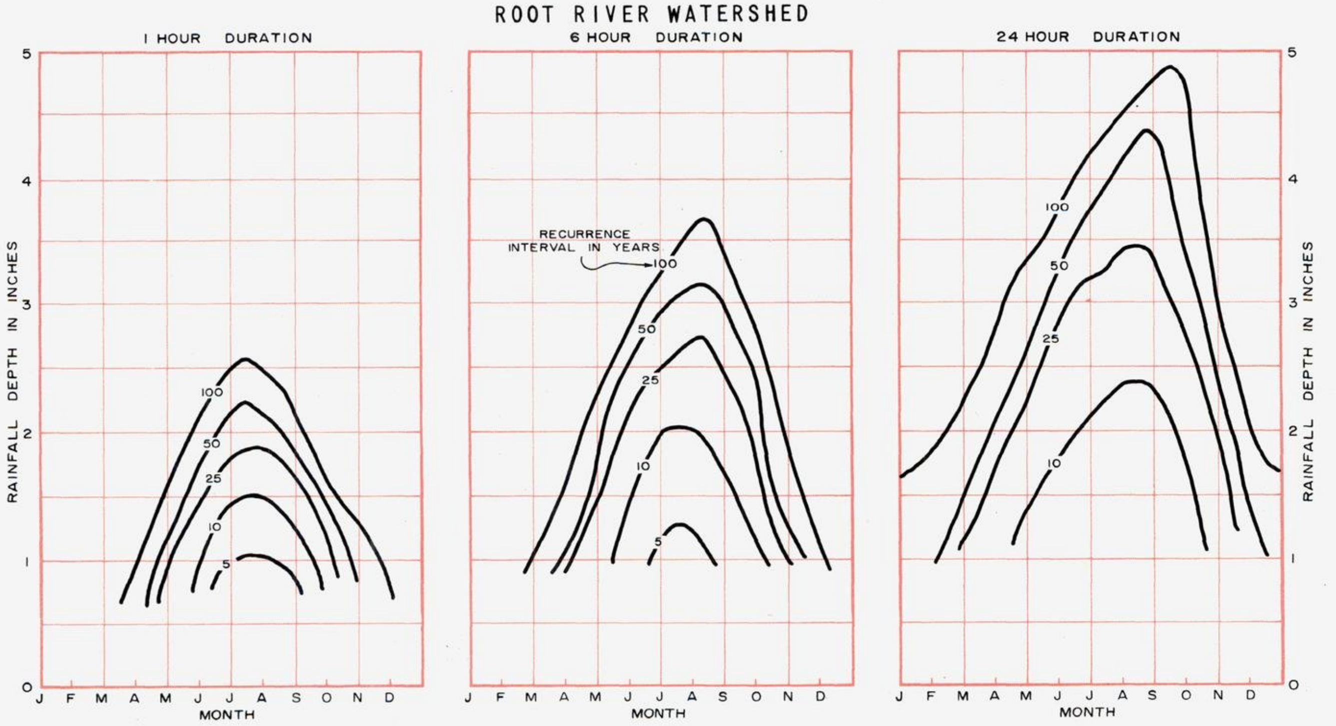
3. Time of Concentration

One of the basic assumptions underlying the rational method is that runoff is a function of the average rainfall rate during the time required for water to flow from the remotest part of the drainage area under consideration to the point under design. In the application of the method, this time of concentration must be estimated in order that the average rainfall rate of a corresponding duration can be determined from the rainfall intensity-duration-frequency curves.

For urban storm sewers, this time of concentration consists of an inlet time, or time required for runoff to flow over the surface to the nearest inlet, and time of flow in the sewer from the uppermost inlet to the point under consideration. The latter time can be closely estimated from the hydraulic properties of the sewer. Inlet time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. In general, the higher the rainfall intensity, the shorter the inlet time. Common practice varies the inlet time from 5 to 30 minutes. It should be noted that the time of concentration has no relationship to the time of beginning of rainfall, being related rather to the position of the peak rainfall intensity. When dealing with pipe systems, the time of concentration may be readily

Figure 4

SEASONAL VARIATION OF RAINFALL FREQUENCY
ROOT RIVER WATERSHED



SOURCE: U.S. Weather Bureau Technical Paper No. 40

calculated from the inlet time plus time of flow in each successive pipe run. The latter value is calculated from the velocity of flow as given by the Manning Formula for hydraulic conditions prevailing in the pipes.

4. Coefficient of Runoff

The coefficient of runoff (C factor) is the variable of the rational method judged least susceptible to precise determination. Its use in the formula implies a fixed ratio for any given drainage area; whereas, in reality its value may vary greatly with seasonal conditions. The coefficient is intended to represent losses between rainfall and runoff due to retention in surface depressions, interception by vegetation, infiltration into the soil, and evaporation and transpiration.

As a part of the Root River watershed planning program, a series of weighted runoff factors related to varying conditions of slope, soil permeability, and land use were prepared for use within the Region. While engineering judgment will always be required in the selection of C values for particular design problems, the values developed in the study and presented herein should not only promote the application of more uniform design criteria throughout the Region but should assist municipal engineers within the Region in applying newly available soil information to the selection of C values.

The approach used to develop the recommended C values differs from older approaches primarily in the emphasis placed upon soils. Because of the prevalence within the Region of low-density residential development having a relatively high proportion of pervious area, the infiltration characteristics of the soils are believed to be a most significant consideration in the selection of composite C values. The SEWRPC has completed, in cooperation with the Soil Conservation Service, U. S. Department of Agriculture, a detailed operational soil survey of the entire seven-county Region. In this soil survey, the various soils of the Region have been mapped in great detail and the physical, chemical, and biological properties of the soils identified and interpreted for planning and engineering purposes. As a part of the soil property interpretations, all of the soils within the Region have been classified into four hydrologic groups, A, B, C, and D, for use in conjunction with the rational method of storm water runoff determination. The A soils group exhibits the highest, and the D soils group the lowest infiltration capacity. Recommended C values corresponding to the four soil groups are shown in Table 7 for varying slope ranges. The soil infiltration capacities as represented by the recommended C values assume "normal" soil moisture conditions and do not consider the possible effects of abnormally high or low antecedent rainfall conditions. Detailed soil maps and interpretive tables linking the mapped soil units to the hydrologic grouping are available from the SEWRPC.

The detailed soil maps available from the SEWRPC also provide information on the general range of ground slope which, together with the four hydrologic soil types, provide one entrance to the matrix of recommended C values set forth in Table 8. The second entrance to the matrix is by land use. The land use categories utilized are those adopted for the regional land use plan presently under preparation by the SEWRPC and are generally consistent with, or readily adaptable to, local land use plans in existence within the Region. Three residential density classifications are provided having net and gross density value ranges as shown in Table 4.

Table 4

RECOMMENDED RESIDENTIAL DENSITY CLASSIFICATIONS

Residential Density Classification	Net Lot Area Per Dwelling Unit	No. of Dwelling Units Per Net[a] Residential Acre	No. of Persons Per Net[a] Residential Acre	No. of Persons Per Gross[b] Square Mile
Low	20,000 sq. ft. and over	0.2 - 2.2	0.5 - 7.2	350 - 3,499
Medium	6,000 - 19,999 sq. ft.	2.3 - 6.9	7.3 - 22.8	3,500 - 9,999
High	Under 6,000 sq. ft.	7.0 - 17.9	22.9 - 59.2	10,000 - 25,000

[a] *A net residential acre includes only land actually devoted to residential use; that is, land within the 'site' boundaries including the building ground area coverage together with the necessary 'on-site' yards and open spaces.*

[b] *A gross residential square mile includes the net area devoted to residential use plus the supporting land uses, such as streets, parks, schools, churches, and neighborhood shopping centers.*

The approximate percentages of impervious and pervious areas recommended to be used in the determination of C values for each land use category are set forth in Table 5.

Table 5

APPROXIMATE PERCENTAGES OF IMPERVIOUS AND PERVIOUS AREAS FOR VARIOUS SELECTED LAND USE CATEGORIES

Land Use	Percent Impervious	Percent Pervious
Industrial	90	10
Commercial	95	5
High Density Residential	60	40
Medium Density Residential	30	70
Low Density Residential	15	85
Agricultural	5	95
Open Space	2	98
Freeways and Expressways	70	30

Recommended C values for various types of impervious surfaces are set forth in Table 6. Recommended C values for lawns and other unpaved and pervious areas are set forth in Table 7, by hydrologic soil group and slope range.

The C values recommended in Table 7 are presented as ranges because retention capacity expressed as a proportion of rainfall will vary with the total volume of rainfall. Retention by a given surface will form a higher proportion of a low-intensity rainfall than of a high; therefore, the lower C values in each range should be used in conjunction with the relatively lower rainfall intensities associated with 2- to 10-year design recurrence intervals. The higher values in each range should be used with the higher rainfall intensities associated with longer 25- to 100-year design recurrence intervals. Higher C values should also be used for very large drainage basins having long times of concentration and, therefore, longer design rainfall durations.

Table 6

RECOMMENDED COEFFICIENT OF RUNOFF VALUES FOR VARIOUS SELECTED IMPERVIOUS SURFACES

Surface	Runoff Coefficient
Streets:	
Asphaltic.	0.07 - 0.95
Concrete	0.80 - 0.95
Drives and Walks. . .	0.75 - 0.85
Roofs	0.75 - 0.95

Recommended weighted values of the coefficient of runoff, C, for composite land use, slope, and soil conditions are presented in Table 8. The percentage of impervious area used to calculate the weighted values for each representative major land use are based on representative present land use conditions as set forth in Table 5. Coefficients of runoff for the impervious portions are based on averaged values presented in Table 6 and for pervious areas in Table 7. The values in Table 8 are presented as ranges in order to allow the design engineer to exercise judgment in consideration of the effect of variation in the duration and intensity of the design rainfall. As already noted, the lower C values in each range should be used with the relatively low intensities associated with shorter design recurrence intervals, while the high values should be used for the relatively heavier rainfall intensities associated with longer design occurrence intervals. Similarly, the higher C value should be used for very large drainage basins having long times of concentration and, therefore, longer design rainfall durations. Coefficient of runoff curves based upon the approximate midpoints of the values presented in Table 8 are shown in Figures 5 through 8 for each hydrologic soil group.

Table 7

RECOMMENDED COEFFICIENT OF RUNOFF VALUES FOR PERVIOUS SURFACES BY SELECTED HYDROLOGIC SOIL GROUPINGS AND SLOPE RANGES

Slope		Runoff Coefficient			
		A Soils	B Soils	C Soils	D Soils
Flat	0 - 2%	0.04 - 0.09	0.07 - 0.12	0.11 - 0.16	0.15 - 0.20
Average	2 - 6%	0.09 - 0.14	0.12 - 0.17	0.16 - 0.21	0.20 - 0.25
Steep	Over 6%	0.13 - 0.18	0.18 - 0.24	0.23 - 0.31	0.28 - 0.38

APPLICATION OF RATIONAL METHOD

From the foregoing presentation, it is apparent that application of the rational method to a design problem requires determination of the following basic data:

1. Drainage area tributary to point under design.

2. Existing and probable future land use in the drainage area.

3. Soil and slope characteristics of the drainage area.

4. Rainfall intensity-duration-frequency data for the locality.

5. Time of concentration.

6. Tentative arrangement of the proposed drainage system and location of inlets to permit division of the whole drainage area into the component parts tributary to each section of the system.

In order to illustrate the application of the method with the design criteria recommended herein, the following example is provided:

It is desired to prepare a general storm sewer system plan for a presently undeveloped portion of a rapidly urbanizing community. The basic data inputs available include:

1. Topographic map prepared to National Map Accuracy Standards having a horizontal scale of 1" = 100' and a vertical contour interval of 2 feet (see Map 1).

2. Official Map setting forth existing and proposed street layout and land subdivision based upon existing and proposed land use (see Map 2).

3. Detailed soils map (see Figure 9).

4. The storm sewer design criteria presented herein.

Table 8

WEIGHTED RUNOFF COEFFICIENTS FOR USE IN THE RATIONAL FORMULA

Hydrologic Soil Group	A			B			C			D		
Slope Range	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +
LAND USE												
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
High Density	0.47	0.49	0.50	0.48	0.50	0.52	0.49	0.51	0.54	0.51	0.53	0.56
Residential	0.58	0.60	0.61	0.59	0.61	0.64	0.60	0.62	0.66	0.62	0.64	0.69
Medium Density	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
Residential	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Low Density	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.28	0.35
Residential	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Agricultural	0.08	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Open Space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Freeways and	0.57	0.59	0.60	0.58	0.60	0.61	0.59	0.61	0.63	0.60	0.62	0.64
Expressways	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78

Figure 5

COEFFICIENT OF RUNOFF CURVES
FOR HYDROLOGIC SOIL GROUP "A"

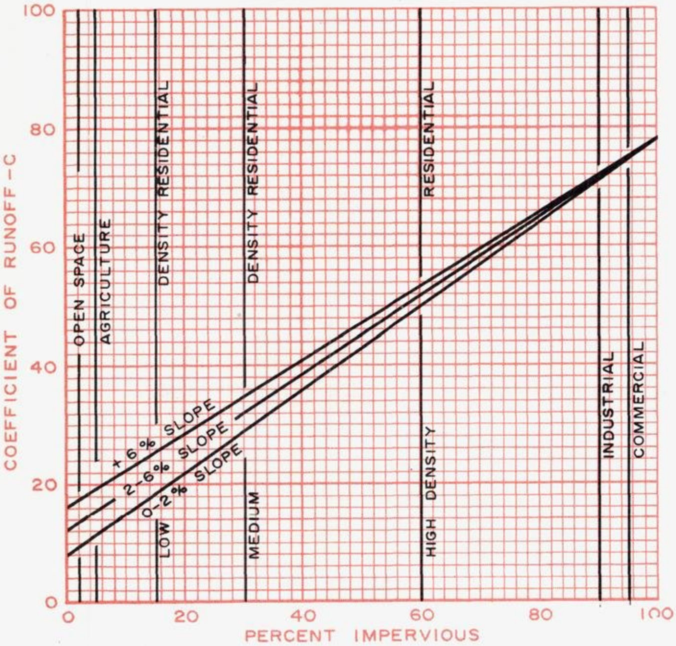
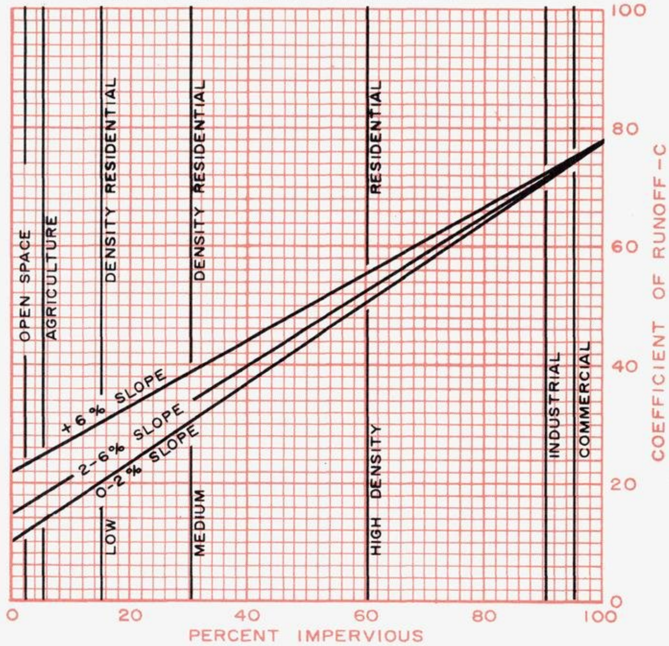


Figure 6

COEFFICIENT OF RUNOFF CURVES
FOR HYDROLOGIC SOIL GROUP "B"



Utilizing this information, proposed established street grades are developed, the total tributary drainage area to be considered is delineated, a proposed storm sewer system is delineated in relation to the existing and proposed street system, and the total tributary drainage area divided into component parts tributary to each point of inlet to the proposed storm sewer system, as shown in Figure 10.

Figure 7

COEFFICIENT OF RUNOFF CURVES
FOR HYDROLOGIC SOIL GROUP "C"

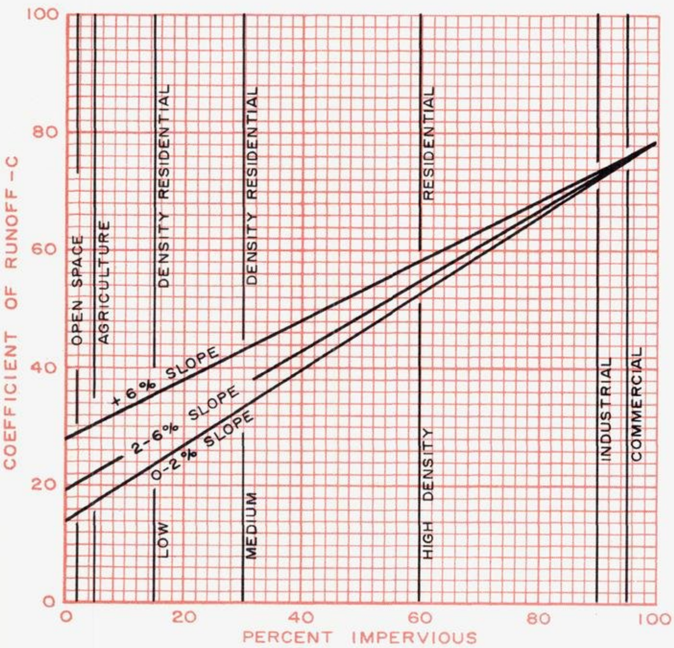
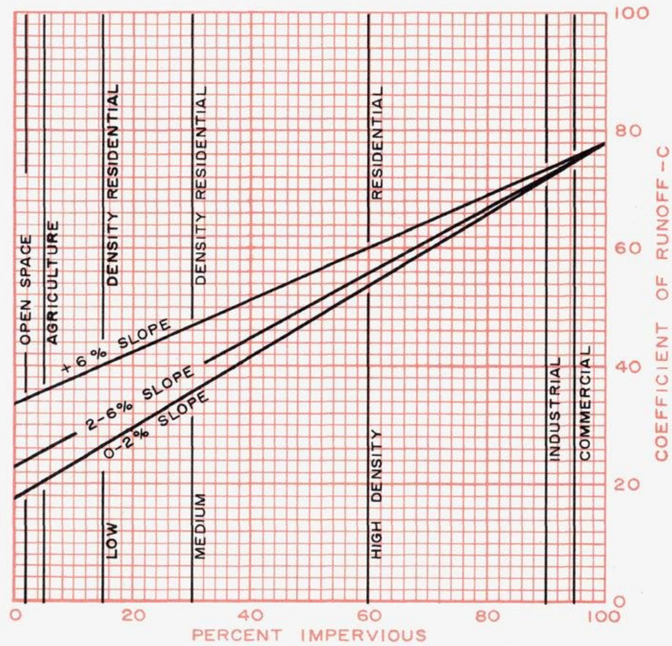


Figure 8

COEFFICIENT OF RUNOFF CURVES
FOR HYDROLOGIC SOIL GROUP "D"

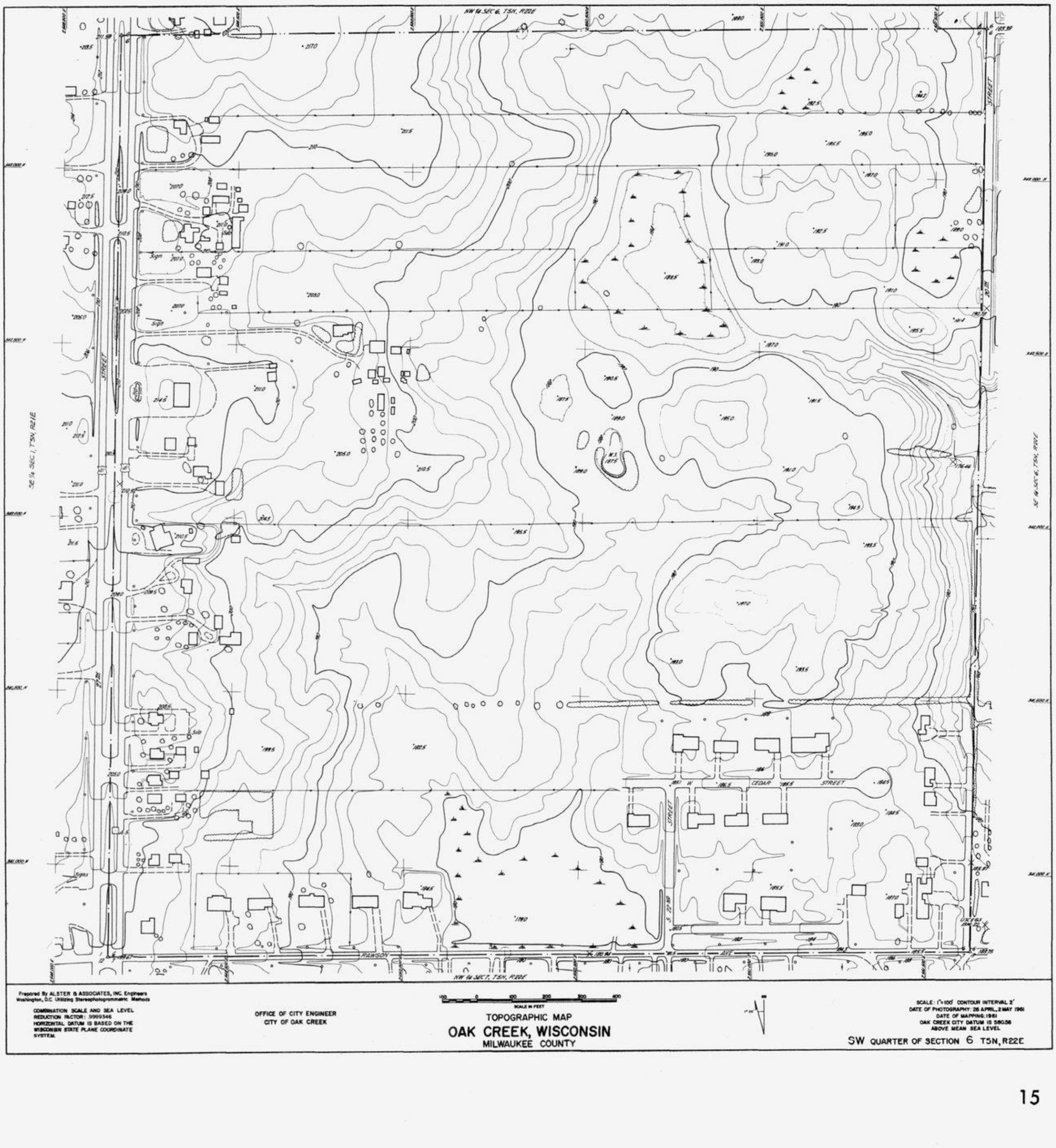


Utilizing the detailed soils map, interpretative maps based upon the hydrologic soil groups (see Figure 11) and upon slopes (see Figure 12) are also prepared. The interpretative maps indicate that virtually all of the soils within the tributary drainage area fall into hydrologic group C with slopes ranging from 2 to 6 percent. Utilizing the hydrologic soil and slope interpretations, representative C values are selected from Table 8 for the proposed land uses; that is, medium density residential: C equals 0.33; commercial: C equals 0.72.

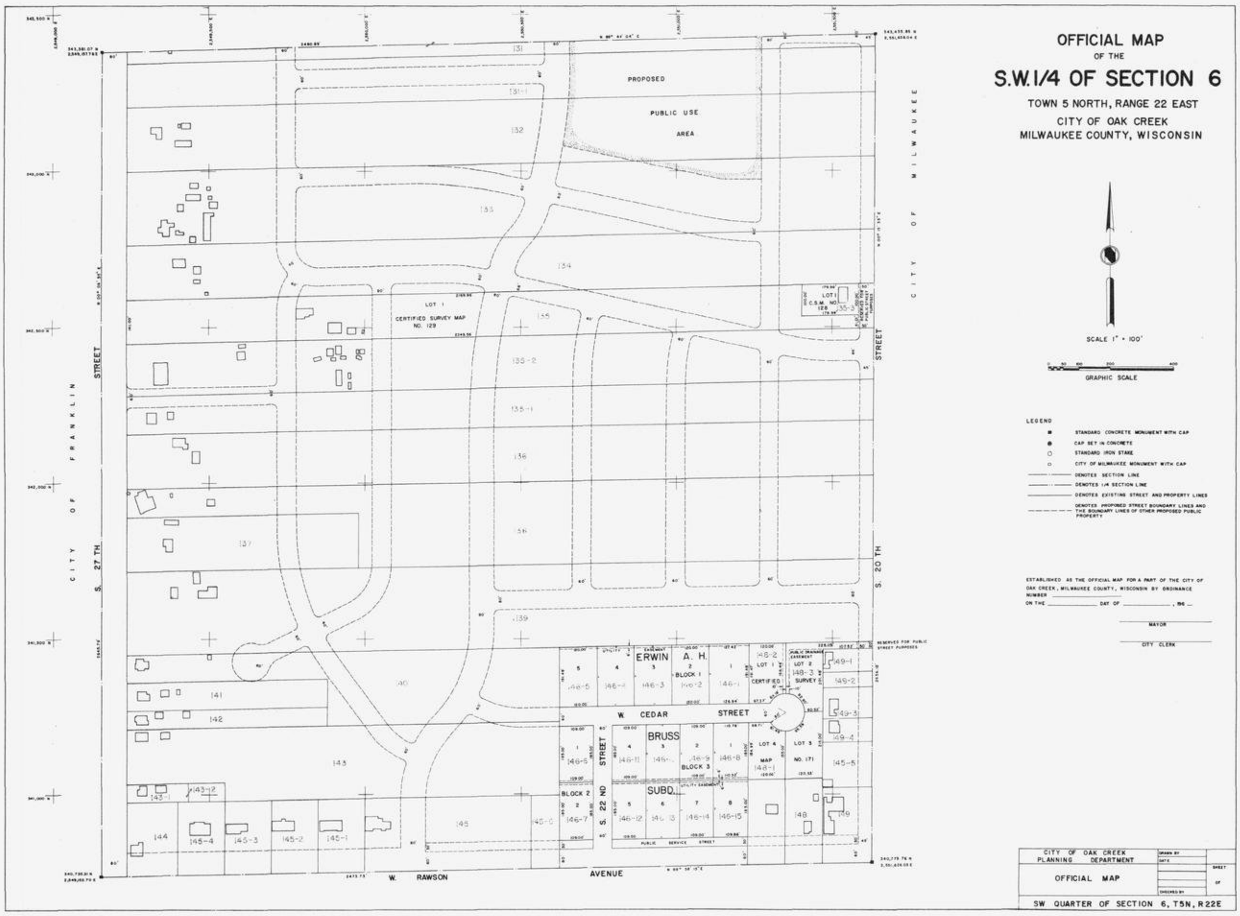
The design recurrence interval selected is 10 years, and the initial inlet time selected is 15 minutes. The design computations are then carried out as shown in Table 9, utilizing the rainfall intensity-duration-frequency curve presented in Figure 2 to determine i, measuring A directly on the design map, and calculating pipe capacity and

Map I

TOPOGRAPHIC MAP

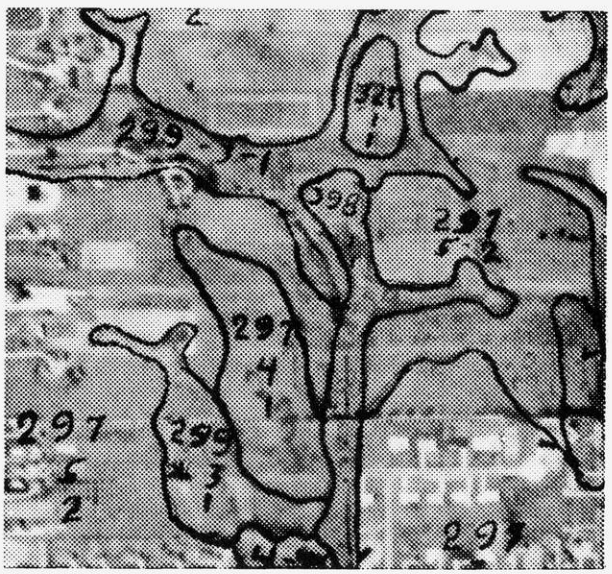


Map 2
OFFICIAL MAP



flow parameters utilizing the Manning Formula with n equal to 0.013. The resulting pipe sizes, grades, and elevations are noted in Figure 10.

Figure 9
DETAILED SOILS MAP



LEGEND

299 SOIL TYPE
6 PERCENT OF SLOPE
2 AMOUNT OF EROSION

CONCLUSION

The design criteria presented herein should be of interest and use to all municipal engineers within the Region engaged in the planning and design of urban drainage systems. These criteria are believed to represent the best present engineering practice and should serve to assist the design engineer in the difficult task of determining

Table 9

STORM SEWER DESIGN COMPUTATION

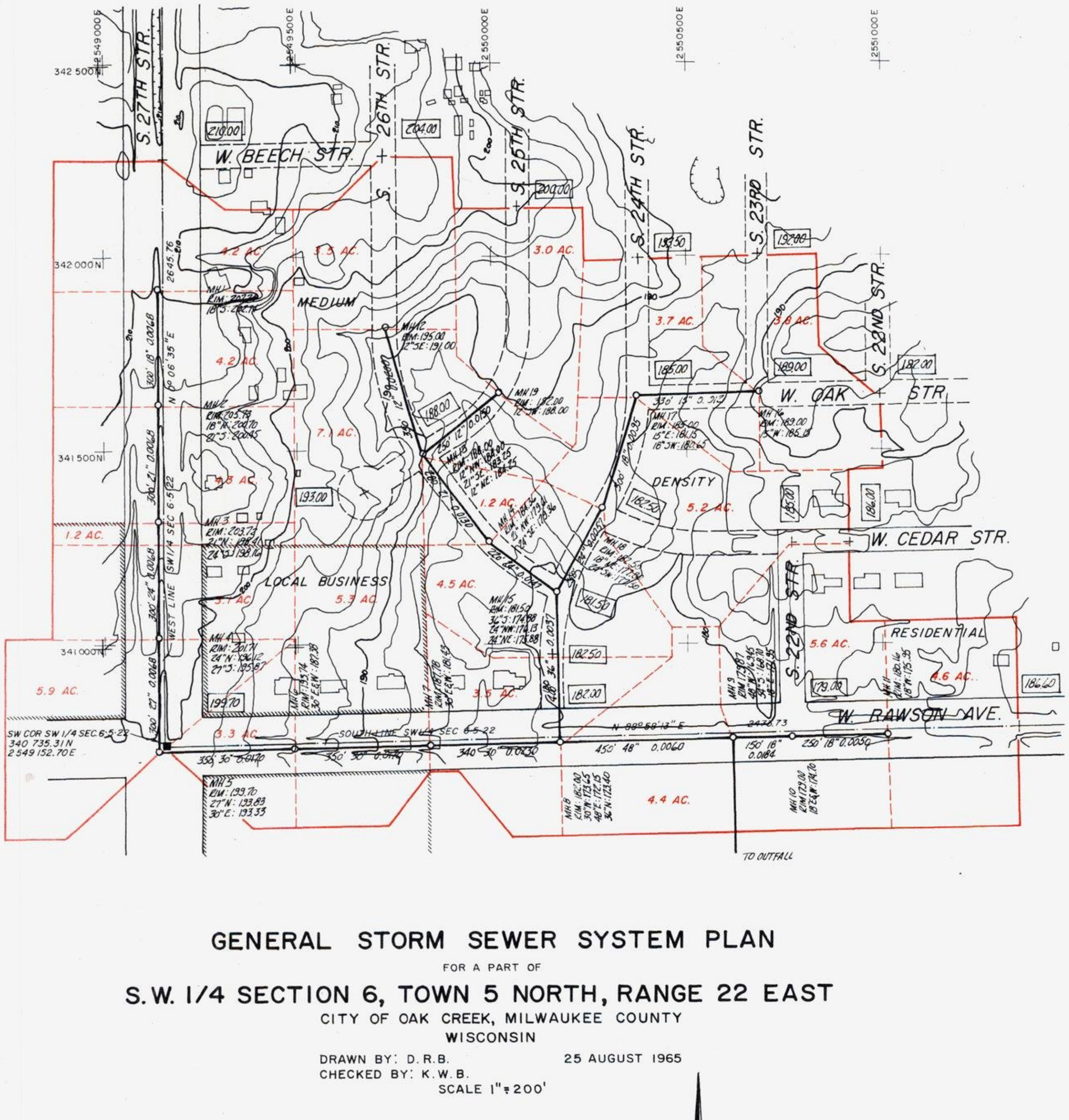
COMPUTED BY: ____________ DATE: ____________

LOCATION: SW 1/4 Sec. - 6 - 5 - 22
City of Oak Creek, Milwaukee County, Wisconsin

CHECKED BY: ____________ DATE: ____________

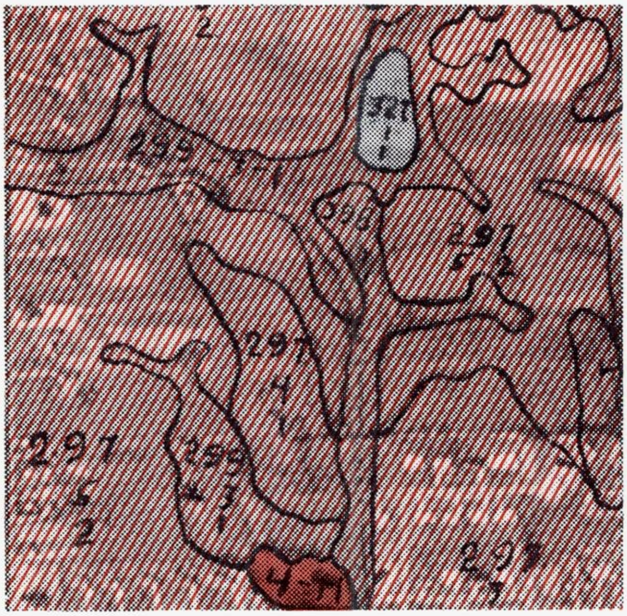
In	From M.H. No.	To M.H. No.	Length (Ft.)	Incremental Area (Acres)	A (Acres)	Time Of Concentration (Minutes)	Time In Sewer (Minutes)	i (In./Hr.)	C	Q (CFS)	Size (In.)	Slope (Ft./Ft.)	Capacity (CFS)	Velocity (FPS)	Depth of Flow (Ft.)	Elev. Of Hydraulic Grd. Line		Invert Elev.		Rim Elev.	
																Upper	Lower	Upper	Lower	Upper	Lower
S. 27th St.	1	2	300	4.2R	4.2R	15.0	0.0	4.2	0.33	5.8	18	0.0068	8.5	5.2	0.90	203.64	201.60	202.74	200.70	207.74	205.73
S. 27th St.	2	3	300	4.2R	8.4R	16.0	1.0	4.1	0.33	11.4	21	0.0068	13.1	6.4	1.24	201.94	199.65	200.45	198.41	205.73	203.72
S. 27th St.	3	4	300	4.3R	12.7R	16.8	0.8	4.0	0.33	16.8	24	0.0068	18.3	6.9	1.48	199.64	197.60	198.16	196.12	203.72	201.71
S. 27th St.	4	5	300	1.2R	13.9R	17.5	0.7	3.9	0.33	17.8											
				3.1C	3.1C			3.9	0.72	8.7											
										26.5	27	0.0068	26.5	6.5	2.25	198.12	196.08	195.87	193.83	201.71	199.70
W. Rawson Ave.	5	6	350	0.0R	13.9R	18.3	0.8	3.8	0.33	17.4											
				5.9C	9.0C			3.8	0.72	24.6											
										42.0	30	0.0170	53.0	12.1	1.62	194.95	189.00	193.33	187.38	199.70	193.74
W. Rawson Ave.	6	7	350	0.0R	13.9R	18.8	0.5	3.8	0.33	17.4											
				3.3C	12.3C			3.8	0.72	33.6											
										51.0	30	0.0170	53.0	12.3	1.80	189.18	183.23	187.38	181.43	193.74	187.78
W. Rawson Ave.	7	8	340	0.0R	13.9R	19.3	0.5	3.7	0.33	17.0											
				5.3C	17.6C			3.7	0.72	46.8											
										63.8	30	0.0230	64.0	13.8	2.50	183.93	176.15	181.43	173.65	187.78	182.00
W. Rawson Ave.	8	9	450	35.5R	49.4R	19.7	0.4	3.6	0.33	58.5											
				0.0C	17.6C			3.6	0.72	45.6											
										104.1	48	0.0060	104.1	10.0	4.00	176.15	173.45	172.15	169.45	182.00	179.87
W. Rawson Ave.	9	Outfall	--	14.6R	64.0R	20.6	0.9	3.5	0.33	73.8											
				0.0C	17.6C			3.5	0.72	44.3											
										118.1	54	0.0033	120.0	8.8	3.60	172.30	--	168.70	--	179.87	--
W. Rawson Ave.	11	10	250	4.6R	4.6R	15.0	0.0	4.2	0.33	6.4	18	0.0050	7.3	4.8	1.08	177.03	175.78	175.95	174.70	180.16	179.00
W. Rawson Ave.	10	9	150	5.6R	10.2R	15.9	0.9	4.1	0.33	13.8	18	0.0184	8.1	10.0	1.50	176.20	173.45	174.70	171.95	179.00	179.87
S. 26th St.	12	13	350	3.5R	3.5R	15.0	0.0	4.2	0.33	4.9	12	0.0200	4.9	6.1	1.00	192.00	185.00	191.00	184.00	195.00	188.00
S. 26th St.	13	14	280	10.1R	13.6R	16.0	1.0	4.1	0.33	18.4	21	0.0130	18.4	7.6	1.50	184.75	180.11	183.25	179.61	188.00	184.36
S. 26th St.	14	15	220	1.2R	14.8R	16.6	0.6	4.0	0.33	19.6	24	0.0147	27.2	9.7	1.26	180.62	177.39	179.36	176.13	184.36	181.50
S. 24th St.	15	8	400	17.2R	32.0R	17.0	0.4	4.0	0.33	42.2	36	0.0037	42.2	5.9	3.00	177.88	176.40	174.88	173.40	181.50	182.00
W. Oak St.	16	17	330	3.8R	3.8R	15.0	0.0	4.2	0.33	5.3	15	0.0121	7.0	6.3	0.79	185.94	181.94	185.15	181.15	189.00	185.00
S. 24th St.	17	18	300	3.7R	7.5R	15.9	0.9	4.1	0.33	10.1	18	0.0095	10.1	5.8	1.50	182.15	179.30	180.65	177.80	185.00	182.50
S. 24th St.	18	15	250	5.2R	12.7R	16.8	0.8	4.0	0.33	16.7	24	0.0057	16.7	5.5	2.00	179.30	177.88	177.30	175.88	182.50	181.50
S. 25th St.	19	13	250	3.0R	3.0R	15.0	0.0	4.2	0.33	4.2	12	0.0150	4.2	5.5	1.00	189.00	185.25	188.00	184.25	192.00	188.00

Figure 10
GENERAL STORM SEWER SYSTEM PLAN



storm water runoff and in applying newly available soil information to such determination. Finally, the application of these criteria by municipal engineers will serve to promote common storm sewer design methods and criteria within the Region and thereby better permit local storm water drainage proposals to be related to receiving drainage ways and stream channels on a common areawide basis.

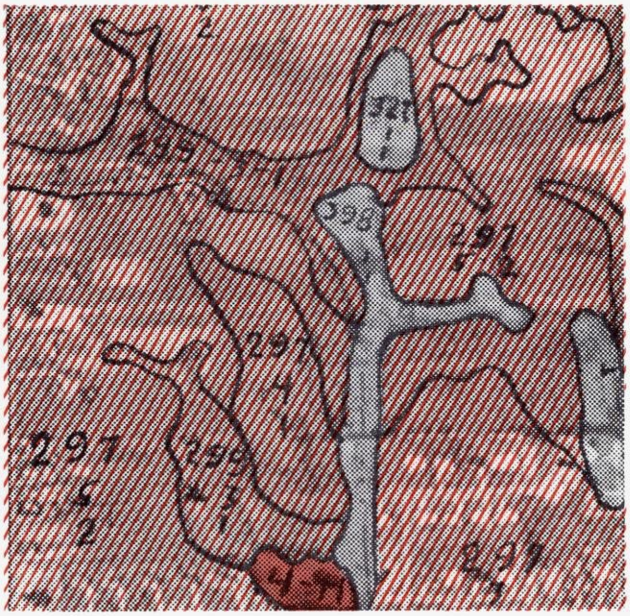
Figure 11
INTERPRETIVE SOILS MAP
HYDROLOGIC SOILS GROUP



LEGEND

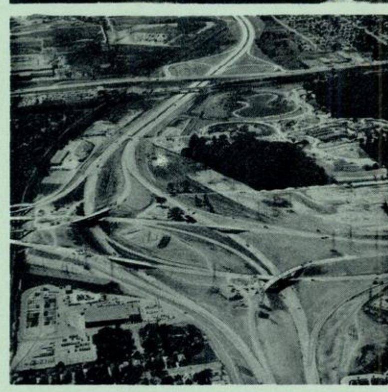
- GROUP B
- GROUP C
- GROUP D

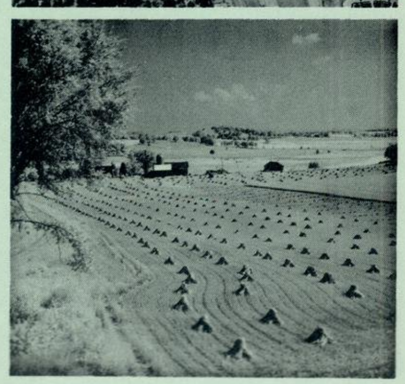
Figure 12
INTERPRETIVE SOILS MAP
SCOPE RANGE



LEGEND

- 0-1 %
- 2-5 %
- 9-11 %

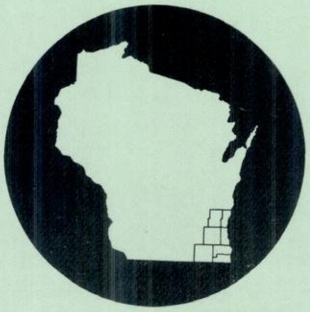




THIS IS SOUTHEASTERN WISCONSIN

Important vital statistics on the Region and percent of totals for the State of Wisconsin.

Land and Water Area (sq. mi.)	2,688	5%
Population (1960)	1,573,620	40%
Resident Employment (1960)	612,723	42%
Resident Unemployment (1960)	24,174	41%
Resident Labor Force (1960)	636,897	42%
Resident Man'f. Employment (1960)	253,292	52%
Resident Non-Man'f. Employment (1960)	359,431	37%
Disposable Personal Income (1960)	$3,572,000,000	46%
Retail Establishments (1958)	15,780	33%
Retail Sales (1960)	$2,045,000,000	42%
Property Value (1960)	$8,726,000,000	46%
Total Shared Tax (1960)	$62,777,000	54%
Total State Aids (1960)	$35,474,000	26%
Total Property Tax Levy	$239,380,000	50%
Total Long Term Public Debt	$378,592,000	55%
Total Highway (miles) (1960)	8,740.45	8.9%
Value of Mineral & Non-Metal Production (1961)	$15,494,487	20.08%
Total Vehicle Registration (1962-1963)	633,540	36.8%
Auto Vehicle Registration (1962-1963)	551,188	40%
Truck Registration (1962-1963)	55,950	23%
State Parks & Forest Areas (acres) (1963)	12,546	3.02%



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