EIS-75-3833D

METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

O'HARE SERVICE AREA WASTEWATER CONVEYANCE SYSTEM

DRAFT ENVIRONMENTAL IMPACT STATEMENT

MARCH 1975

Prepared by:
US ENVIRONMENTAL PROTECTION AGENCY
REGION V
Chicago, Illinois

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FOR THE

METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

DES PLAINES - O'HARE CONVEYANCE SYSTEM

PREPARED BY

THE UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

REGION V

CHICAGO, ILLINOIS

MARCH, 1975

SUMMARY SHEET

1. Name of Action (Check one)

   Administrative action (X)
   Legislative action ( )

2. Brief description of action indicating what states (and counties) are particularly affected.

   The proposed projects consist of a system of conveyance tunnels known as Upper Des Plaines Intercepting Sewers 20, 20A, 20B, 20C and 21, and drop shafts to intercept and convey wastewater from a 58.2 sq. mile service area in the Northwest region of the Metropolitan Sanitary District of Greater Chicago to the proposed O'Hare Water Reclamation Plant. (A separate EIS has been prepared on the O'Hare Water Reclamation Plant). Upper Des Plaines Intercepting Sewers 20, 20A, and 21 will also intercept and convey flows from combined sewer outfalls presently discharging to Weller's Creek and Feehanville Ditch and will provide partial storage of the combined wastewater for later treatment at the proposed O'Hare Water Reclamation Plant.

   The O'Hare MSDGC service area consists of all or part of the following communities within Cook County: Arlington Heights, Buffalo Grove, Des Plaines, Elk Grove, Mount Prospect, Prospect Heights, Rolling Meadows, and Wheeling, Illinois.

3. Summary of Environmental Impact and adverse environmental effects.

   A. Short Term Impacts

      1) Construction

         a) Blasting

Construction of the drop shaft access manholes and parts of some tunnels will require blasting. To minimize the impacts of blasting particle velocities will be restricted by matting and explosive charge selection to values that prevent any physical damage to surface structures and appropriate screening of dust particles will be required.

b) Noise and Vibration

1) Blasting operations will be restricted to certain hours of the day.

2) Heavy machinery, trucks, and other vehicles will increase ambient noise levels in residential areas.

c) Water Quality and Quantity

1) Dewatering of tunnels will temporarily lower the water table of the shallow aquifer. No effect on local wells is anticipated.

2) Increased siltation in Higgins Creek may occur with dewatering of the tunnels. (Presently, a half hour detention time is being planned to minimize this effect).

d) Air Quality

1) Dust from construction activities at surface sites will be minimized by using hard paved surfaces and dust control measures.

2) Operation of heavy construction equipment powered by internal combustion engines will add to the air pollutant loading. However, it is not anticipated that this would result in a significant temporary change in ambient air quality.

2) <u>Operation Impacts</u>

All conveyance tunnels will be grouted and lined with concrete to minimize infiltration. In general, some slight positive infiltration into the tunnels is planned to prevent possible degradation of the groundwater supplies from exfiltration of combined sewage into the groundwater aquifers.

Based on analyses of storms of record, groundwater levels and design parameters, occasional exfiltration into the groundwater aquifers might occur. A groundwater well monitoring program is planned to discover any problems which may develop.

B. Long Term Impacts

1) Combined sewage overflows to Weller's Creek and Feehanville Ditch will be reduced from approximately 80 to 6 flows a year. This would result in a 92% BOD reduction and 75% flow reduction in combined sewage waste overflows to Weller's Creek and Feehanville Ditch.

2) Relief of existing interceptors (which are presently overloaded during wet weather) will also be provided.

3) No adverse long term impacts are anticipated.

4. Alternatives Considered:

    a) Separation of combined sewers and construction of conventional interceptors.

    b) Collection and conveyance of combined overflows.

    c) Collection, conveyance and storage of combined overflows.

5. Irreversible and Irretrievable Commitment of Resources

    a) Labor and energy expended in construction of the proposed facilities and,

    b) Capital cost of tunnels is not recoverable.

6. The following Federal, State and local agencies are being requested to comment on this Draft Environmental Impact Statement:

Council on Environmental Quality
Department of Agriculture
    Soil Conservation Service
U.S. Army Corps of Engineers
    North Central Division
    Chicago District
Department of Health, Education and Welfare
Department of Housing and Urban Development
Department of the Interior
    Bureau of Outdoor Recreation
    Fish and Wildlife Service
    Geological Survey
Department of Transportation
    Federal Aviation Administration
Energy Research and Development Administration
    Argonne National Laboratory

Governor of Illinois
Illinois Institute for Environmental Quality
Illinois Environmental Protection Agency
Illinois Division of Waterways
Illinois Department of Conservation
Illinois Department of Public Health

Northeastern Illinois Planning Commission
Cook County Department of Environmental Control

Metropolitan Sanitary District of Greater Chicago

City of Des Plaines
Village of Elk Grove
Village of Arlington Heights
Village of Mount Prospect
Village of Palatine
Village of Wheeling

Others

7. Date Draft made available to:

   a) Council on Environmental Quality - March, 1975

   b) Public - March, 1975

Acknowledgement

Portions of this Environmental Impact Statement were taken directly from the Environment Assessment prepared by the MSDGC (November, 1974), and the "Facilities Planning Study - MDSGC Overview Report" and "O'Hare Facility Area" (January, 1975) also prepared by the MSDGC.

TABLE OF CONTENTS

TABLE OF CONTENTS

CHAPTER 1

BACKGROUND

A. Identification of Grant Applicant and Planners

The grant applicant for the proposed conveyance system projects is the Metropolitan Sanitary District of Greater Chicago. The facilities Planning Report for the Metropolitan Sanitary District of Greater Chicago is comprised of eight separate reports. These reports consist of an overview report and individual reports for the seven facility areas.

B. Description of the Proposed Actions

The Upper Des Plaines tunnel conveyance system consists of four major elements. These are: the tunnels, eight drop shafts and one main shaft, seventy access manholes, and nine monitoring wells. These elements will be constructed as five separate projects. A brief description of each is given below.

1. Connections and laterals: Weller's Creek, various locations, Upper Des Plaines 20A (73-318-2S).

   This project consists of constructing 22,000 linear feet of 20 foot diameter tunnel in rock at a depth of 160 feet; five drop shafts; one main construction shaft, access manholes and miscellaneous and appurtenant construction.

2. Connections and laterals: Weller's Creek, various locations, Upper Des Plaines 20A (73-318-2S).

   This project consists of constructing special diversion structures to control and direct flow from existing interceptors and local combined sewer outfalls to the drops shafts and/or tunnels.

3. Earth tunnel: Weller's Creek, Mt. Prospect Road, Princeton Street and Wolf Road, Upper Des Plaines 20B (73-319-2S).

   This project consists of constructing 6,000 linear feet of five-foot diameter earth tunnel at a depth of 60 feet; together with manholes and connecting structures. This sewer will divert sanitary sewage flows in the Upper Des Plaines 14A system from the North Side Plant to the proposed O'Hare Water Reclamation Plant.

4. Rock tunnels and drop shafts: Weller's Creek and Feehanville Ditch, Lonnquist Boulevard and William Street, Upper Des Plaines 21 (73-320-2S).

   This project consists of constructing 11,200 linear feet of 16-foot diameter deep rock level tunnel, 2,000 linear feet of nine foot diameter deep rock level tunnel, three drop shafts, special diversion structures, access manholes and miscellaneous and appurtenant construction.

5. Intercepting sewer: Upper Des Plaines 20C (69-307-2S).

   This project consists of a five foot diameter interceptor from a junction structure at Wildwood Road and Oakton Street East along Oakton Street for approximately 11,000 linear feet at a depth of 40 feet terminating at drop shaft seven of the Upper Des Plaines tunnel conveyance system. The sanitary sewage will flow to the proposed O'Hare Water Reclamation Plant for treatment.

C. <u>General and Specific Location of the Proposed Actions</u>

The Upper Des Plaines Basin covers an area of 58.2 square miles

(37,250 acres) in the northwest portion of the Metropolitan Sanitary District, shown in Figures 1-1 and 1-2. This area is predominantly residential in character. Growth of the area has been spurred by several factors. Among the more significant of these is the proximity of O'Hare Airport, the Northwest Tollway, the Tri-State Tollway, and the Chicago and Northwestern Railway's tracks which bisect the basin in a northwesterly direction.

The area includes the communities of Arlington Heights, Mount Prospect, Prospect Heights, Wheeling, and a part of the City of Des Plaines as well as newer urban developments such as Elk Grove Village, Rolling Meadows and Buffalo Grove. As illustrated in Figure 1-1, the boundaries encompass an area which lies generally West of the Des Plaines River. Several major drainage courses traverse the basin in a generally East-West direction and empty into the Des Plaines River. Two of the waterways are of concern, since they receive combined sewer overflows even during low intensity storms. They are Weller's Creek and Feehanville Ditch. No other waterways within the Upper Des Plaines River Basin receive combined overflows.

D. Water Quality and Water Quantity Problems in the Area

1. Sources of Water Supply in the Service Area

There are three water supply sources to the service area:

a. Groundwater from shallow glacial-till Silurian aquifer. The well records indicate that the majority of wells in the shallow aquifer are private domestic service with pumpout rates between 5 to 50 gpm.

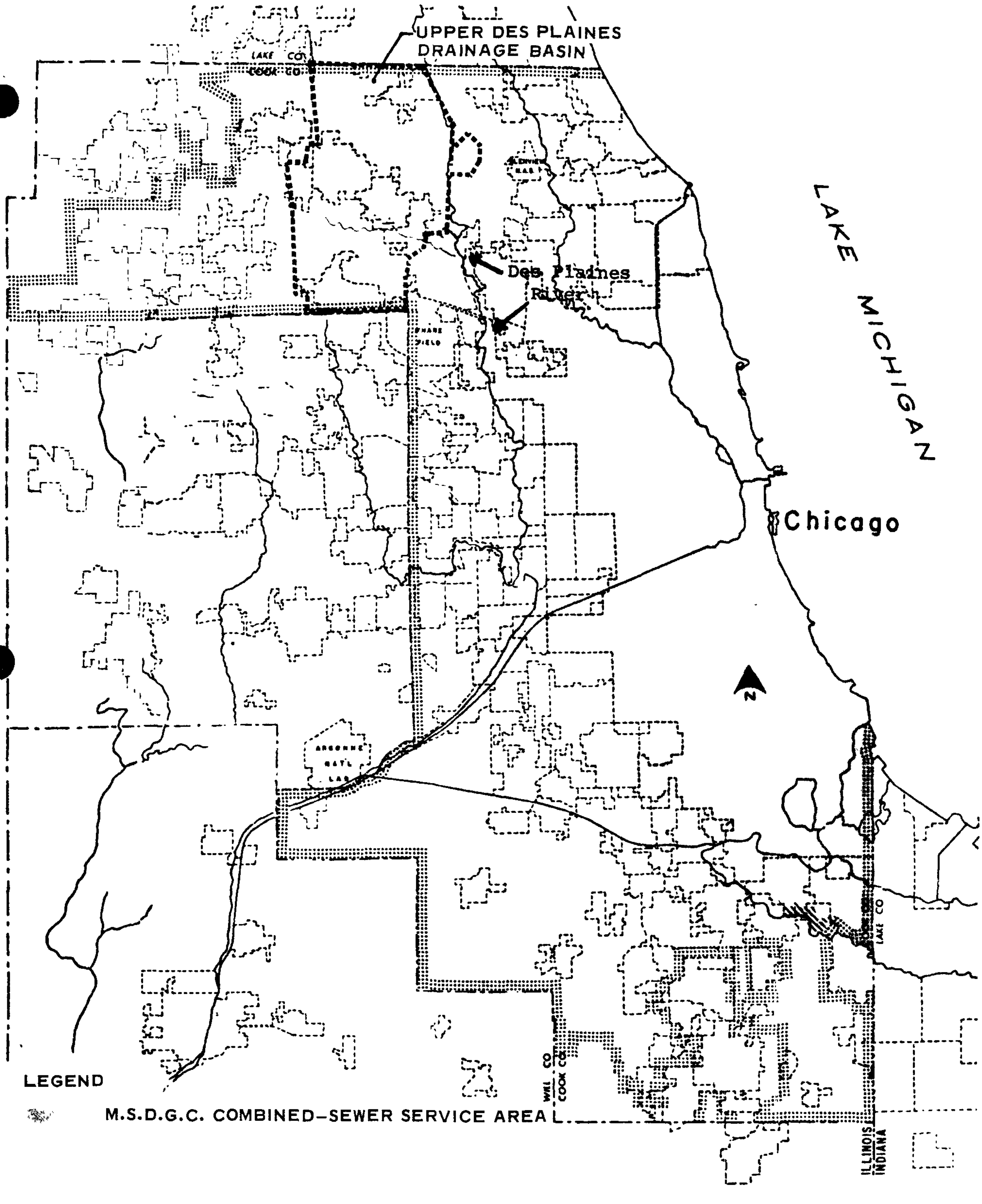


FIGURE 1-1

METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO GENERAL SERVICE AREA

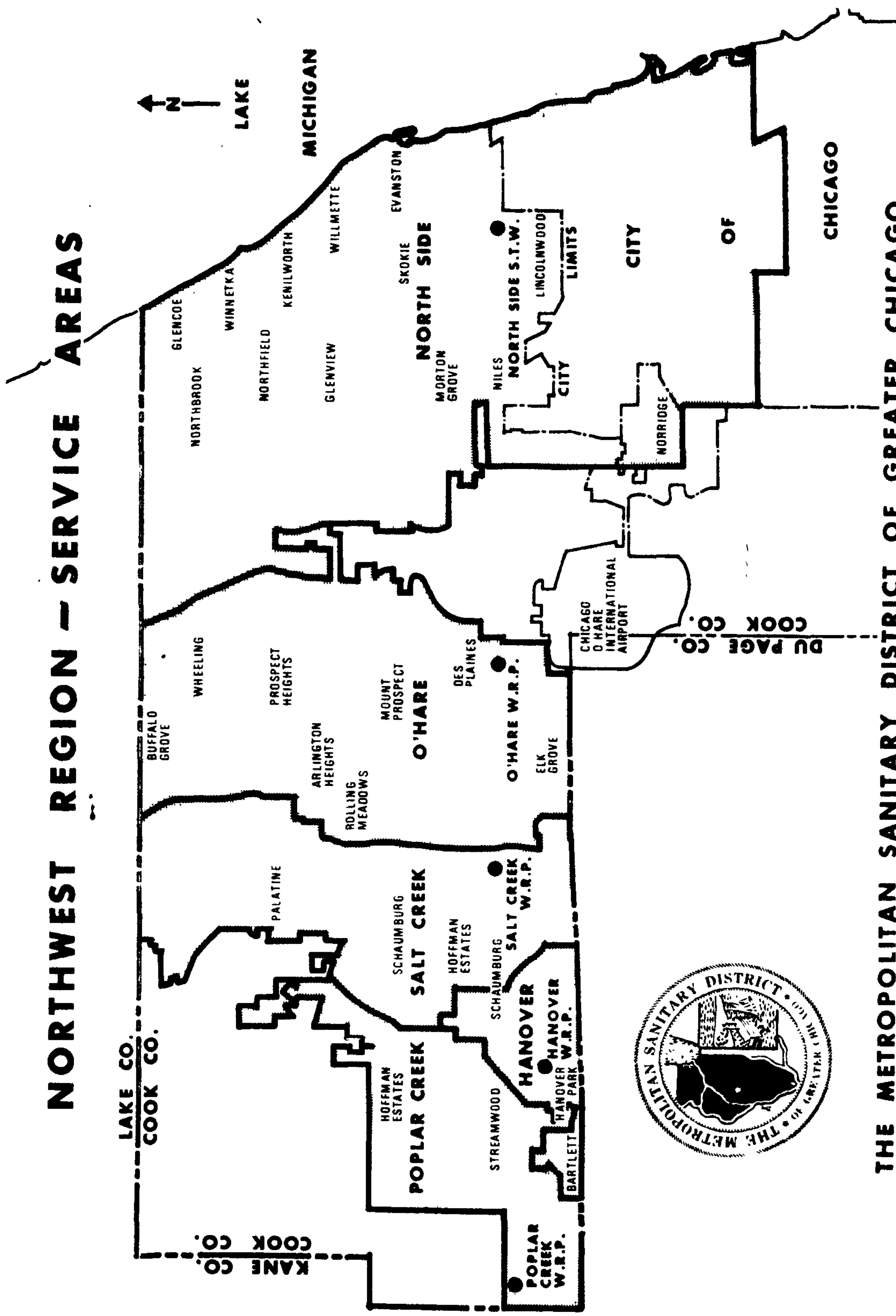

NORTHWEST REGION — SERVICE AREAS
THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO
LAKE MICHIGAN
N
LAKE CO.
COOK CO.
KANE CO.
COOK CO.
DU PAGE CO.
COOK CO.
CITY OF CHICAGO
CITY LIMITS
NORTH SIDE
NORTH SIDE S.T.W.
O'HARE
O'HARE W.R.P.
SALT CREEK
SALT CREEK W.R.P.
HANOVER
HANOVER W.R.P.
POPLAR CREEK
POPLAR CREEK W.R.P.
GLENCOE
WINNETKA
KENILWORTH
WILLMETTE
EVANSTON
NORTHBROOK
NORTHFIELD
GLENVIEW
SKOKIE
MORTON GROVE
NILES
LINCOLNWOOD
NORRIDGE
WHEELING
PROSPECT HEIGHTS
MOUNT PROSPECT
DES PLAINES
BUFFALO GROVE
ARLINGTON HEIGHTS
ROLLING MEADOWS
ELK GROVE
CHICAGO O'HARE INTERNATIONAL AIRPORT
PALATINE
SCHAUMBURG
HOFFMAN ESTATES
STREAMWOOD
HANOVER PARK
BARTLETT
THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

FIGURE 1-2

b. Groundwater from the deeper Cambrian-Ordivician aquifer. The pumpage rate in the region of the project has reportedly exceeded the sustained yield of the Cambrian-Ordivician aquifer which has resulted in a decline of the piezometric head averaging about 10-15 feet/year in the project area. The municipal and industrial pumpage appears to be from the deep aquifer which estimated on population, may have amounted to 20 to 25 MGD for 1970 in the project area.

c. Surface water from Lake Michigan. It is anticipated that larger quantities of Lake Michigan water will be made available to municipalities outside of Chicago in the future to limit the pumpage rates to the practical sustained yield in the project area. Des Plaines presently obtains 70 percent of its water from Lake Michigan through the City of Chicago System.

For a more detailed discussion of water supply issues, see REGIONAL WATER SUPPLY REPORT #8, Northeastern Illinois Planning Commission, September, 1974.

2. Sanitary and Combined Sewers

Approximately 5,000 acres of the 37,250 acres in the Upper Des Plaines Basin are expected to remain undeveloped and unsewered. This 5,000 acre area consists of special use land such as the Ned Brown Forest Preserve, cemeteries and the U. S. Military Reservations. Of the remaining 32,250 acres, 26,298 acres are presently (or will be in the near future) serviced by separate sanitary and storm sewers, and 5,952 acres are serviced by combined sewers. In addition, there are 1,370 acres of separate sewered areas that connect directly to the combined sewer systems in such a way that the flows cannot be physically separated except through extensive and costly construction. Figure 1-3 illustrates the area contributing combined overflows to the Upper Des Plaines project. Those areas indicated are: 1) the combined sewered area, 2) the separated sewer area contributing to the project, and 3) the boundaries of all sewered areas contributing to Weller's Creek and Feehanville Ditch. There are about 5,448 acres within the boundary of the sewered area contributing to Weller's Creek and Feehanville Ditch that are served by separate sewer systems. The storm flows from these areas will continue to discharge directly into Weller's Creek and Feehanville Ditch and are not a part of the proposed project.

At present all sanitary sewage, and the combined sewage in the O'Hare Service Area, except for overflows, is finding its way into Metropolitan Sanitary District interceptors through regulated control structures, and is

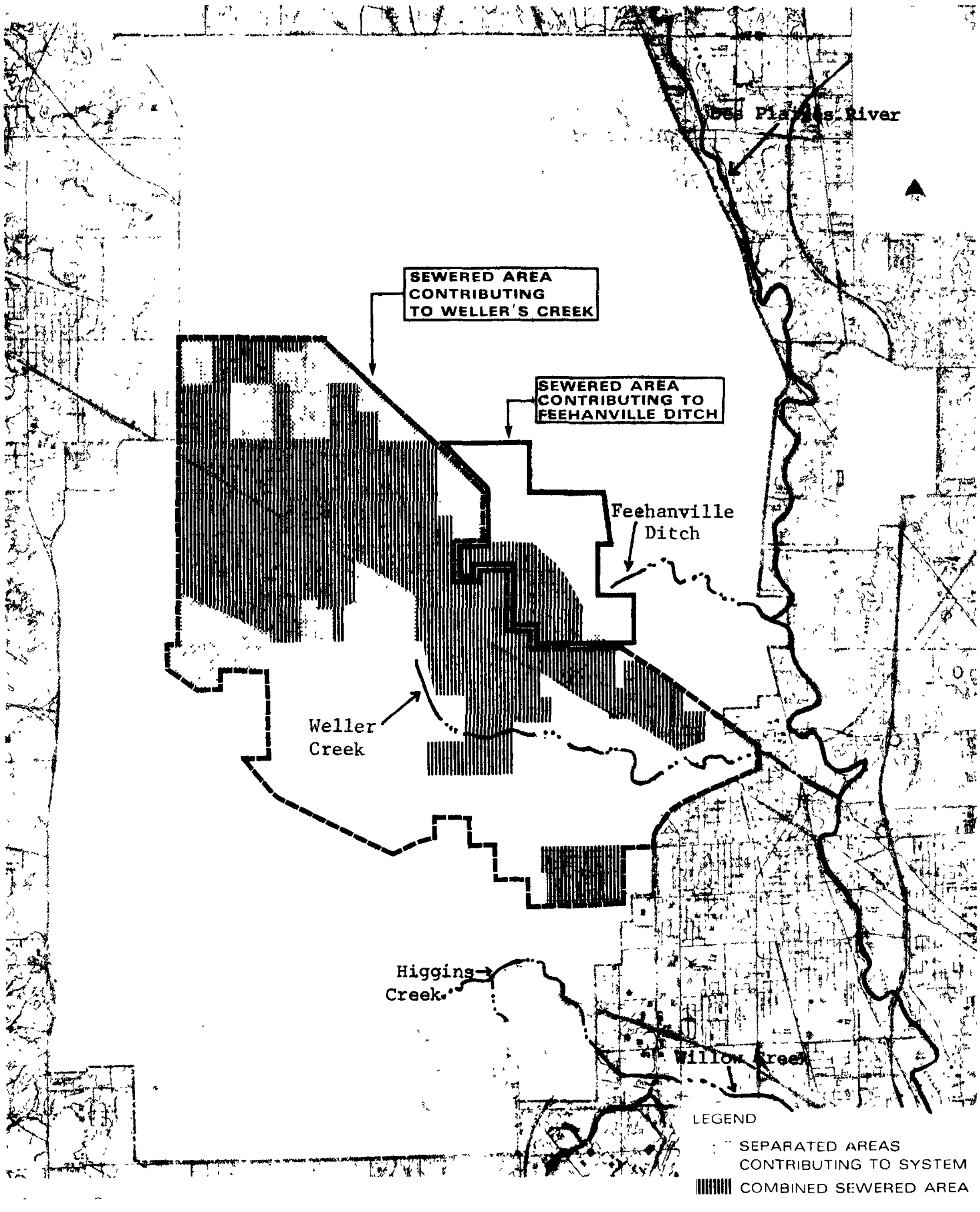


FIGURE 1-3

COMBINED-SEWER SERVICE AREA

diverted through existing interceptors to the MSDGC's North Side Sewage Treatment Works for treatment.

During wet weather, the North Side Sewage Treatment Plant is presently overloaded and existing interceptors are approaching capacity. In addition there is frequent discharge (average: 80 per year) of combined sewage to Weller's Creek and Feehanville Ditch creating an unsightly, odorous condition, as well as a potential health hazard. This untreated sewage then flows into the Des Plaines River. Weller's Creek serves as the main conveyance facility for the discharge from combined sewers serving the watershed. Backup in the combined sewers is the primary cause for basement flooding. Some homes are affected in this manner from almost all rainfalls in the watershed. Combined backup will flood approximately 100 basements for the 5- to 10-year storm event. Street flooding will begin to appear for this same storm event.

Overbank flooding does not occur until the 25-30-year storm event occurs.

The water quality standards that determine the effluent parameters for the proposed Wastewater Reclamation Plant are found in the WATER POLLUTION REGULATIONS OF ILLINOIS.

E. Water Quality & Water Quantity Objectives in the Area Other Than Solution of Preceding Problems

The following programs are relevant:

1. The Federal Water Pollution Control Act Amendments of 1972

(P.L. 92-500) require:

a. Secondary treatment of wastes for municipal sewage and best practicable treatment for industrial discharges by July 1, 1977.

b. Best practicable waste treatment technology for municipal wastes and best available treatment for industrial wastes by July 1, 1983.

c. All point-sources discharges require a permit under the NPDES program (National Pollutant Discharge Elimination System). The NPDES permit states the allowable waste loading and flow volume that can be discharged to a receiving stream, lake or ocean.

2. The National Flood Insurance Act of 1968 requires the designation of flood-prone areas in the United States and participation by the appropriate communities and homeowners to qualify for national flood insurance protection. The flood-prone areas in the O'Hare Service Area have been determined for the 100 year storm event and these maps, except for the Arlington Heights quadrangle, are available from the Northeastern Illinois Planning Commission (NIPC).

3. The Flood Control Activities planned by MSDGC for the O'Hare Service Area are discussed in Appendix A.

4. The MSDGC Tunnel and Reservoir Plan (TARP) for control of flood and pollution problems due to combined sewer discharges

in the general service area of the MSDGC is described in Appendix B. The U.S. Senate Committe on Public Works (93rd Congress, 1st session) directed the Army Corps of Engineers to determine the Federal interest in participating in the TARP program. Since the Corps viewed any potential Federal participation to be a significant Federal action, they determined that part of their response in determining Federal interest should be the preparation of an Environmental Impact Statement. Prior to the issuance of a draft EIS in November 1973, an Environmental Assessment (EA) on the TARP program was prepared. USEPA participated in discussions during the preparation of that EA and made suggestions with respect to potential environmental impacts to be addressed. A public hearing on the TARP EA was held July 26, 1973 and discussion was presented relating to the alternative plans presented. The O'Hare Service Area, since it contains some combined sewers, was considered in all alternative TARP plans. In some TARP alternatives, the O'Hare service area was sewered by tunnels only, with wastewater treatment occurring at the MSD North Side STP or WSW (Stickney) STP. Although this alternative was considered, it was not supported in other engineering studies for the O'Hare Service Area. These reports support a WRP for the O'Hare Service Area and are discussed in Appendix C.

USEPA has determined that the O'Hare Service Area can be separated from the TARP program with respect to building the treatment plant and the conveyance system to it. No determination has been made with respect to building a combined sewage overflow reservoir or interconnecting the proposed conveyance system to the lower Des Plaines TARP system.

F. Costs and Financing

The project cost for the O'Hare conveyance system is approximately \$36.5 million. This estimate includes the cost of all physical elements of tunnels, interceptors, and connecting structures together with a 20 percent factor for contingencies.

Financing of the conveyance facilities portion of the project would be through local and Federal funds. Twenty-five percent, \$9.1 million, of the project would be financed from an existing \$380 million MSDGC bond issue with the remaining 75 percent, \$27.4 million, from Federal grants.

G. History of the Application

Most MSDGC projects, proposed for the O'Hare Service Basin have been given a priority ranking of 31 by the Illinois Environmental Protection Agency (IEPA). The infiltration/inflow analysis for the service basin was transmitted to the IEPA on January 31, 1974. It has since been revised by the MSDGC and is under review by the IEPA. An informal review is also presently underway by this agency.

CHAPTER 2

THE ENVIRONMENT WITHOUT THE PROPOSED ACTION

A. General

The Upper DesPlaines Area Service Basin, under the jurisdiction of the Metropolitan Sanitary District of Greater Chicago (MSDGC), is located in Northern Cook County, Illinois, within the Chicago SMSA (Standard Metropolitan Statistical Area).

The service area is a 58.2 square mile in the northwest region of the MSDGC's total jurisdiction of 860 square miles within the County.

The service area has experienced rapid population growth during the last 15 years. The population for Northeastern Illinois increased 12.2% from 1960 to 1970. The following figures for communities to be served by the proposed water reclamation plant (WRP) and tunnel system indicate this growth.

Community	1960	1970	% change
Arlington Heights	27,878	64,884	132.7
Buffalo Grove	1,492	11,799	690.8
DesPlaines	34,886	57,239	64.1
Elk Grove	6,608	21,866	231
Mount Prospect	18,906	34,995	85.1
Prospect Heights	. . .	13,333	. . .
Rolling Meadows	10,879	19,178	76.3
Wheeling	7,169	14,746	105.7

(All figures from U.S. Dept. of Commerce, Bureau of the Census, publication PC(1)A15-Ill.)

Figure 1-2 indicates the service area and location of the communities. The service area is predominantly residential in character. The 1970 population for the area was 223,000. Growth in the area has been encouraged by several factors including the presence of O'Hare Airport, Northwest Tollway, Tri-State Tollway and the Northwestern Railroad line. The area is 60% developed and construction of light industrial facilities and residential units (both single family and multi-family) is ongoing to date.

The economic condition of the area's population is above the Chicago SMSA median family income of $11,841 and State of Illinois Median family income of $10,959.

1970 census figures indicate the following Median family incomes:

Arlington Heights	$17,034
Buffalo Grove	$14,833
DesPlaines	$14,056
Elk Grove Village	$14,155
Mount Prospect	$16,503
Prospect Heights	$15,992
Rolling Meadows	$13,343
Wheeling	$13,398

These figures indicate a healthy economic situation within the service area.

Few environmentally sensitive areas are within the service area. A small portion of the Cook County Forest Preserve District's Ned Brown Preserve of 3,600 acres occupies the western portion of the area. The Forest Preserve District's holdings along the DesPlaines River are located in the eastern portion of the area.

B. <u>Detailed Description</u>

1. Climate

The continental climate of the service area has relatively warm summers and cold winters, with frequent short fluctuations in temperature, humidity, cloudiness, and wind direction. Temperatures of 96°F. or greater occur in about half of the summers while about half of the winters may have low extremes of -15°F. The mean annual temperature is 49°F. Precipitation averages 33 inches per year, with about 10% of this occuring as snow. Summer rainfall is unevenly distributed in intense local showers while precipitation in the fall, winter, and spring tends to be more uniform over large areas. Winds are most commonly from the southwest and the northwest, on an annual basis. Tornadoes occur in Northern Illinois and are most prevalent in March, April, May, and June. Other periodic hazards include severe thunderstorms, hail, and ice storms. Fog is infrequent in the Chicago area. Detailed climatological data are available from O'Hare International Airport, at the south end of the study area.

2. Topography

The service area is 58.2 square miles, sloping from about 700 feet above sea level at the western boundary to about 625 feet above sea level at the Des Plaines River, 6 1/2 miles to the east.

Landforms are gently rolling, with slight vertical relief. Most of the study area has undergone the transition from a region of small towns and farm land to an extensive suburban area of single family homes, apartments, and commercial and industrial development.

3. Geology

The project area is underlain by three geologic systems. The stratigraphic sequence is the Quarternary System, the Silurian System and the Ordovician System. (See Figures 2-1 and 2-2).

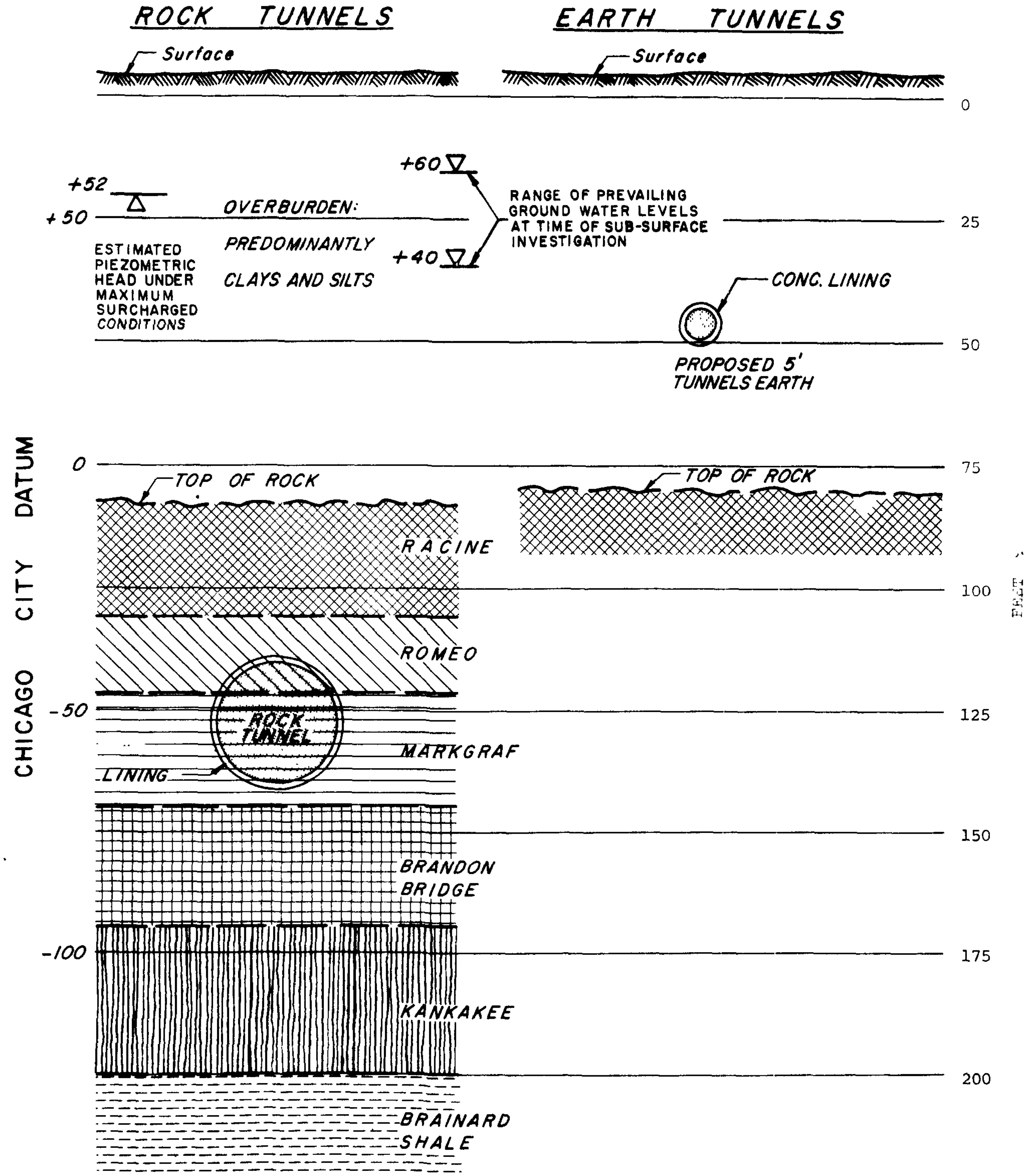
The Quarternary System is composed solely of material from the Pleistocene series. The formations contained within the series are the Wadsworth and Wedron. The main constituents of both are clayey silts with sand lenses orginating from glacial deposits.

The Silurian System lies under the Quarternary System and contains material originating from the Niagaran and Alexandrian series. The Niagaran series contains the Racine, Waukesha and Joliet formations. The Racine and Waukesha formations are composed of argillaceous fine grained dolomite while the Joliet formation is a lighter gray dolomite. The Kankakee and Edgewood formations comprise the Alexandrian Series. Dolomite is also the major portion of these formations ranging from fine to shaly in texture.

System	Series	Formation/Member		Column	Description
QUATERNARY	Recent				
QUATERNARY	Pleistocene	WADSWORTH MEMBER			Till and outwash deposits. Clayey silt with sand lenses. (Gravel lenses possible but not probable - described in soils report)
QUATERNARY	Pleistocene	WEDRON FORMATION			Bouldery till, clayey silt with sand lenses, gravel, boulders common near base and at unconformity. (Described in soils report.)
SILURIAN	Niagaran	RACINE (0-300')			Gray-brown, argillaceous, fine grained, thin bedded dolomite containing reefs of pure, gray, massive, vuggy, dolomite.
SILURIAN	Niagaran	(WAUKESHA) (0-20')			Gray, fine grained, silty dolomite. (Generally absent in northern area)
SILURIAN	Niagaran	JOLIET (40-70')	Romeo		Light gray, pure, porous dolomite
SILURIAN	Niagaran	JOLIET (40-70')	Markgraf		Light gray, silty, very fine grained dolomite
SILURIAN	Niagaran	JOLIET (40-70')	Brandon Bridge		Red or greenish gray dolomite and interbedded shale
SILURIAN	Alexandrian	KANKAKEE (20-50')			Light brown, fine grained dolomite with prominent wavy clay partings.
SILURIAN	Alexandrian	(EDGEWOOD) (0-100')			Brown to gray shaley dolomite. (Cherty near top. Not recognized in project area)
ORDOVICIAN	Cincinnatian	Maquoketa Group	NEDA (0-15')		Oolite and red shale (Generally absent)
ORDOVICIAN	Cincinnatian	Maquoketa Group			Oolite and red shale. (Generally absent)
ORDOVICIAN	Cincinnatian	Maquoketa Group	BRAINARD SHALE (0-100')		Green to brown fossiliferous mudstone
			Base not described		

FIGURE 2-1

STRATIGRAPHIC SEQUENCE



GENERALIZED STRATIGRAPHIC SECTIONS

FIGURE 2-2

The final system is the Ordovician, composed of the Cincinnation series which has two formations; the NEDA and Brainard Shale Red shale and fossiliferous mudstone comprise the majority of these formations.

The above discussion emcompases approximately the first eight hundred feet of earth. There are two main aquifers contained in the above mentioned geologic structures. They lie in the Silurian and Ordivician Systems.

The Silurian aquifer has an average depth of 108 to 205 feet. It is composed mainly of glacial till material. The uppermost material, in the area of access tunnels and work shafts is slightly more porous than that surrounding the rock tunnel. The coefficient of permeability (C_p) of the glacial till is 10^{-6} to 10^{-8} cm/sec. Because of the low C_p there will be no significant release of water to the tunnel through seepage. Any seepage that will occur results from openings primarily in the form of cracks and joints in the rock. The location of inflows in this case can easily be located after excavation, particularly within the machine bored section of the tunnel and may be appropriatly grouted. Another source of inflow may occur during the boring of work shafts or access tunnels. The inflow will originate from ground water seepage, to the upper portions of the shaft and tunnels. This seepage may also be arrested by the use of grouting techniques. Within the formation there exists sand gravel pockets which hold limited amounts of water. If they are encountered by construction, the water may be released to the tunnel. These quantities of water appear to be extremely limited and are not known to be used as potable water supply sources.

As an overall view it is anticipated that the drawdown of the water supply aquifier during operation of the facility will be virtually zero. It is expected that grouting will reduce the groundwater flow into the tunnels to less than 300 gpm over the total length of tunnels based on the results obtained from previous projects. Tunnel lining which is planned, will further reduce inflows.

Since the tunneling lies within the glacial till-Silurian system and concurrently within the Silurian aquifier area, a discussion of the Ordivician aquifier will be left to the water supply section of this statement. Groundwater and surface water recharge of the aquifers will also be addressed in that section. A more complete discussion of the bedrock geology can be found in Appendix D.

4. Soils

The soils of the study area have developed from glacial parent materials, under prairie and transitional (prairie to woodland) vegetation. Alluvial soils have developed along stream floodplains. Most soils generally have fairly slow permeabilities and high seasonal water tables, resulting in poor drainage. Despite the slow drainage erosion control is desirable to avoid soil loss and the sedimentation of streams.

5. Hydrology

A. Surface Water

The study area is located in the drainage basin of the DesPlaines River. Several small streams originate in the study area and flow eastward to join the DesPlaines River. The streams and their drainage basins have been and are being modified as the area develops. Named tributaries of the project area include: Buffalo Creek and Wheeling Drainage Ditch; McDonald Creek; Weller's Creek and Weller's Drainage Ditch; Higgins and Willow Creek; and Feehanville Ditch. All of these watercourses have a 7-day once in 10-year low flow of zero. The natural drainage boundaries for Weller's Creek and Feehanville Ditch are indicated in Figure 2-3.

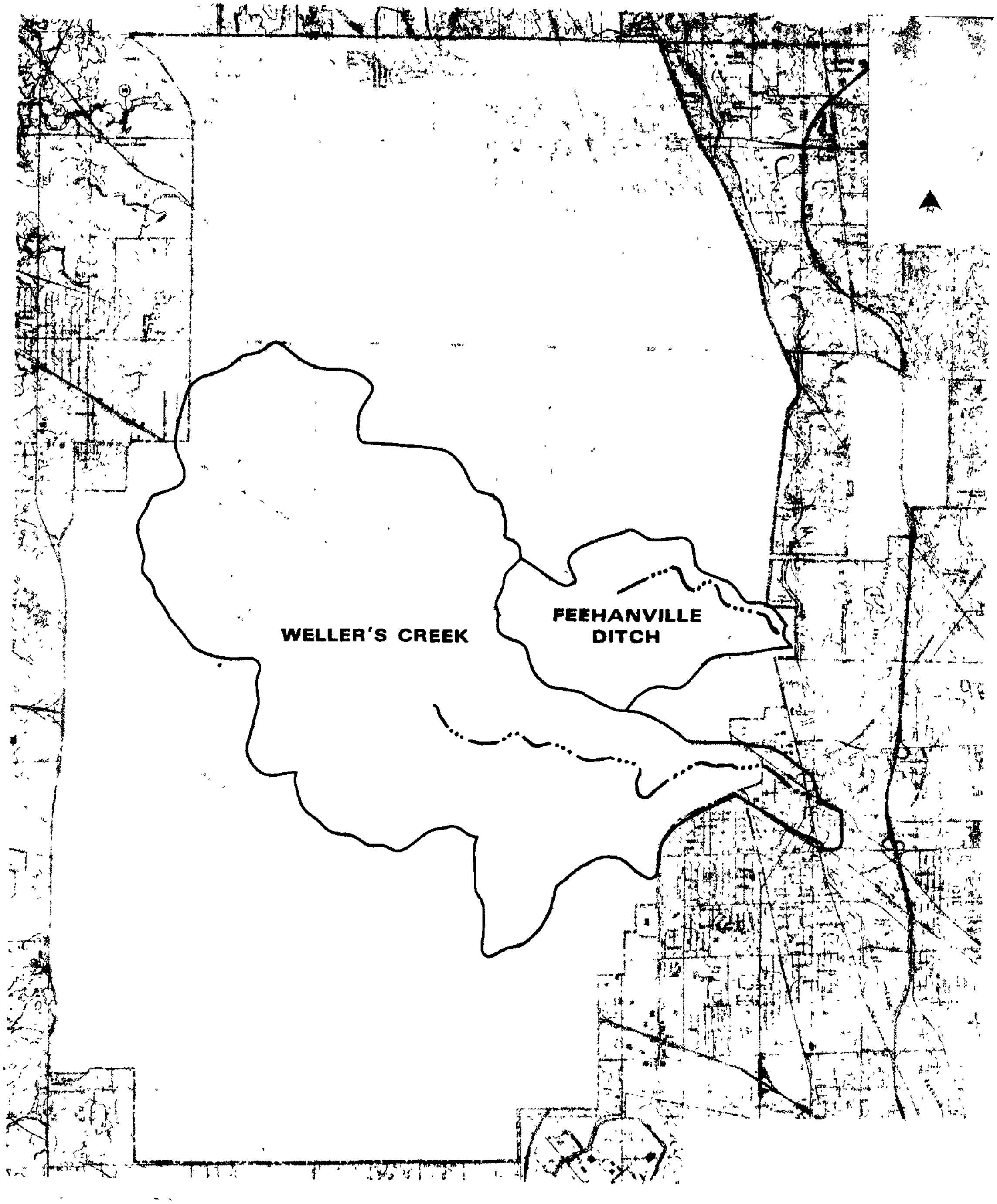


FIGURE 2-3

NATURAL DRAINAGE BOUNDARIES
WELLER'S CREEK AND FEEHANVILLE DITCH

Weller's Creek, which has a total length of approximately 6.5 miles, is joined by a number of smaller tributaries and drains an area of approximately 10,780 acres. Portions of this stream have been relocated, some areas have been channelized and other areas are in underground conduits. Modifications of dendritic extremities have been most extensive as many have been eliminated by developments, and other portions are underground. The vast majority of this drainage basin has been urbanized.

Feehanville Ditch extends for approximately 2.5 miles and drains an area of approximately 1,990 acres. This watercourse and its drainage basin have been substantially modified by urbanization. The headwaters of Feehanville Ditch are underground as is a portion north of Maryville Academy, and a large portion of the stream has been channelized.

Higgins Creek is about five miles in length and drains approximately 5,000 acres before joining Willow Creek at a point approximately three to three and a half miles upstream from the Des Plaines River. The majority of the Higgins Creek area is highly urbanized with some industrial and agricultural uses. Higgins Creek has been filled, relocated and channelized in several places.

The flow rates of Weller's Creek at Golf Road have been monitored by the United States Geological Survey. The rates of flow are as follows: two-year flood - 520 cfs; five-year flood - 900 cfs; ten-year flood - 1,200 cfs; 25-year flood - 1,600 cfs; 100-year flood - 2,400 cfs. No data are available for the smaller drainage basin of Feehanville Ditch.

The water quality of Weller's Creek and Feehanville Ditch has drastically deteriorated with the increasing urbanization of the respective drainage basins. Computer model simulation estimated that during the 21-year period from 1949 to 1969, effluents from the combined sewers of this area have overflowed into the above streams 1,660 times discharging a total 140,000 acre-feet of sewage which had contained 146,000,000 pounds of suspended solids, and created a BOD (Biological Oxygen Demand) of 20,200,000 pounds.

The United States Geological Survey estimated the 10, 50, 100 and 500-year flow rate of Higgins Creek at Mount Prospect Road to be 840,1,250, 1,650 and 2,180 cfs respectively. The water quality in Higgins Creek is poor and is probably below State standards. Existing urban activities contribute polluted stormwater runoff to the natural flow of Higgins Creek probably adding significant quantities of inorganic and organic pollutants.

The Illinois Environmental Protection Agency sampled Weller's Creek during 1971. Table 2-1 compares many parameters

WATER QUALITY DATA OF WELLER'S CREEK*
COMPARED TO STATE STANDARDS**

Table 2-1

Weller's Creek	Unit	State Standards
Water		
Temperature (F°)		
Number of Analyses	10	The maximum temperature rise above natural temperature shall not exceed 5°F. January 60°F. Maximum August 90°F. Maximum
Maximum Value	75	
Minimum Value	36	
Average Value	55	
Field Dissolved Oxygen (mg/1)		
Number of Analyses	8	Not less than 6.0 mg/1 during at least 16 hours of any 24 hour period,nor less than 5.0 mg/1 at any time.
Maximum Value	8.5	
Minimum Value	0.0	
Average Value	2.9	
Turbidity (JTU)		
Number of Analyses	10	Waters shall be free from unnatural sludge or bottom deposits, floating debris, visible oil, odor, unnatural plant or alga growth, or unnatural odor or turbidity.
Maximum Value	800	
Minimum Value	17	
Average Value	102	
Total Solids (Dissolved) (mg/1)		
Number of Analyses	10	1,000 mg/1
Maximum Value	1,309	
Minimum Value	234	
Average Value	701	

* Water Quality Network, 1971, Summary of Data, Volume 2. State of Illinois, Environmental Protection Agency.

** Illinois Pollution Control Board, Rules and Regulations, Chapter 3, Water Pollution. July 1973.

WATER QUALITY DATA OF WELLER'S CREEK
COMPARED TO STATE STANDARDS

Table 2-1 (continued)

Weller's Creek	Unit	State Standards
Fecal Streptococcus (per 100 ml)		
Number of Analyses	3	No State standards
Maximum Value	37,000	
Minimum Value	270	
Average Value	15,090	
Coliform (per 100 ml)		
Number of Analyses	5	No State standards
Maximum Value	700,000	
Minimum Value	13,000	
Average Value	188,600	
Chemical Oxygen Demand (mg/1)		
Number of Analyses	10	No State standards
Maximum Value	120	
Minimum Value	22	
Average Value	49	
Biochemical Oxygen Demand (mg/1)		
Number of Analyses	1	No State standards
Maximum Value	5	
Minimum Value	5	
Average Value	5	

WATER QUALITY DATA OF WELLER'S CREEK
COMPARED TO STATE STANDARDS

Table 2-1 (continued)

Weller's Creek	Unit	State Standards
pH		
Number of Analyses	10	Shall be within the range of 6.5 - 9.0.
Maximum Value	8.3	
Minimum Value	7.3	
Average Value	7.7	
Total Phosphate (mg/1 of PO_4)		
Number of Analyses	10	Phosphorus as P shall not exceed 0.05 mg/1.
Maximum Value	4.2	
Minimum Value	0.3	
Average Value	1.7	
Ammonia (mg/1 of N)		
Number of Analyses	5	Shall not exceed 1.5 mg/1
Maximum Value	3.8	
Minimum Value	0.6	
Average Value	2.1	
Chloride (mg/1)		
Number of Analyses	10	Shall not exceed 500 mg/1
Maximum Value	395	
Minimum Value	70	
Average Value	181	

WATER QUALITY DATA OF WELLER'S CREEK
COMPARED TO STATE STANDARDS

Table 2-1 (continued)

Weller's Creek	Unit	State Standards
Fluoride (mg/l)		
Number of Analyses	7	Shall not exceed 1.4 mg/l
Maximum Value	0.6	
Minimum Value	0.3	
Average Value	0.4	
Iron (Total) (mg/l)		
Number of Analyses	1	Shall not exceed 1.0 mg/l
Maximum Value	0.1	
Minimum Value	0.1	
Average Value	0.1	
Phenols (mg/l)		
Number of Analyses	3	Shall not exceed 0.1 mg/l
Maximum Value	70	
Minimum Value	0	
Average Value	23	
Sulfate (mg/l)		
Number of Analyses	10	Shall not exceed 500 mg/l
Maximum Value	215	
Minimum Value	42	
Average Value	99	
Fecal Coliforms (per 100 ml)		
Number of Analyses	10	Based on a minimum of 5 samples taken over not more than a 30 day period, shall not exceed a geometric mean of 200/100ml nor shall more than 10% of the samples during any 30-day period, exceed 400/ml.
Maximum Value	80,000	
Minimum Value	400	
Average Value	18,180	

of water quality with State standards. The water quality of Weller's Creek is below State standards for the following parameters; dissolved oxygen, total dissolved solids, total phosphate, ammonia, phenols and fecal coliforms. The water quality of Feehanville Ditch probably approaches the same magnitude of degradation as presently exists in Weller's Creek.

6. Groundwater Aquifers in the Service Area

The Silurian bedrocks of the study area are overlain by 45 to 100 feet of glacial material. The textural composition of material, which is often interbedded, ranges from clay to clayed silt, and usually contains varying amounts of sand, gravel and boulders. Waterbearing sand layers are common to this glacial deposit. Analysis of drilling data indicates the water tables of this area vary from 20 to 25 feet in the summer to around 40 feet in the winter.

The shallow aquifers of this glacial drift are hydraulically connected with the underlying Silurian rocks. Groundwater in the Silurian and Ordovician rocks occurs in joints, fissures, solution cavities and other openings. The water-yielding openings are irregularly distributed both vertically and horizontally. Available geohydrologic data indicate that the rocks contain numerous openings which extend for considerable distances and are interconnected on an areal basis.

Large quantities of groundwater are withdrawn from wells in shallow dolomite aquifers of Silurian and Ordovician age in northern Illinois. The Niagaran and Alexandrian Series of Silurian age yield moderate to large quantities of groundwater.

Most water-yielding openings occur in the upper one-third of the shallow dolomite aquifers. A good relationship exists between glacial drift and the upper part of the shallow dolomite aquifers. Highest yielding wells are found in areas where the glacial drift immediately overlying the shallow dolomite aquifers is composed of sand and gravel.

Probable ranges in yields of shallow dolomite wells can be estimated from specific-capacity frequency graphs, aquifer thickness and areal geology maps, and water-level data. On the basis of these data, potential wells of the project area could yield up to 40 to 60 gpm (gallons per minute).

Recharge of the upper glacial drift-Silurian aquifer appears to occur from local precipitation, but the low permeability of the overburden soils may be reason to suspect some horizontal movement from the west.

The lower Cambrian Ordovician aquifer reportedly received water from horizontal movement in recharge areas in North-Central Illinois and Southern Wisconsin; and vertical leakage through the overlying Maquoketa formation. In 1958 this leakage was estimated to be approximately 11 percent of the total water pumped from deep sandstone wells in the Chicago region. The vertical leakage through the Maquoketa shale is generally due to the large differential head between the aquifers (and locally may be facilitated by faults in the rock).

According to Walton (Future Water Declines In Deep Sandstone Wells in Chicago Region, Ill. State Water Survey-Reprint Series No. 36, 1964) the practical sustained yield of the deep aquifers in the Chicago region is 60 MGD which is less than the actual pumpage. It is anticipated that Lake Michigan water may be made available to municipalities in the future to limit the pumpage rates to the practical sustained yield in the project area.

Regionally, the shallow groundwater aquifer system reportedly has a supply in excess of pumpage and any lowering of the groundwater elevations is not anticipated except for seasonal fluctuations and local variations due to pumpage.

The pumpage rate and the pumpage subdivided by use over the whole basin from this aquifer has not been established, but available well records indicate that the majority of wells in the shallow aquifer are private domestic service with pumpout rates between 5 to 50 GPM. Municipal and industrial pumpage appears to be from the deep aquifer which estimated on population, may have amounted to approximately 20 to 25 MGD for 1970, in the project area. (Average per capita consumption 115gpd) of which approximately 3 MGD (11%) infiltrated from the shallow Silurian aquifer.

Regional groundwater quality and quantity data for Cook, DuPage, Lake, McHenry, Kane and Will counties are presented in Appendix E. This material is available in Technical Report #8 - Regional Water Supply Report, September 1974, by the Northeastern Illinois Planning Commission.

In addition, Water Conservation measures are described in the above mentioned NIPC Technical Report and are included in Appendix F.

c. Water Quality Management

Section 208 of the 1972 Federal Water Pollution Control Amendments Act of 1972 provides for areawide planning for waste treatment management in large urban - industrial areas of the nation which have severe and complex water quality problems. The northeastern Illinois counties of which the service area is a part have been identified as having such water quality problems. The Northeastern Illinois Planning Commission is currently organizing a 208 planning effort with local governmental units. With the support of local governments, the Governor of Illinois may designate an areawide waste treatment management planning area (208 area) and may designate the Northeastern Illinois Planning Commission (NIPC) as the official "responsible planning agency" for 208 planning.

At this writing, the following service area governmental units have supported through resolution, the designation of the six-county area snd NIPC as the 208 planning agency:

Arlington Heights
Mount Prospect
DesPlaines
Cook County
Buffalo Grove

MSDGC has prepared a proposal as to their participation within the 208 planning process.

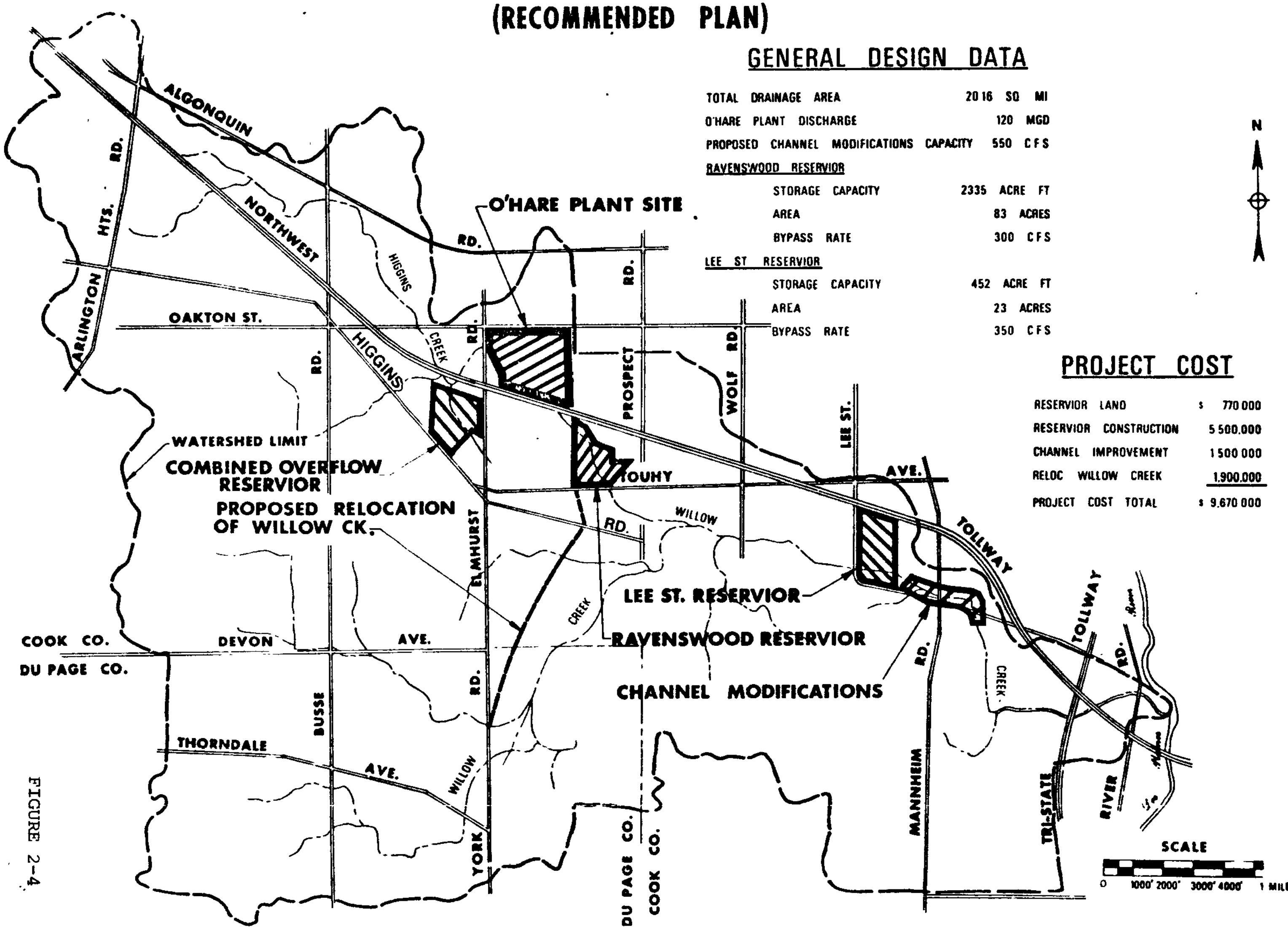
The Northeastern Illinois Planning Commission has also completed a Regional Wastewater Plan (1971), which will be a major component of the 208 study.

The Illinois Environmental Protection Agency has the responsibility for Section 303 of the 1972 Amendments whereby water quality problems are identified and overall pollution abatement strategies are established for all major river basins in the state.

d. Flood hazards

The flood-prone areas in the O'Hare service area have been mapped for the 100 year recurring flood event. These maps are available in 7.5 minute series (topographic) from the Northeastern Illinois Planning Commission. Channelization of Higgins Creek is part of the Willow-Higgins Creek Watershed, plan illustrated in Figure 2-4. This plan consists of locating storm reservoirs along Willow-Higgins Creek and channelizing various sections to protect against the 100-year flood. A summary of the O'Hare area flood control activities is found

WILLOW - HIGGINS CREEK WATERSHED (RECOMMENDED PLAN)



GENERAL DESIGN DATA

TOTAL DRAINAGE AREA	20.16 SQ MI
O'HARE PLANT DISCHARGE	120 MGD
PROPOSED CHANNEL MODIFICATIONS CAPACITY	550 CFS
RAVENSWOOD RESERVIOR	
STORAGE CAPACITY	2335 ACRE FT
AREA	83 ACRES
BYPASS RATE	300 CFS
LEE ST RESERVIOR	
STORAGE CAPACITY	452 ACRE FT
AREA	23 ACRES
BYPASS RATE	350 CFS

PROJECT COST

RESERVIOR LAND	$ 770 000
RESERVIOR CONSTRUCTION	5 500,000
CHANNEL IMPROVEMENT	1 500 000
RELOC WILLOW CREEK	1,900,000
PROJECT COST TOTAL	$ 9,670 000

FIGURE 2-4

in Appendix A.

6. Biology

Most of the study area has become urbanized, with the original prairie vegetation and oak-hickory deciduous forest being replaced by agricultural lands, yards, parks, and urban areas. Principal remaining natural areas occur along the Des Plaines River and its tributary streams, and in the Ned Brown Forest Preserve. A variety of birds and small mammals inhabit the service area. Agricultural and urban runoff have polluted streams and affected the original composition of stream plants and animals. No endangered or rare species from State and Federal lists are known to be present in this area.

7. Air Quality

In order to evaluate the existing air quality in the vicinity of the proposed projects, air quality data was gathered from several sources. These included the Illinois Environmental Protection Agency, the Cook County Department of Environmental Control, the City of Chicago Department of Environmental Control, and an "Airport Vicinity Air Pollution Study" conducted by the Energy and Environemntal Systems Division of Argonne National Laboratory.

2. Particulate Matter

The greatest amount of data available is the result of particulate matter sampling. Data from the Argonne study indicate that for sampling stations west of O'Hare levels vary from 46 $\mu g/m^3$ in upwind conditions to 66 $\mu g/m^3$ for downwind conditions. On the other hand, levels at stations east of O'Hare vary from 112 $\mu g/m^3$ in upwind conditions, to 66 $\mu g/m^3$ in downwind conditions. The increase in particulate values when winds are from the west suggests that the airport does make a measurable contribution to the particulate loading downwind of the airport.

The primary national ambient air quality standard is an annual average no greater than 75 $\mu g/m^3$ and a 24-hour maximum no greater than 260 $\mu g/m^3$. Samples taken on airport property show that 100% of the 24-hour values were 240 $\mu g/m^3$ <u>or less</u> while 100% of the 24-hour samples outside the airport were 180 $\mu g/m^3$ <u>or less</u>.

At a Cook County sampling station southeast of O'Hare (Franklin Park) the annual mean concentration of particulate matter in 1974 was 74 $\mu g/m^3$. At another station northwest of O'Hare (and downwind), the annual mean concentration for 1974 was 67 $\mu g/m^3$. While both of these stations met the primary standard for particulate matter, they were in violation of the secondary standard of 60 $\mu g/m^3$. Data from a City of Chicago sampling station east of O'Hare (Taft High School) from January, 1966 to December, 1974 shows an average annual mean concentration of 89 $\mu g/m^3$. Obviously, it is very difficult to draw any conclusions from this data because of the variability of wind direction and the effects of surrounding area emissions. It does appear however, that samples taken close to airport sources generally violate standards, but that the concentrations of particulate matter decrease with increasing distance from the airport.

b. Nitrogen Oxides

Because there is even less data available on this pollutant, it becomes even more difficult to note any significant trends. National ambient air quality standards state that, as an annual average, photochemical oxidants should not exceed 160 $\mu g/m^3$ nor should they exceed 0.08 ppm as a one-hour maximum. While some samples taken during the Argonne study recorded levels as high as 540 $\mu g/m^3$ (or 0.262ppm), the variability in samples was extensive with some readings as low as 2.4 $\mu g/m^3$. For example, samples taken along the northern perimeter of the airport range from 220 $\mu g/m^3$ to 540 $\mu g/m^3$. Along the eastern perimeter of the airport values ranged from 52 $\mu g/m^3$ to 187 $\mu g/m^3$. Comparisons of samples on airport property and those outside O'Hare show levels of 209 $\mu g/m^3$ for the former and 109 $\mu g/m^3$ for the latter.

Results of samples taken by Cook County show an annual 1974 mean of 65 $\mu g/m^3$ with a range from 32 $\mu g/m^3$ to 110 $\mu g/m^3$. Similar samples taken by the City of Chicago east of O'Hare (Taft High School) indicate a 1974 annual average of 0.036 ppm. The Argonne study concluded that concentrations of NO and NO_x were substantially higher in active mobile source areas of the airport than in the surrounding neighborhood. The highest NO_x readings were obtained at both the gate areas and near the ends of runways 14R and 14L. As with particulate matter, it can be seen that monitoring over a long period of time results in annual averages which are well within the standards. However, it is very common that in certain areas, spot samples will result in readings which greatly exceed the hourly standard.

c. Total Hydrocarbons

In the case of this pollutant it was found that the background levels of total hydrocarbons (TCH) were so high that it was not possible, in the case of the Argonne study, to determine the impact of aircraft emissions on the air quality in the area. The maximum standard for a 3-hour period, which is not to be exceeded more than once a year, is 160 $\mu g/m^3$ (or 0.24 ppm). Sampling of the northern perimeter revealed THC levels from 1934 $\mu g/m^3$ to 2330 $\mu g/m^3$ with a range from 1700 $\mu g/m^3$ to 1950 $\mu g/m^3$ along the eastern perimeter. THC levels outside O'Hare in Elk Grove Village (west of Site #1) ranged from 1535 $\mu g/m^3$ to 2100 $\mu g/m^3$. The Argonne study noted that the high background THC could be largely methane which is relatively stable in the atmosphere while the contributions coming from aircraft may contain a substantial fraction of reactive hydrocarbons so that these contributions could be significant with regard to the production of photochemical smog.

The Argonne study indicated that it was highly questionable whether aircraft emissions would have a detectable effect at ground level because of the interference with ground based emissions. Visual observations of the exhaust plumes saw them transported to ground level at distances of about one to two miles from the runway end. The visibility of the exhaust plumes near the surface within one or two miles of the airport as well as their detectability at flight levels suggest that at least one type of impact of particulate emissions is to increase the atmospheric pall in the airport vicinity.

In general, it appears that air quality in the vicinity of the project sites is severely degraded because of the proximity to O'Hare airport. While comprehensive sampling indicates that the standards for some pollutants are not violated, spot sampling would certainly indicate a noticeable degradation of the air quality in the area.

8. Land Use

Figure 2-5 indicates various land uses in the area of the proposed interceptor. The Northern Illinois Planning Commission (NIPC) has identified by 24 categories, the acreage of actual land use as of 1970. The NIPC "Landuse 70" elements by code number are:

1 - Residential - single family
2 - Residential - multi-family
3 - Residential - mobile homes
4 - Manufacturing - except wholesale
5 - Transportation, Communications, Utilities
6 - Railroad right-of-way
7 - Airports
8 - Streets
9 - Trade
10 - Services - private
11 - Services - institutional
12 - Military
13 - Cemeteries
14 - Entertainment assembly
15 - Public buildings
16 - Public and quasi-public open space
17 - Mining and excavations
18 - Vacant, Agriculture, Forest
19 - Vacant - under development
20 - Water - exclusing public open space
21 - Warehousing - storage structures
22 - Shopping centers - including parking
23 - Hotels, motels, transient lodging
24 - Parking - independent

The following table indicates the land uses of the quarter-sections through which the interceptors are proposed:

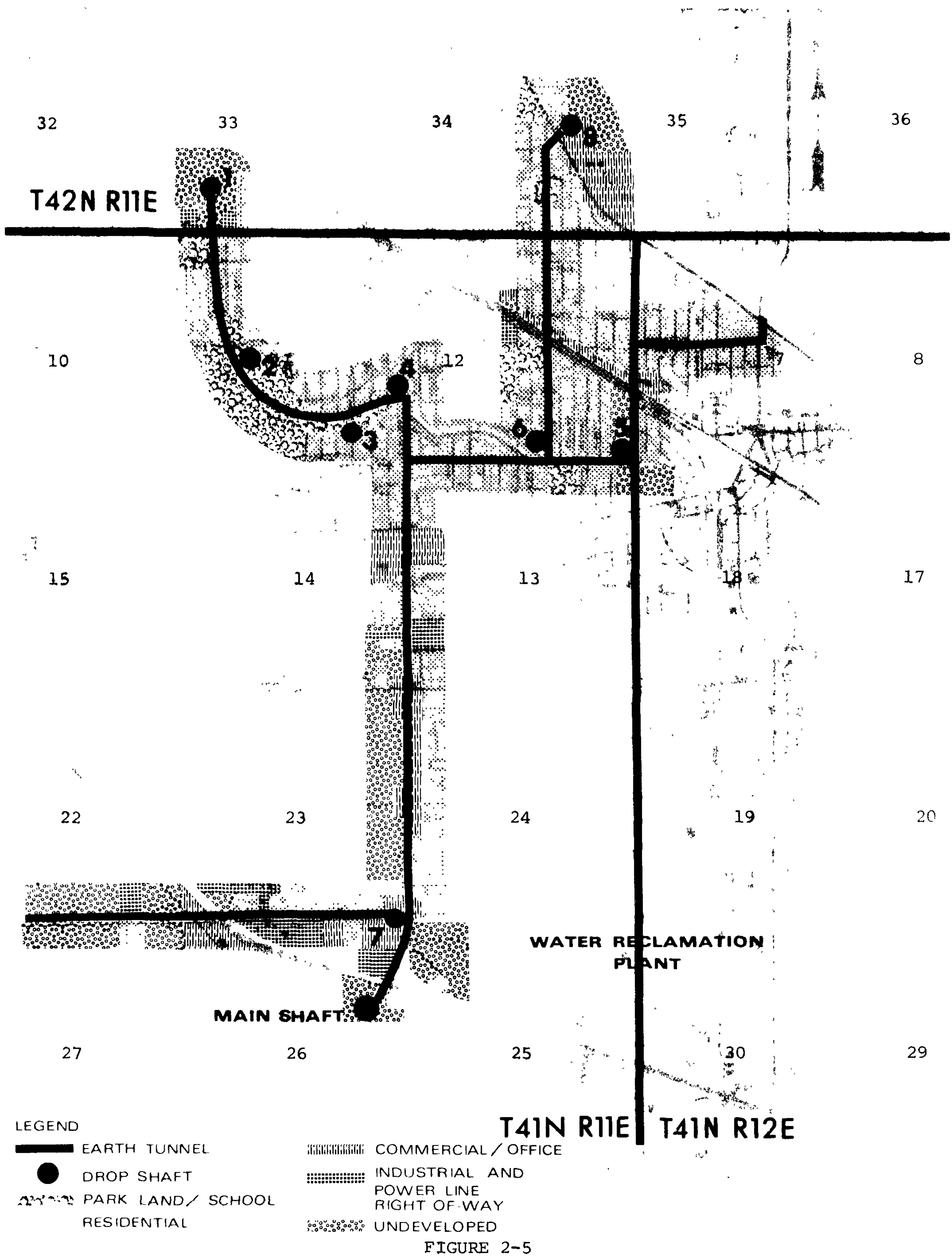


FIGURE 2-5

LAND USE

NIPC Category #	Land Use Type	Acreage
1	Residential-Single Family	1430.4 acres
2	Residential-Multi Family	82.8
3	Residential-Mobile Homes	11.8
4	Manufacturing-except wholesale	334.1
5	Transportation, Communications, Utilities	55.7
6	Railroad right-of-way	21.2
8	Streets	606.6
9	Trade	123.5
10	Services-Private	6.9
11	Services-Institutional	103.9
16	Public & quasi-public open space	171.7
18	Vacant-agriculture, forest	1218.9
19	Vacant-under development	39.7
21	Warehousing-storage	16.6
22	Shopping centers	18.7
23	Hotels, Motels	6.0
24	Parking independent	3.3

The above table indicates that the predominant existing land use categories are single family residential, streets and vacant.

According to MSDGC estimates, the ultimate growth of the facility area will include:

Residential & commercial land uses -	25,000 Acres
Industrial	7,300 Acres
Open Space (includes forest preserve, cemeteries & municipal parks)	9,400 Acres

The growth trends have shown vacant land developed to residential, commercial and industrial uses. Residential growth trends indicate multifamily residential units becoming more prevalent than the past predominantly single-family home suburban-type development.

Air and water quality may be threatened by the trend in land use changes which include more people, cars, and construction of homes, offices, industrial plants and shopping centers. The availability of vacant land is not the only criteria for future development. Several open space agencies exist within the service area (for example local Park Districts) which are authorized to acquire lands for park and recreation purposes. These agencies contribute to the overall environmental improvement by preserving lands for recreational and environmental educational uses. The trend toward open space preservation should be included in land use alternatives considered in the various plans prepared by local agencies.

Comprehensive planning is the process by which a public planning agency provides for orderly development of an area and promotes a desirable environment. By this process, physical development is coordinated in accordance with present and future needs.

Plans and programs usually include a land use plan, a thoroughfare plan, a common facilities plan and public improvements program. Administrative and regulative measures to control and guide physical development according to the plans include a zoning ordinance, an official map and subdivision regulations.

A land use plan shows the location and extent of lands designated for various kinds of residential, institutional, commercial, industrial and public purposes. Current land use planning within the service area is being carried on by a variety of governmental units.

The "Summary of Local Planning Documents in Illinois," prepared by the State of Illinois Department of Transportation (1973), lists the following plans:

Arlington Heights	Comprehensive Village Plan (revised 1967) Preliminary Planning Report 1968
DesPlaines	Comprehensive Plan 1971, Zoning Ordinance 1971
Elk Grove Village	Comprehensive Plan, 1967
Mount Prospect	Comprehensive Plan, 1968
Rolling Meadows	Subdivision Control Ordinance (amended 1964)
Wheeling	General Development Plan, 1965

The Cook County Zoning Board of Appeals is currently preparing a new zoning map and zoning ordinance. Additionally, the county has a traffic safety study in progress.

9. Sensivite Areas

No properties included in or eligible for inclusion in the National Register of Historic Places are in the area of the tunnel system. No rare or endangered species, from either State of National lists, are known to occur in this area. The major open space area is the Ned Brown Forest Preserve. It is important both as a biological and recreational resource. Tunnels will be constructed under parkland in Des Plaines and Mount Prospect. About 1.4 acres of parkland would be affected by construction of the dropshafts and access manholes.

10. Population Projections and Economic Forecasts

The projected population forecast of the Northeastern Illinois Planning Commission (NIPC) is shown in graphical form in Figure 2-6, and in Table 2-2. The present population in the O'Hare Service Area is approximately 250,000. The projected population for the design year of 2000 is 300,000.

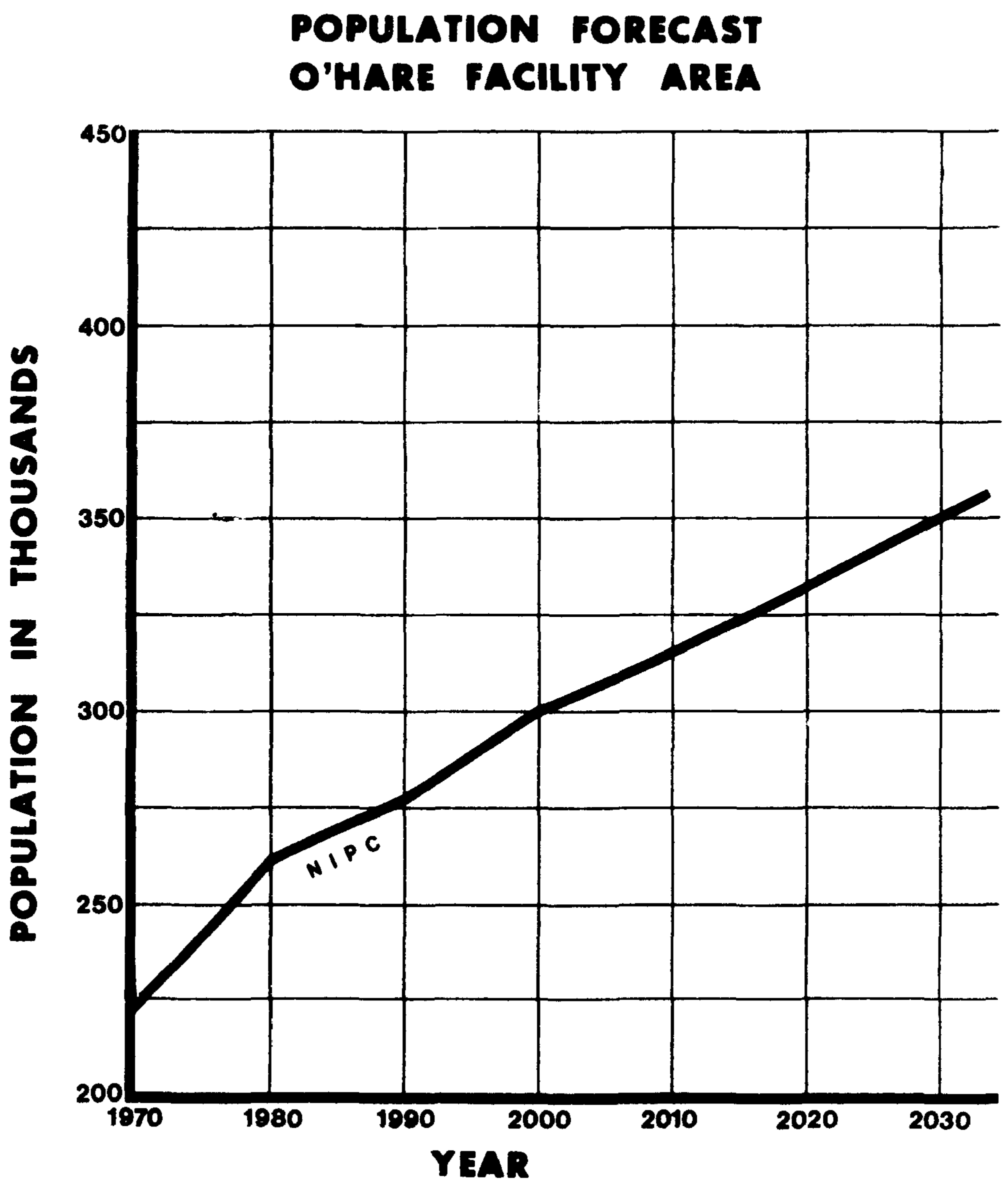


FIGURE 2-6

POPULATION FORECASTS

Table 2-2. Population forecasts for the O'Hare Service Area.
(Source: Northeastern Illinois Planning Commission)

YEAR	Forecast Population
1976	223,000
1980	261,000
1990	277,000
2000	300,000

Economic forecasts available are limited to projections of employment by townships prepared by NIPC. The three townships principally in the O'Hare Service Area are Elk Grove, Maine and Wheeling. The employment forecasts for these townships are shown in Table 2-3.

Table 2-3. Employment forecasts for the O'Hare Service Area
(Source: Northeastern Illinois Planning Commission)

Township	1970	1980	1990	2000
Elk Grove	37,257	43,400	46,300	47,100
Maine	52,767	68,600	74,300	75,800
Wheeling	24,916	31,200	34,300	34,700
TOTAL	114,940	143,200	154,900	157,600

11. Other Programs in the Area

New federal legislation entitled the "Housing & Community Development Act of 1974" provides the possibility of funding for community development activities. Within the service area, two communities, Arlington Heights and DesPlaines have populations greater than 50,000 and thus are eligible for their own "entitlement" moneys. Cook County would be eligible for funds as an "Urban County" under this act. Sewer construction is one eligible activity under the program. Future growth capacity could be stimulated by this federal program and ultimately serviced by the MSDGC.

CHAPTER 3

CONVEYANCE SYSTEM ALTERNATIVES

A. Project Objectives

1. The elimination of combined sewer overflows into Weller's Creek and Feehanville Ditch.
2. The conveyance of wastewater generated within the Upper Des Plaines Drainage Basin to the proposed Water Reclamation Plant (WRP) located in the service area.
3. The minimization of the environmental impacts of construction on the extensively developed residential areas in the Upper Des Plaines Drainage Basin.

B. Constraints

There were three principal constraints on the selection and design of alternative systems:

1. The Weller's Creek and Feehanville Ditch Drainage Basins are extensively developed, primarily in residential uses sensitive to disruption.
2. The 29 outfalls in the study area will be, by December 1977, in violation of the Illinois Environmental Protection Agency (IEPA) water pollution regulations adopted by the Illinois Pollution Control Board in July 1973 and approved by USEPA.
3. For the project to be economically feasible, routings between a treatment plant and the overflow points should be as direct as possible.

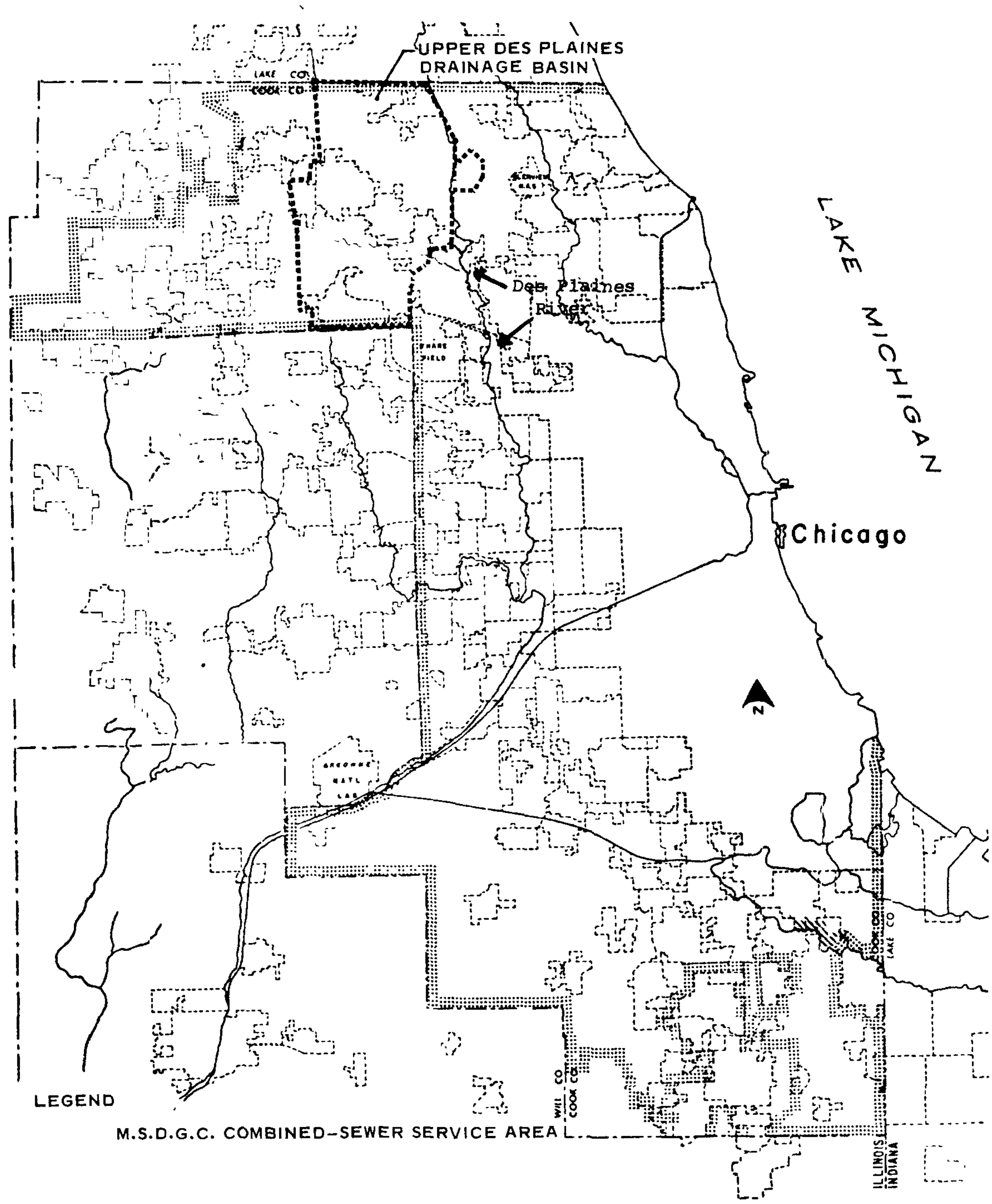


FIGURE 3-1

METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO GENERAL SERVICE AREA

C. Chronology of Plans and Studies

The Metropolitan Sanitary District of Greater Chicago is divided into eight service basins. Of particular interest in this statement is the Upper Des Plaines Drainage Basin,delineated in Figure 3-1. Collection and treatment of sewage generated in the Upper Des Plaines Basin has been the subject of many studies and reports. Based on a report by Greeley and Hansen submitted in 1962, a tentative decision was made to convey all sewage from the area to the West-Southwest treatment works at Stickney. Following the Greeley and Hansen 1962 report, additional studies and investigations, carried out primarily because of the trend toward higher standards for disposal of treated effluent, have indicated the advisability of collecting and treating the sewage from each drainage area separately. The policy of separation of drainage areas has been adopted by the MSDGC and four treatment works are planned for the Northwest Area. The four systems have been designated as O'Hare (Upper Des Plaines), Salt Creek, Hanover Park and Poplar Creek.

A preliminary design concept for the O'Hare Water Reclamation Plant, and an estimate of cost of both the intercepting sewers and the reclamation facilities was prepared in report form for the MSDGC by Brown and Caldwell, Consulting Engineers, dated June 1968. Preliminary plans for the intercepting sewers were prepared by De Leuw, Cather & Company, Consulting Engineers, in accordance with an agreement between that firm and the MSDGC.

D. Alternatives

There are three basic alternatives to the wastewater management problem within the Upper Des Plaines Drainage Basin. Given the need to provide additional treatment capacity, and the laws governing the discharge of combined sewage into surface waterways, there are: 1) Sewer separation, 2) Conventional interceptors, and 3) Interception and conveyance of combined sewage. Certain of these alternatives can be combined; in addition, the basin alternatives themselves have sub-alternatives within the context of their own plan.

1. Sewer Separation -- This alternative consists of the elimination of the combined sewer system throughout the Villages of Arlington Heights and Mount Prospect, and the City of Des Plaines.

   Detailed engineering and cost analyses have not been made for the the Upper Des Plaines Basin for this alternative; however, such analyses were made for Palatine, Illinois, a neighboring community of similar character. In the latter instance, two methods were investigated: construction of a new sanitary sewer in every street within the combined area and conversion of the existing combined sewer to a storm sewer; or, construction of new storm sewers and conversion of the existing facility to a sanitary sewer. In practice, there would be some waste in either method, since the existing sewers are generally larger than required by sanitary flows, and not large enough to accommodate storm flows of the magnitude required to eliminate flooding. Perhaps a more efficient solution may be conversion of the larger existing facilities to storm sewers to sanitary sewers.

The alternative would involve construction within every block of every street. Construction within any given block length would probably require several weeks, excluding construction time for the replacement of permanent pavement. Such a program could extend over a period of several years, depending upon the magnitude of the area the municipality chose to impact at one time.

2. Conventional Intercepting Sewer-- Two conventional systems were examined:

   a. A system of conduits which would intercept the existing MSDGC intercepting sewers within the Upper Des Plaines Basin and direct the sanitary flows to the proposed Water Reclamation Plant. Two subalternatives were developed which differ only in alignment. The least costly scheme would have a total project cost of approximately $24.2 million (1972 dollars).

   b. Conventional intercepting sewers, together with sewer separation.

3. Collection and Conveyance of Combined Overflows -- This alternative consists of collecting overflows from the existing outfalls, and conveying combined sewage to the proposed Water Reclamation Plant for treatment prior to discharge to the waterway. The system would consist of large diameter rock tunnels, together with necessary appurtenant structures to connect the existing combined outfalls and redirect the combined overflows to the rock tunnel system.

4. A no action alternative was also considered for purpose of evaluation. This alternative consists of simply doing nothing with regard to combined sewer overflows into Weller's Creek and Feehanville Ditch or diversion of flows to the proposed O'Hare Water Reclamation Plant. Impacts on water quality of Weller's Creek as a result of no action are discussed in Chapters 2 and 5. All sanitary sewage generated within the basin would continue to flow thru interceptors to North Side Sewage Treatment Works.

E. Comparative Analysis of Alternatives

To evaluate the various options available for wastewater management within the Upper Des Plaines Basin, six alternatives were assessed in relation to project objectives (See Table 3-1).

Table 3-1 Comparison of Alternatives and Project Objectives

		Project Objective		
	Alternative No.	Eliminate Combined Overflows	Conveyance to Plant	Minimize Environmental Impacts of Construction
1.	Sewer Separation	yes	no	no
2A.	Conventional Interceptors	no	yes	yes
2B.	Separation with	yes	yes	no
3A.	Collection and Conveyance of Combined Overflows	*yes	yes	yes
3B.	Collection, Conveyance and Storage of Combined Overflows	yes	yes	yes
4.	No Action	no	no	yes

* Partial

Two alternatives--separation with interceptors (Alt. 2B) and collection, conveyance and storage (Alt. 3B) would meet the first two project objectives. In addition, the collection/conveyance alternative (Alt. 3A) would meet the second objective and greatly reduce overflows. This alternative partially meets objective one and would result in a 72 percent reduction in total volume of spills, a 93 percent reduction in number of spills and a reduction in the number of average yearly spills from approximately 80 to six.

In the selection of alternatives for further screening, only those which were at least in part responsive to project objectives were carried forward.

F. Final Systems Screening

Three alternatives were selected for more detailed analysis. These were:

* Separation with interceptors.
* Collection and conveyance of combined overflows.
* Collection, conveyance and storage of combined overflows. The proposed Water Reclamation Plant is designed to handle peak dry weather flows only and not the peak wet weather flows from combined sewer areas. MSDGC plants are normally operated at full capacity before and after storms to minimize the overflows of untreated flow. It is not cost-effective or feasible to increase the plant peaking capacity to match the rate of storm runoff, which for the 11.4 square mile combined sewer area may reach over 30 times the dry weather flow for the entire 58.2 square mile basin.

1. Separation with Interceptors Alternative

Of the 7,322 acres of combined area in the Upper Des Plaines Basin, 1,370 have separate local sewer systems, which are recombined at the dowstream end with the combined system, leaving 5,952 acres which would require new separated sewer systems. A detailed analysis of Palatine, Illinois indicates that the cost of a new separate sanitary sewer system would be approximately $12,700 per acre. For the O'Hare Service Area this results in an estimated separation cost of $75.6 million. In addition, if separation were accomplished, conventional interceptors would still need to be constructed at an estimated cost of $23.2 million for a total system cost of $98.8 million.

While this alternative would achieve the objective of elimination of combined sewer overflows, it would have other severe environmental implications. The amount of surface disturbance necessary would be extensive. Even with considerable construction safeguards, there would still be a significant quantity of erosion and runoff due to the construction and separation of the sewer systems. In addition, the quantity of material resources necessary for this alternative would be significant.

2. Collection, Conveyance and Storage of Combined Overflows Alternative

A detailed engineering analysis was made of tunnel and reservoir alternatives. A report titled "Preliminary Plans for O'Hare Collection Facility," dated November 1972, presents a summary of this analysis and tunnel and reservoir concept plans. The estimate for the preferred tunnel and reservoir scheme selected for the total basin is $88.5 million. This figure is established from Chapter XI, of the De Leuw, Cather report by adding the estimated land cost to the estimated construction cost (including 20 percent for contingencies) and deducting the $18.8 million estimated cost of the Palatine area projects. (Palatine area projects are no longer planned to be interconnected to the O'Hare tunnel conveyance system).

It has been estimated that the average yearly volume of combined sewage intercepted by the tunnels and reservoir would be 6670 acre-feet. This is equivalent to a runoff of approximately eleven inches over entire combined sewer area in the basin. This 6670 acre-feet is equivalent to a yearly volume of 2.17 billion gallons or 5.95 MGD average additional flow to the plant.

The treatment cost for the additional combined over-flows intercepted by the tunnel and reservoir concept projects may be estimated as follows:

	Yearly Cost	Total Cost
Additional flow due to interception of combined sewage flows 5.95 MGD	$155/MG times 365 days/year times 5.95 mgd= $336,621/year times (Present Worth Factor of 6%/50 years 15.762) =	$5,305,820

Therefore a cost comparison for separation versus a tunnel and reservoir plan by only $5.0 million, the following factors should be considered:

a. The maintenance and operation cost estimated at $144/MGD is very conservative for combined sewage treatment. It is based on the MSDGC surcharge ordinance which relates to the treatment of sanitary sewage and industrial wastes.

b. The assumption of a 50-year maintenance and operation cost is conservative.

c. The cost of inconvenience to the public by excavation in every street in the combined sewered area, as would be required in the sewer separation alternative, is not reflected in the cost comparison.

d. The cost of required replumbing for buildings in the combined sewered area is not included in the $12,700 per acre cost used for sewer separation.

e. The monetary value of pollution reduction by the treatment of polluted urban runoff is not reflected in the cost comparison.

3. Collection and Conveyance of Combined Overflows Alternative

The MSDGC has opted not to construct the reservoir portion of the tunnel and reservoir plan at this time. Such a decision results in a system of rock tunnels identical in structure to those of the tunnel and reservoir plan. This option is referred to as the Rock Tunnel Alternative. All flow characteristics, connections, locations and sizes would remain the same as those under the tunnel and reservoir plan, but the storage volume would be reduced to that of the volume of the tunnels.

The completed Rock Tunnel Alternative system will provide approximately 200 acre-feet of storage for combined sewage capture. This is sufficient volume to contain approximately 1/3 inch of runoff from the entire 7322 - acre combined sewer area. Using a combined runoff factor of 45 percent, this is equivalent to the runoff from a 3/4 inch rainfall over the Upper Des Plaines combined sewer area.

Computer model studies indicate that approximately 72 percent of the volume of all combined sewer discharges to Weller's Creek and Feehanville Ditch would no longer occur on completion of the tunnel system. This would reduce the biochemical oxygen demand (BOD) and suspended solids discharged by approximately 92 percent and 93 percent, respectively. Cost of a system of collection and conveyance rock tunnels including all appurtenant structures to provide an operating facility is estimated to be $36.5 million (1972 present worth).

The Rock Tunnel Alternative is described in greater detail in Chapter 4 and assessed in Chapter 5 for comparative purposes with the no action alternative.

CHAPTER 4

DESCRIPTION OF THE PROPOSED ACTION

The proposed action consists of a network of tunnels and shafts ranging in diameter from 5 to 20 feet. A majoritv of the tunnels (6.63 miles) would be located approximately 160 feet below the ground surface in rock. The remaining 3.22 miles of earth tunnel would be at depths approximately 40 feet below the surface. See Figure 4-1.

This network is designed to collect all combined sewer overflows within the project area and direct them together with sanitary flows, to the south end of the tunnel system. A description of each contract of the proposed project is given in Chapter 1. All earth tunnels will be lined with 12 inches of concrete and all rock tunnels with 10 inches of concrete.

A. Main Tunnel

The main rock tunnel would be 20 feet in diameter and run north along Elmhurst Road from a main shaft located approximately 400 feet southwest of the intersection of Elmhurst Road and Northwest Tollway to Drop Shaft 4. From Drop Shaft 4 the main tunnel would proceed northwest along Weller's Creek to Drop Shaft 1 located approximately 400 feet north of the intersection of Central Road and Weller's Creek.

B. Branch Tunnels

The east branch tunnel would be 16 feet in diameter. It would begin at a junction with the main rock tunnel at the intersection of Elmhurst Road and Lonnquist Boulevard, and proceed east in Lonnquist Boulevard to Drop Shaft 6 at Williams Street, then turn north and follow Williams Street to Drop Shaft 8 located approximately 200 feet northeast of the intersection

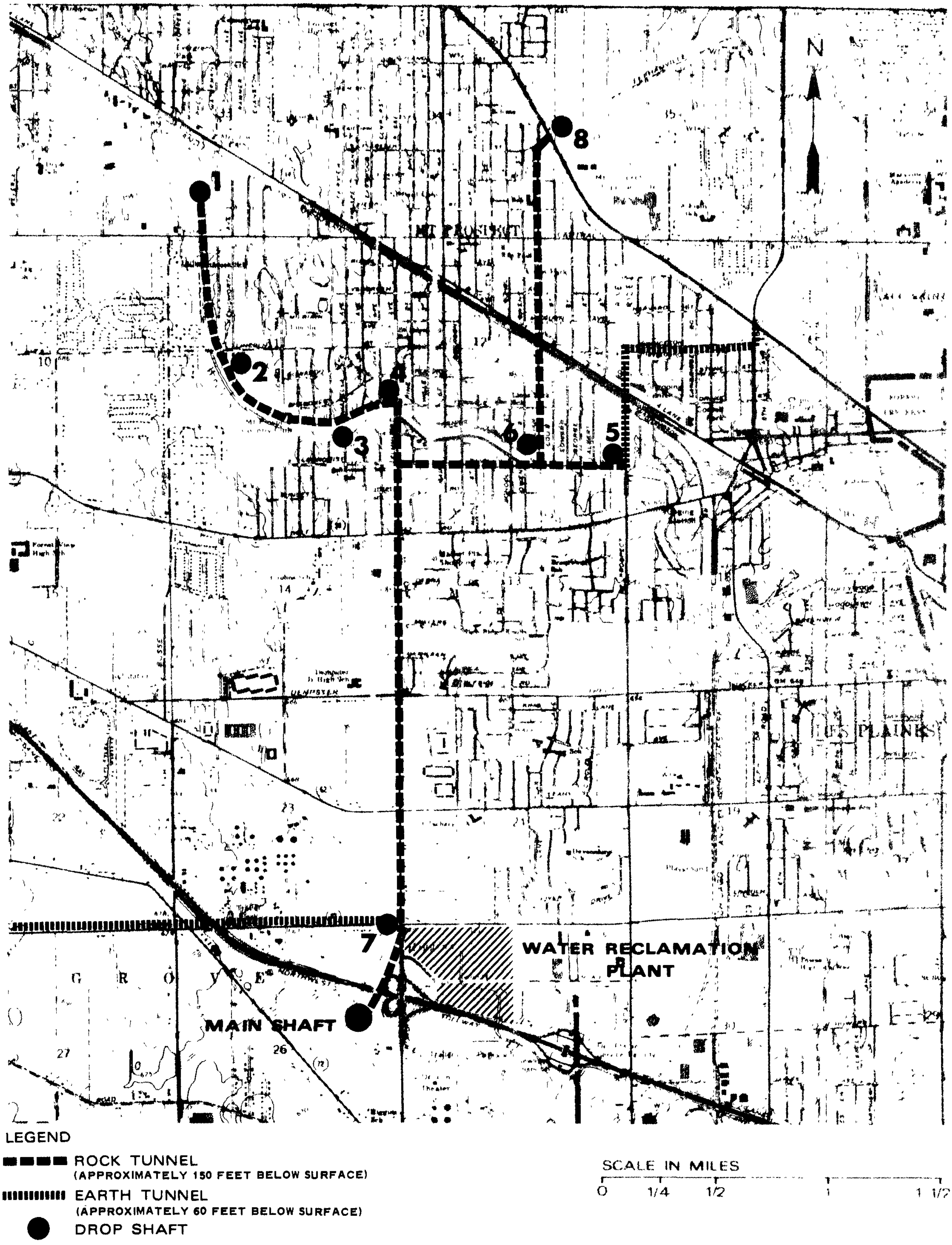


FIGURE 4-1

ROCK TUNNEL ALTERNATIVE

of Isabella Street and Rand Road. A nine-foot rock tunnel would junction with the east branch at Lonnquist Boulevard and Williams Street, and proceed east along Weller's Creek to Crop Shaft 5 located at Weller's Creek and Mt. Prospect Road.

A five-foot earth tunnel, to be lined with concrete, would begin at Drop Shaft 5 and proceed north on Mr. Prospect Road to Princeton Street, then east in Princeton to Wolf Road and north in Wolf Road to intersection of Rand Road and Wolf Road.

Beginning at Drop Shaft 7 a five-foot concrete-lined earth tunnel would extend west under Oakton Street approximately 2.08 miles to the intersection of Oakton Street and Wildwood Road.

C. Sequencing of Tunnel Construction

It is anticipated that construction of the 4 tunnel contracts will commence on approximately the same date. The connection and laterals contract, U.D. 20A (Contract 73-318-2S) is expected to be awarded approximately one year later.

Tunnels are generally excavated in an upgrade direction since this facilitates dewatering and muck removal. However, for the rock tunnel project it is anticipated that spoil removal, specified shaft location and the magnitude of the projects will be a more important consideration.

1. U.D. 20 (Contract 73-317-2S)

Tunnelling will commence at the main shaft at the southwest corner of the Northwest Tollway and Elmhurst Road and proceed northerly and westerly to the site of Drop Shaft No. 1 located at Central Road east of Busse Road

2. U.D. 21 (Contract 73-320-2S)

Tunnelling for the 16 foot diameter tunnel will commence at the site of Drop Shaft 8 at Rank Road and Isabella Street and proceed, reverse grade, southerly and westerly to a junction with U.D. 20 at Elmhurst Road and Lonnquist Boulevard.

Tunnelling for the 9 foot diameter tunnel will commence at the location of Drop Shaft No. 5 at Mt. Prospect Road and Lonnquist Boulevard extended and extend westerly to the junction with the 16' diameter tunnel at Lonnquist Boulevard and Williams Street.

3. U.D. 20B (Contract 73-319-2S)

Tunnelling will commence at Lonnquist Boulevard (extended) and Mt. Prospect Road and proceed northerly and easterly to Wolf and Rand Roads.

4. U.D. 20C (Contract 69-307-2S)

Tunnelling will commence at the location of Drop Shaft 7 at Elmhurst Road and Oakton Street and proceed westerly to Oakton Street and Wildwood Road.

The tunnel water detention basin required for U.D. 20 will be located immediately west of the main shaft. Water infiltrating into the tunnels will drain by gravity to the low point in the tunnel system at the location of the junction with the 7-foot diameter plant influent tunnel. From this point it will be pumped out through the main shaft to the detention basin which will be pumped or drained by gravity to Higgins Creek immediately west of the site of the Main shaft.

Excavated material will be removed from the tunnels at main shaft sites. Transportation in the tunnels will be by muck cars traveling on rail tracks laid in the tunnels. The material will be removed from the tunnels by a crane or elevator hoisting system. It will then be deposited near the shaft at a temporary storage location or hauled immediately from the site to its ultimate user. Market conditions and available storage space will dictate the amount and time of storage. The MSDGC owns approximately 112.5 acres adjacent to the main shaft, part of which will be made available for spoil storage.

The material excavated will be composed of spalled or laterally split rock of small dimensions with a large percentage of fines. As this material does not have a gradation conforming to accepted specifications for concrete aggregate or roadway base material, it is not generally acceptable for these purposes. However it has been used in private developments for such things as stone base for parking lots. The primary use of this material is expected to be as land fill.

D. Main Shaft and Drop Shafts

The main shaft is the location for lowering and removing the mining machines. Men, equipment and material will enter and exit from the tunnel system from this shaft during construction. Dewatering during construction will take place at this location.

After construction, the main shaft will serve as part of the exit structure from the tunnel system should a reservoir eventually be constructed at its terminus. The main shaft is in fact located on the proposed site for a main reservoir which was proposed by DeLeuw, Cather and Company in the preliminary plans. After construction the main shaft, having a finished internal diameter of 16 feet, will be capable of being used for lowering a maintenance vehicle into the tunnel system.

A drop pipe will be provided at the Main Shaft to extend service to the portion of the service basin area south of the Tollway.

Drop Shafts

Shaft Number	Location
1	Approximately 400 feet north of the intersection of Central Road and Weller's Creek.
2	At the intersection of Weller's Creek and Lincoln Street.
3	Within the park along Weller's Creek approximately opposite Wa-Pella Avenue.
4	At the intersection of Weller's Creek and Elmhurst Road.
5	At the intersection of Weller's Creek and Mt. Prospect Road.
6	At the intersection of Williams Street and Lonnquist Boulevard.
7	At the intersection of Elmhurst Road and Oakton Street.
8	Approximately 200 feet northeast of the intersection of Isabella Street and Rand Road.

The shafts would require excavation approximately 160 feet deep into the overburden soils and rock. These excavations would require temporary sheeting and bracing to support the adjacent earth until the permanent structures are constructed and backfill work is completed. Blasting would be required for excavation of the rock portions. This blasting would continue for approximately one month at each shaft and be limited to one blast every two or three days. During the construction of the shafts the work areas would be fenced and secured in accordance with MSDGC standard practices. The shafts would vary in size from five to nine feet in internal diameter. Construction would proceed from the surface downward to the tunnel level.

A sectional view of the Type D-4 drop shaft to be used in the Upper Des Plaines Tunnel Conveyance System is shown in Figure 4-2. It is noted that the air insufflated into the water on its way down the shaft in the downcomer is released in the separation chamber. This air rises in the vent shaft and becomes reinsufflated into the incoming sewage at the top of the downcomer. There is, therefore, no net movement of air out of the drop shafts. During model studies the vent shaft openings to the atmosphere at the top of the structures were sealed during some of the experiments, thereby precluding the possibility of air movement out of the shafts. This did not affect operation of the drop shaft.

While the aspect of aerosol release at the top of the vent shaft or downcomer was not specifically studied in the model study program, MSDGC observed that this phenomenon did not appear to be occurring. In the event that the prototype drop shaft does operate differently from the models and the phenomenon of air movement out of the shafts does occur the openings to the surface of the ground could be sealed without hydraulically affecting the structures.

Sluice gates will be placed in collection structures at the location of all major sources of storm water inflow into the tunnel system. These gates will be provided with an alternate power supply or other means to insure their operation when required. The gate status will be continuously

DROPSHAFT D-4 TYPE
UPPER DES PLAINES TUNNEL PLAN

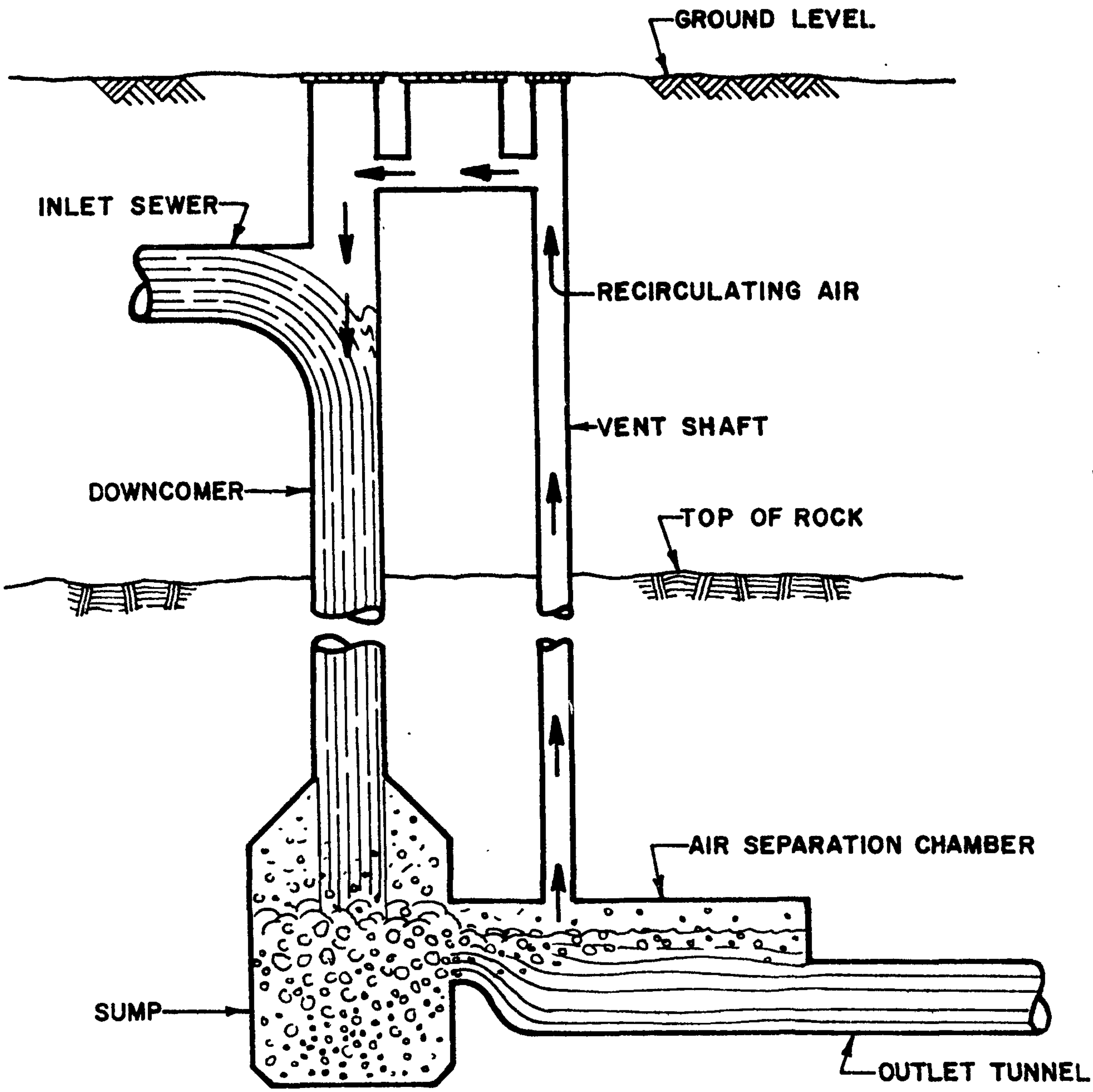


FIGURE 4-2

THE METROPOLITAN SANITARY DISTRICT
OF GREATER CHICAGO

ENGINEERING DEPARTMENT

JAN., 1975

monitored (visually and be computer) at the proposed O'Hare Water Reclamation Plant. In the proposed tunnel conveyance system the gates will be closed as necessary during major storm events to prevent filling of the rock level tunnel system above a level higher than the crown at the upstream ends of these tunnels. This will prevent rapid filling of the drop shafts which might result in hydraulic surges. In addition by keeping the rock level tunnels from being surcharged potential backup of local sewer systems will be prevented.

E. Access Manholes

Access manholes would be constructed at approximately 2,000-foot spacing along the work tunnel alignments. Under current MSDGC practices manholes are placed at approximately 600-foot spacing along tunnel constructed through earth. These manholes would be constructed from the surface downward in a manner similar to the drop shafts; however, they would require a much smaller excavation. Some blasting would also be required in the excavation of some of these elements. Access manholes would be located within or adjacent to the parkway or shoulder area of public roadways. The construction period for an access manhole is approximately three months.

F. Relationship of Proposed Action to Existing Facilities and Other Projects.

The physical relationships between the existing combined sewer systems and the proposed conveyance system are exhibited in the preliminary plans for the project. (Preliminary Plans for O'Hare Collection Facility, November 1972, De Leuw, Cather & Company)

These preliminary plans illustrate that under the ultimate plan (reservoir included) sufficient storage capacity will be available with the facility to permit plugging the smaller combined sewer overflows. The larger interceptor outfalls are maintained to serve as safety valves for the system. Under the proposed plan, all existing overflows will be maintained due to limited storage capacity. The volume of the proposed conveyance tunnels is approximately 220 acre-feet.

The existing combined sewer systems have been estimated to be slightly less than capable of handling a storm of 5-year return frequency. This evaluation is a generalization, however, since some subsystems very likely have a capacity of even less than this. The proposed tunnel facilities were planned to provide a flow-through capacity sufficient to handle the "storm of record" without creating surcharging in the existing combined sewer system. The existing systems do not have the capacity to handle the storm of record volume, and accordingly provisions are made in the design of the proposed facility for future relief of the existing system by the local municipalities.

The total mass discharge from all combined sewer outfalls within the project area for the 21 years used in the study analysis is 140,000 acre-feet. This arithmetically averages to 1,867 acre-feet per year being discharged to the waterways after the facility is in operation. These remaining overflows will continue to occur at the existing outfalls.

Tunnel maintenance, when required, must be done by entering the tunnels and performing the work in accordance with procedures common within the industry. No techniques or equipment not presently available are expected to be required. In the event the tunnels must be entered, all inflow and control gates must be closed, and dry weather flow diverted to the North side plant for processing. Gates are provided at all inflow points to the proposed tunnel system.

The design of the tunnel system of the Upper Des Plaines Tunnel Plan does not depend on or interact with the design of small, local storm water retention basins, whether existing or to be constructed by the Villages of Mount Prospect and Arlington Heights. The Upper Des Plaines Tunnel and Reservoir Plan contemplates two reservoirs, the Main or Upper Des Plaines Storage Reservoir, possibly to be located on a site at the southwest corner of the Northwest Tollway and Elmhurst Road adjacent to the Main Shaft, and the Mount Prospect Detention Basin to be constructed adjacent to Drop Shaft No. 1 at Central Road east of Busse Road.

It is noted that the Main Reservoir and Mount Prospect Detention Reservoir would fill only irregularly. Reference to the DeLeuw, Cather and Company Report, (Figure XI-3 on Page X-8), indicates that in the 21 years of records studied, the computer simulation study indicated that the detention basin would detain more than 100 acre-feet only 8 times, and more than 300 acre-feet four times. Further, the reservoir remains completely dry except on 24 occasions in the 21 years of record, or approximately once a year. The volume of the detention basin as proposed in the DeLeuw, Cather and Company Report is 850 acre-feet. Under the simulated condition, when the Mount Prospect Basin filled to the greatest extent (810 acre-feet) during the July, 1957 storm, the basin was empty 19 hours after it began to be filled. (This is seen by examination of Table XI-2 on Page XI-5 of the DeLeuw, Cather Report.)

The Main Reservoir, on the other hand, is a retention basin. As the dewatering rate (24 MGD) is very small relative to the inflow rate during storms, its size is essentially equal to the volume of runoff in the combined sewer area. Therefore the size of the Main Reservoir is almost totally independent of the rate of runoff or presence of local retention basins in the combined sewer area.

The MSDGC is in the process of entering into an agreement with Mount Prospect and Arlington Heights for the construction of a 130 acre-feet storm water retention reservoir on a portion of the site of the proposed Mount Prospect Reservoir. This reservoir could be a shallow gravity type, discharging storm water at a reduced rate to Weller's Creek. The construction of this storm water reservoir would be made possible by sewer separation within the tributary area, to be performed by the Villages of Mount Prospect and Arlington Heights.

If the MSDGC proceeds with construction of the Combined Sewer Detention Reservoir at the Mount Prospect site at a later date, the separate storm water reservoir could be abandoned and its volume used for the larger combined wastewater reservoir. However, based on feasibility studies it is possible that it may prove feasible to have two reservoirs on the same site, a storm water reservoir and a combined wastewater detention basin, thereby not recombining the storm water but continuing to discharge it to Weller's Creek without treatment.

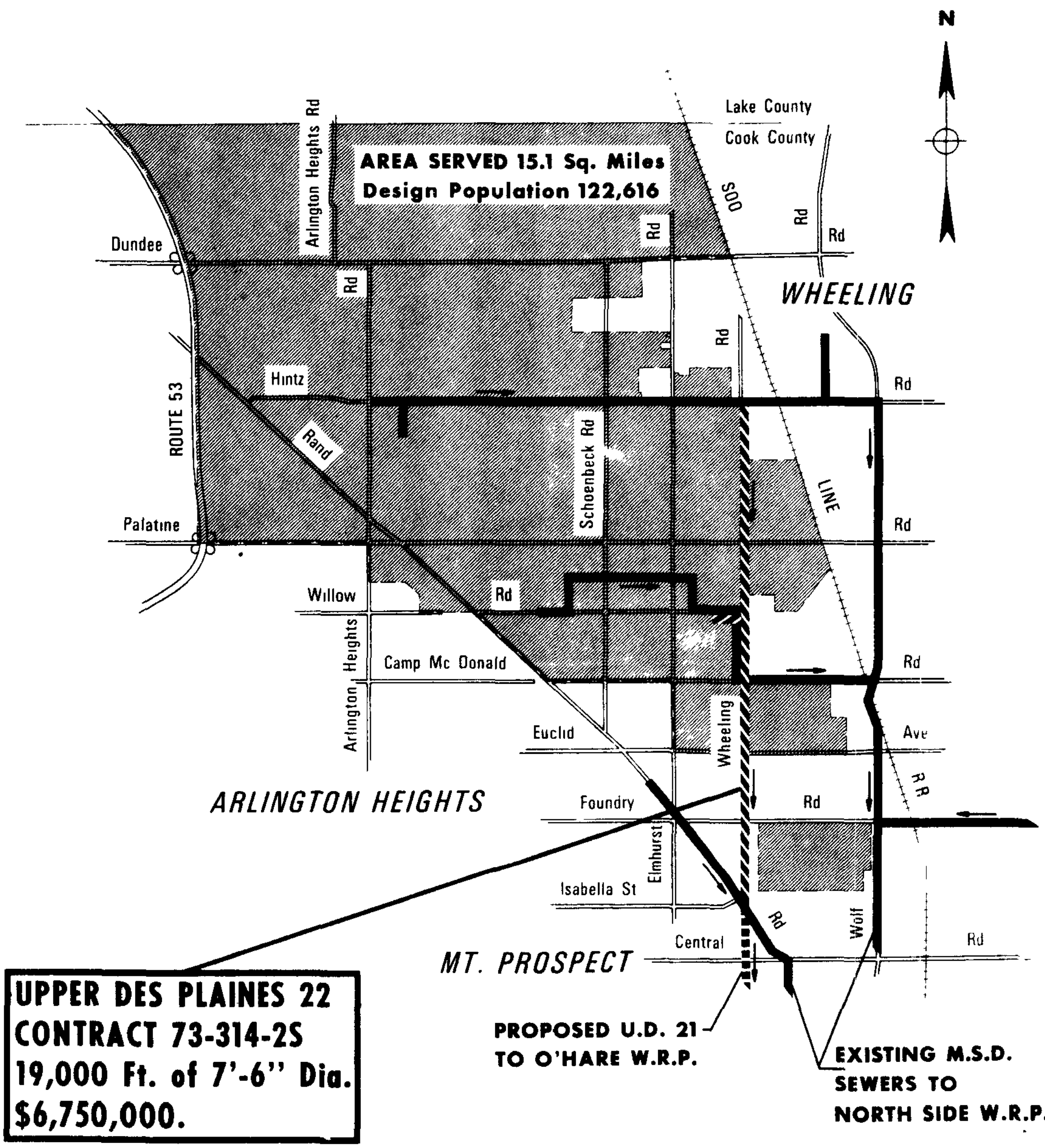
At the present time the feasible alternate has not been determined but this will be established prior to constructing the storm water reservoir. At this time the MSDGC does not plan to treat storm water except where this is the most economical manner of reducing combined wastewater-caused pollution of waterways.

The only sewer tunnel and/or combined waste water reservoir projects which may be implemented in the next 20 years in the Upper Des Plaines Service Basin, are the O'Hare (Main) Reservoir, the Mount Prospect Detention Basin and the Upper Des Plaines Intercepting Sewer 22 (Contract 73-314-2S). This intercepting sewer will be an earth tunnel which will serve a separate sanitary sewer service area and will relieve the existing Upper Des Plaines 14A and 14B interceptors in Wolf Road. The service area, size, location and other information related to this intercepting sewer are shown on Figure 4.3.

No determination of the desirability of constructing the combined waste water reservoir(s) or additional sewer line has been made in this EIS. Possible future connections are reported for the purpose of identifying some options that are available through implementation of the proposed tunnel conveyance system.

The selection of the tunnel conveyance system as the proposed sewer intercepting system for the O'Hare Service Area does not predetermine the site of the Water Reclamation Plant.

The location of the dropshafts, earth and rock tunnels are a function of the existing sewers (especially those serving the combined sewer area), hydraulic design parameters, and ease of construction and operation and maintenance. Nine possible sites are considered in the siting of the proposed Water Reclamation Plant (See the Draft EIS on the O'Hare WRP, Chapter 3). At each site a 7-foot diameter influent sewer would be needed to dewater the main 20-foot diameter rock tunnel. Thus, constructing the recommended tunnel conveyance system leaves open the site selection process for the Water Reclamation Plant.



THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

ENGINEERING DEPARTMENT

FIGURE 4-3 **MAY, 1973**

CHAPTER 5

ENVIRONMENTAL EFFECTS OF THE PROPOSED ACTION

Four key components have been identified for the proposed action. They are:

1) The tunnels

2) Eight drop shafts and one main shaft

3) Seventy access manholes

4) Nine monitoring wells

The proposed plan is thus composed of separate component parts, each of which may produce a given impact or degree of impact depending upon its location and size. In addition, the tunnel conveyance system would have both beneficial and adverse impacts on the area as a whole. These two levels of impacts combine to produce the overall impact that would be associated with the proposed tunnel conveyance system.

A matrix summary of potential impacts at the end of the chapter indicates the range and types of impacts involved in the proposed action.

A. Bedrock Geology

A large portion of the Rock Tunnel Alternative is located in the bedrock strata. The general geologic sequence to be expected along this tunnel alignment is illustrated in Figures 5-1 and 5-2. The geology as shown in these figures is greatly generalized and the location, number, attitude and condition of faults are totally interpretive and not verified by field observations. However, the formations involved with the Rock Tunnel Alternative are stable and the boring of the tunnel would not cause any

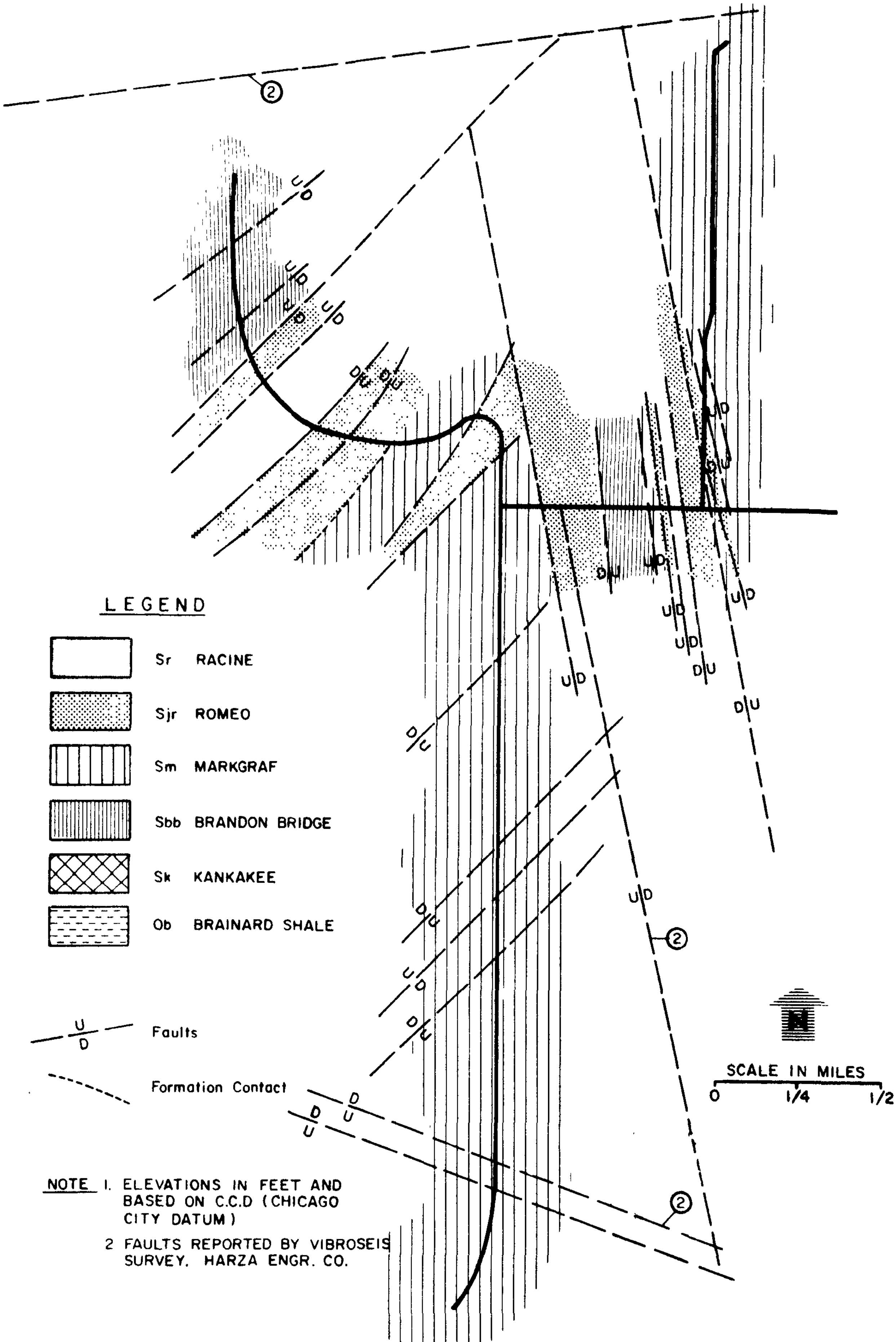


FIGURE 5-1

GEOLOGY AT TUNNEL ROOF

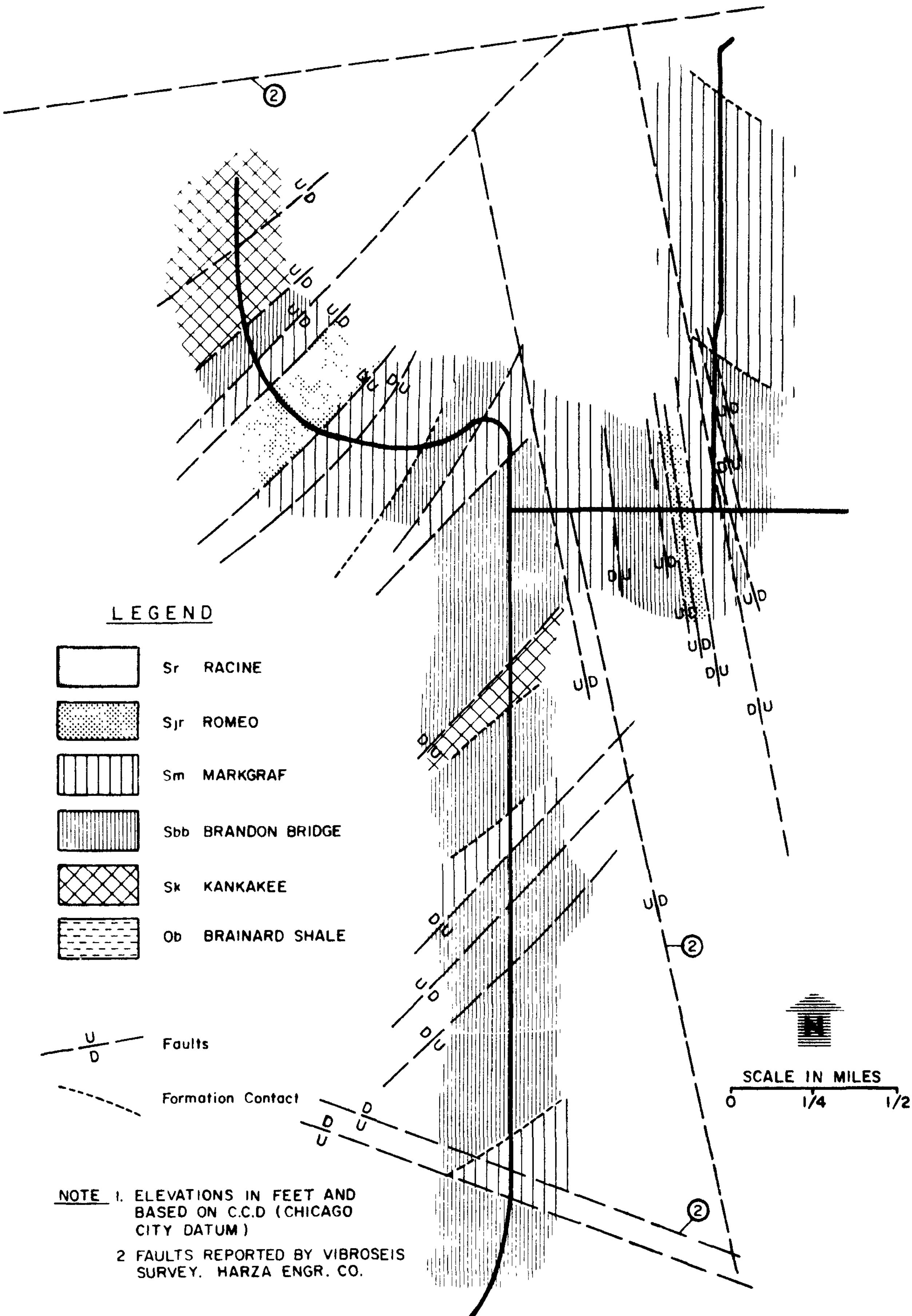


FIGURE 5-2

GEOLOGY OF TUNNEL INVERT

slipping or instabilities.

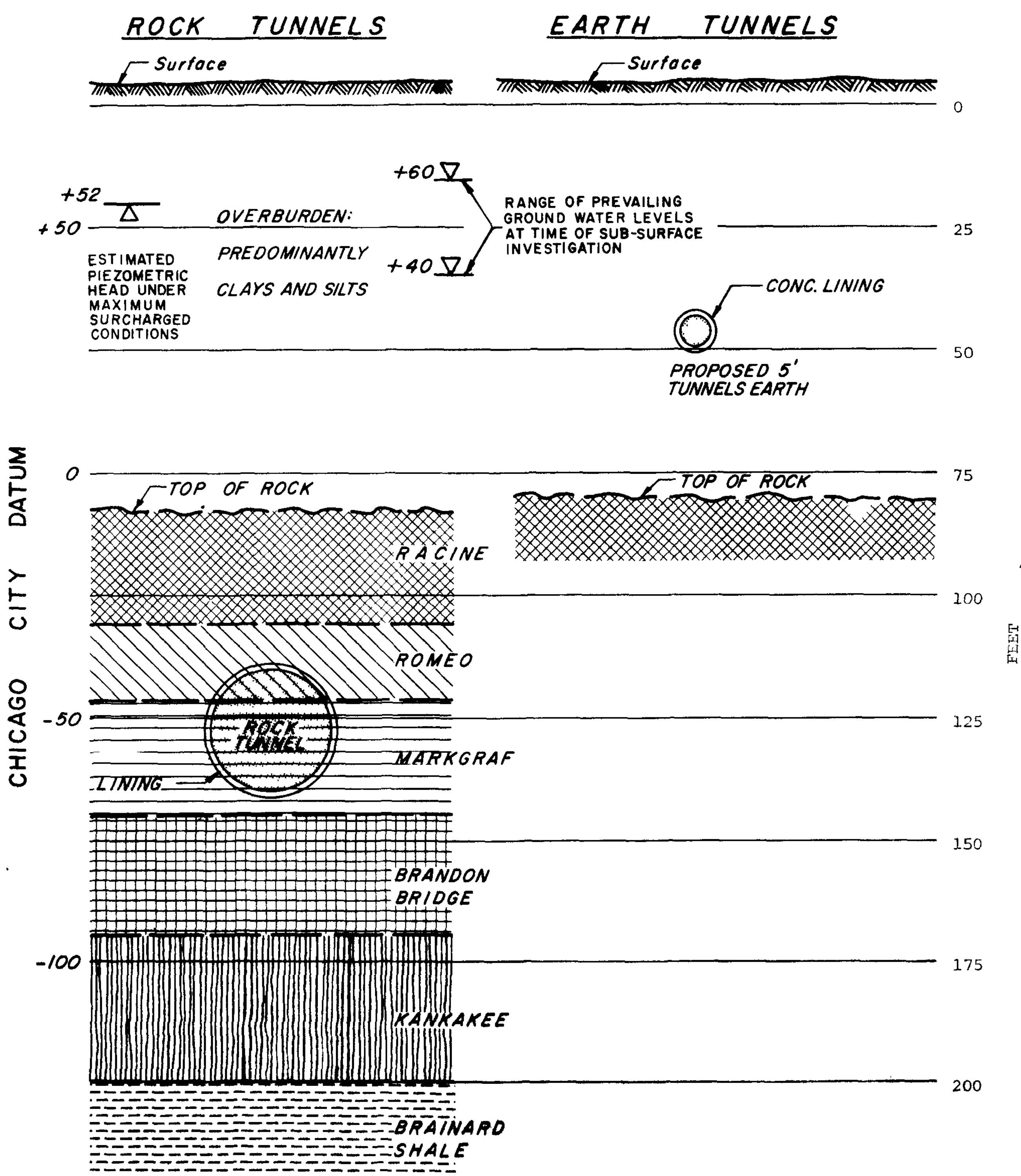
Due to numerous vertical joints and high permeability of formations, groundwater would seep into the tunnel during construction. Because the tunnel would be fully grouted and lined, infiltration would be reduced to a minimum after construction.

The linings for all rock tunnels will be 10-inch plain concrete with a minimum 28 day compressive strength of 4,000 psi. This concrete is the MSDGC standard for dense, watertight, durable concrete in contact with sewage. Linings for earth tunnels will be 12-inch thick concrete identical to that above, or precast concrete pipe depending on the Contractor's selection of method of construction.

In the case of the proposed rock tunnels, the lining serves the principal function of protecting mudstone partings, present in some of the rock members, from continuous contact with the flowing sewage. See Figure 5-3.

The lining, once in place, serves to further reduce potential infiltration and exfiltration by providing a continuous, virtually impervious barrier between the tunnel opening and surrounding rock. For this reason, it is believed that infiltration can be reduced below that accomplished with unlined tunnels.

In addition to the concrete lining, the adjacent rock formations will be grouted where necessary. This grouting is intended to seal openings in the rock through which water may migrate either into or out of the tunnel system. There is a practical limit with respect to opening size, with which cement grouting may be effectively employed. Results of a previous grouting program of a recently constructed facility indicate that openings with infiltration rates less than 1/8 gpm are practically ungroutable, using cement



GENERALIZED STRATIGRAPHIC SECTIONS

FIGURE 5-3

grout, with cost effective results. For this tunnel conveyance system lining and grouting are intended to augment each other.

During the construction period, however, such inflow may temporarily result in a lowered water table in the overlying glacial drift aquifer and dewatering pumpage may contribute to short-term water quality degradation (mainly turbidity) in Higgins Creek.

The proposed system would not be affected by the intensity of future earthquakes predicted to occur in this area.

Approximately 350,000 cubic yards of dolomite would be removed during a 2 to 3 1/2 year construction period estimated for various tunnel segments. These dolomites and the overlying glacial material represent a natural resource which should be utilized in a productive manner.

It is anticipated that the restricted usability of the tunnel spoil material will reduce its value to an extent where double handling is unwarranted. Therefore it is not expected that a high percentage of the material will be stored near the main shaft sites. However in the event of storage of the material it can be expected that it will have a lower permeability than the unpaved ground surface thereby increasing storm runoff. The minor increase in runoff and its short term nature minimize the significance of this negative impact. The rate of removal will be such that it is not expected to have any significant environmental or economic impact.

B. Soils and Surficial Geology

The proposed project will have no significant effect on the weathered surface soils of the project area, as the construction of the eight drop shafts and one main shaft represents the largest modification. There may be some soil compaction resulting from the operation of construction equipment in the vicinity of the drop shafts, manholes, and main shaft.

The earth tunnel portions of the rock tunnel plan would require removal of approximately 8,000 cubic yards of subsoil. The composition of this subsoil ranges from clayey silts to silty clays, with occasional sand lenses and some gravel.

The 12-inch concrete lining in the earth tunnel portion will protect the shallow aquifers present throughout these morainal deposits. Construction techniques are designed to prevent collapse of the earth tunnel and subsequent earth settling. These construction techniques have been utilized in the past and have proven successful.

C. Hydrology

1. Surface Water

Implementian of the proposed plan would result in the improvement of the water quality of Weller's Creek and Feehanville Ditch, as combined sewer overflows to these streams would be reduced. The 29 area outfalls are indicated on Figure 5-4. With the proposed tunnel system in operation, combined overflows to these streams would be reduced from 80 occurrences per year to fewer than six. This would respond to the IEPA Water Pollution Regulations requiring provision for treatment of all combined sewer overflows by December 31, 1977.

A dewatering program will be necessary during the construction of the rock tunnel, as a flow up to 600 gallons per minute could be produced. This water will be discharged into Higgins Creek after the water passes through a settling basin of sufficient size to permit a one-half hour detention period. The settling basin will be located near the main shaft. However, this water may increase the turbidity of Higgins Creek in the short term as the water

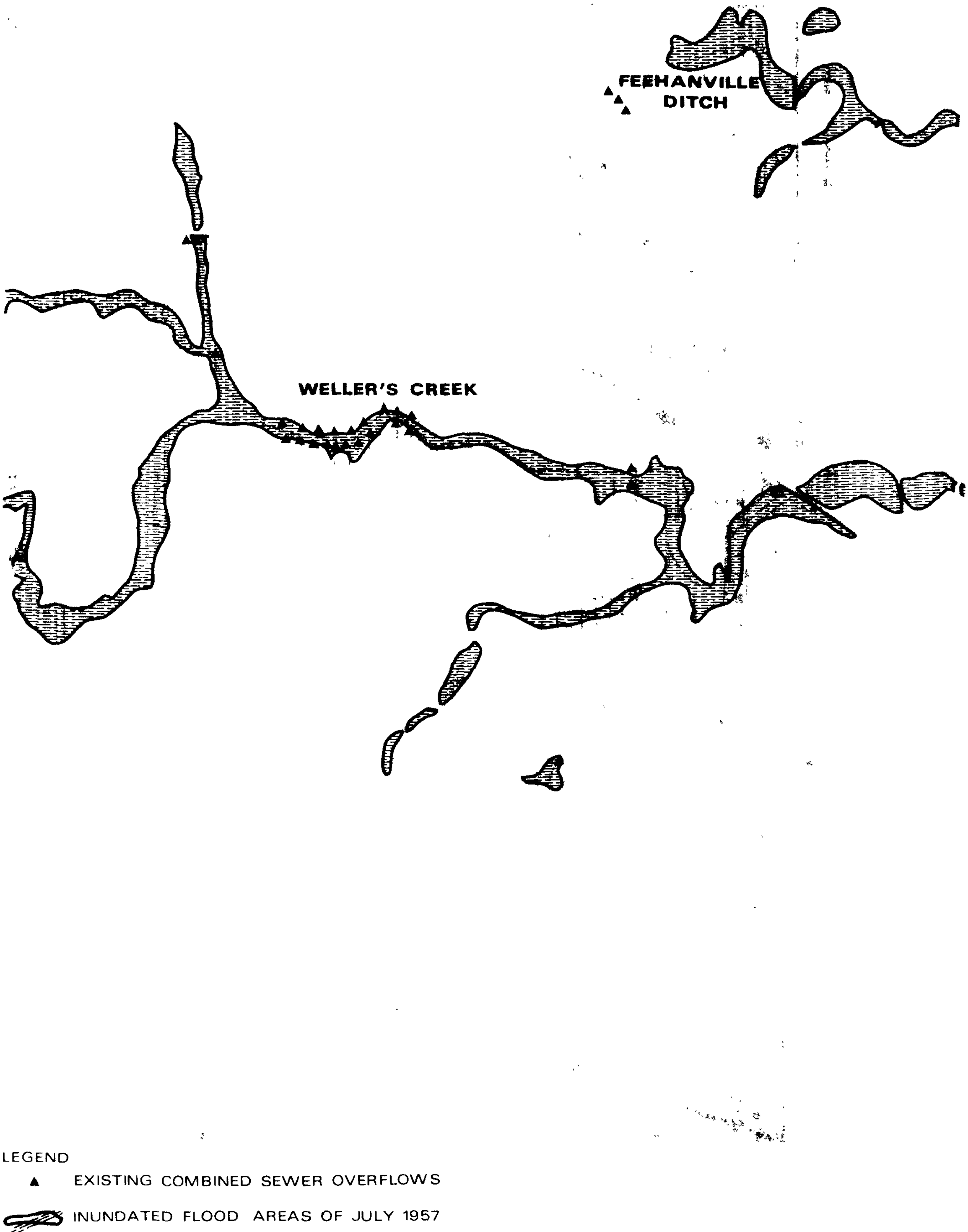


FIGURE 5-4

COMBINED SEWER OVERFLOW POINTS

will not be retained for a sufficient period to eliminate all of the fine limestone particles. A discussion of the impacts of the effluent from the Water Reclamation Plant is discussed in the EIS on the Water Reclamation plant.

2. Aquifers

The proposed modifications should have little effect upon the groundwater of the glacial material and those shallow aquifers in the Silurian dolomites, as all tunnels will be lined to prevent infiltration and exfiltration. However, during construction of the rock tunnels, there will be some loss of water in the Silurian aquifers due to the necessity of dewatering. There should be little water loss during the construction of the earth tunnels.

The MSDGC and its consultants have considered the general subject of interference with private and municipal wells by construction activities or by the completed Upper Des Plaines Tunnel Conveyance System.

The areas of study fall into two general categories -- effects during construction activities and effects after completion of construction. The subjects considered relating to construction operations are as follows:

a. Clouding or contamination of wells by tunnel or shaft construction operations;

b. Lowering of the water table during construction operations;

c. Clouding or contamination of wells by grouting operations; and

d. Reduction in well yields due to grouting operations.

The subjects considered relating to post-construction operation of the completed facility are as follows:

a. Protection of the aquifers from contamination by sewage in tunnels;

b. Effect of completed facility on water table; and

c. Aquifer monitoring well operations, location, and standards.

Following is a discussion of the potential impacts listed above.

There are a number of private wells in the vicinity of the 20-foot and 16-foot rock tunnels (Contracts 73-317-2S and 73-320-2S) that may be influenced by the construction of these tunnels. The effect on these wells is anticipated to be limited to some clouding during grouting operations. These wells are thought to draw most of their water from the soil-rock contact area. A monitoring well has already been located in the vicinity of all known well areas, particularly near areas where complaints of cloudy water were received during the subsurface exploratory program. These wells will receive constant monitoring during construction. Should clouding caused by the grouting operations occur, pump-out of the monitoring wells may be sufficient to prevent the further migration of grout particles. If these procedures fail to prevent cloudiness, fresh potable water may be supplied for the short time required to complete the grouting and for the water to return to its original state. Since the grouting is intended to extend one tunnel diameter beyond the tunnel walls, no long-term effects on the wells are expected.

The potential groundwater drawdown within the glacial till-Silurian aquifer is extremely difficult to predict. The ability of water to migrate from the soil-rock into the tunnel is a function of the number of joints and bedding planes intersected by the tunnel. No data is available which will allow precise arithmethical determination of joints, bedding planes and opening sizes which may be encountered. Several factors are known however, if only in a general way, which enable a review of the existing soil-rock conditions and an evaluation of how these conditions may impact groundwater loss into the proposed conveyance system. The known factors are the results of soil and rock boring programs performed for the project, a review of conditions that prevailed on a similar project before and after grouting.

The glacial till throughout the project area is predominantly fine grained soils with an estimated coefficient of permeability of 10^{-6} to 10^{-8} cm/sec. These soils do not readily release water and consequently will not cause significant inflows into the tunnel which will be detrimental to construction or water levels in the soild. Isolated sand and gravel pockets exist within the glacial till which are discontinuous and not connected to the surface. These pockets will hold limited amounts of water which, if encountered by construction, will release their contained water. The quality of such water appears to be extremely limited and is not known to be used as a potable water supply. These waters will be replaced, in time, after completion of construction, by natural recharge.

The Silurian system is described in depth in Volume I, Bedrock Geologic investigation of the Geotechnical Report on Upper Des Plaines Tunnel and Reservoir Plan, Contracts 73-317-2S and 73-320-2S, dated July 1974. This report describes the rock systems to be encountered and postulates the conditions expected to be encountered during construction. Since the water bearing features within the rock consist of openings, primarily in form of cracks and joints, location of the inflows are easily located after excavation, particularly within machine bored sections. It is the intention of the Contract Specifications that grouting be performed shortly after excavation so that such inflows will be stopped before the pezometric level can be appreciably affected over any sizeable areal area.

Two pump-out tests performed in the course of the subsurface investigations failed to reflect any affect on observation wells as close as 75 feet away.

This supports the contention of Foundation Sciences, Incorporated, Geologic Consultant for the project, that intersection of joints will have an extremely local effect on existing peizometric levels.

During construction of a similar project, groundwater actually rose in nearby observation wells, as a result of seasonal fluctuations. Also, on that earlier project, one of the highest inflow areas was adjacent to a quarry which was open and dry, further indicating the difficulty in predicting groundwater loss or behavior in the localized area of the proposed tunnels.

It is anticipated that drawdown of the aquifer during operation of the facility will be virtually zero. It is expected that grouting will reduce groundwater flow into the tunnel to less than 300 gpm over the total length of tunnels, based on results obtained with previous projects. The tunnel lining will further reduce the inflows. Any openings in the lining which permit significant groundwater inflows must be repaired. Any local drawdown which may occur will be short-term, due to the tunnel tightness, and is expected to return to the original piezometric level.

The design storm of July 1957 represents the most severe storm of the 21-year study period having a postulated frequency of occurrence of greater than once in a 100 years. During this storm event, the predicted maximum hydraulic gradient during peak runoff conditions for the ultimate system, with reservoirs, will vary between evelation +23 (City of Chicago Datum) at the downstream end to elevation +52 at the 16-foot tunnel upstream end. This dynamic condition would last for less than one hour.

Immediately following the storm, the system will fill to static evelation +52, which represents the maximum design surcharged conditions. The length of time the water level within the tunnels will be at elevation

+52 will be approximately 18 hours. Consequently, there may be a period of approximately 18 hours that would occur with a return frequency of greater than once in a 100 years, when the level in the tunnels would be slightly higher than that of the surrounding groundwater.

Accurate data is not available on the groundwater levels, or its seasonal fluctuations. The groundwater readings taken during the boring program range too widely to be used with any arithmetical certainty. Since the tunnels are being lined due to geological reasons, and since it is believed that the tunnels can be maintained in a water tight condition, computations have not been made which would quantify transmissibility between the tunnel system and the aquifer.

The following criteria set up by the MSDGC with respect to the aquifer monitoring wells will apply to the proposed project.

> "To demonstrate that the project is not causing contamination of the groundwater, it will be necessary to set up monitoring programs, consisting of sampling wells with instrumentation to provide continuous recordings of water level and the necessary equipment to extract water samples for laboratory analysis. Instrumentation for recording water level within the tunnel will also be required to obtain the interrelationship between the groundwater levels and the tunnel pressures.
>
> "Samples will be tested for the following parameters:

pH	NH_3-N	Total Bacteria Plate Count
BOD	Total Phosphorus	Coliform (M.F.)
Chlorides	Phenol	Fecal Coliform (M.F.)
Hardness	COD	Fecal Strep. (M.F.)
Alkalinity	Cyanide	Conductivity
	Mercury	T.S.S.

Sampling will be performed at each of the wells at two-week intervals and after each major storm event by the Research and Development Department of the MSDGC. Monitoring wells will be installed at approximately one-half to three-quarter miles along the line of tunnel at a minimum offset distance of 30 feet from the edge of the tunnel so as to be outside the grouted area. Figure 5-5 shows approximate location of monitoring wells considered for this project. In order to have continuous information of changes of groundwater conditions and characteristics due to implementation of this project, all monitoring wells should be in operation prior to tunnel excavation.

In response to the concern of certain citizens who experienced well clouding during the soil and rock exploration program, it is noted that the nature of the work performed during this program was quite different than that to be done during construction of the tunnels and shafts. In the drilling of the rock and soil borings, water was used, under pressure, to remove soil and rock particles. This water pressure apparently caused a migration of soil and rock strata which serves as a water source for private wells. The nature of the construction operations will be such that, during excavation of the tunnels and shafts, no net flow of water will flow from the construction areas, but in fact there will be a movement of water, or at least a tendency of movement, into the excavation areas, thereby precluding the possibility of groundwater contamination.

In areas of the tunnel where infiltration of groundwater into the rock tunnels through bedding planes, faults or fractures is encountered, it will be necessary to grout at these points. These openings will be sealed and groundwater infiltration will be reduced, thereby insuring the maintenance of the groundwater level above the tunnel and thus eliminating the possibility of sewage from escaping into the surrounding rock. The grouting should have no effect on nearby wells as it is performed with quick setting mixtures and is thus kept in close proximity of the periphery of the tunnel.

In the event of any effect on groundwater quality, or potential clouding

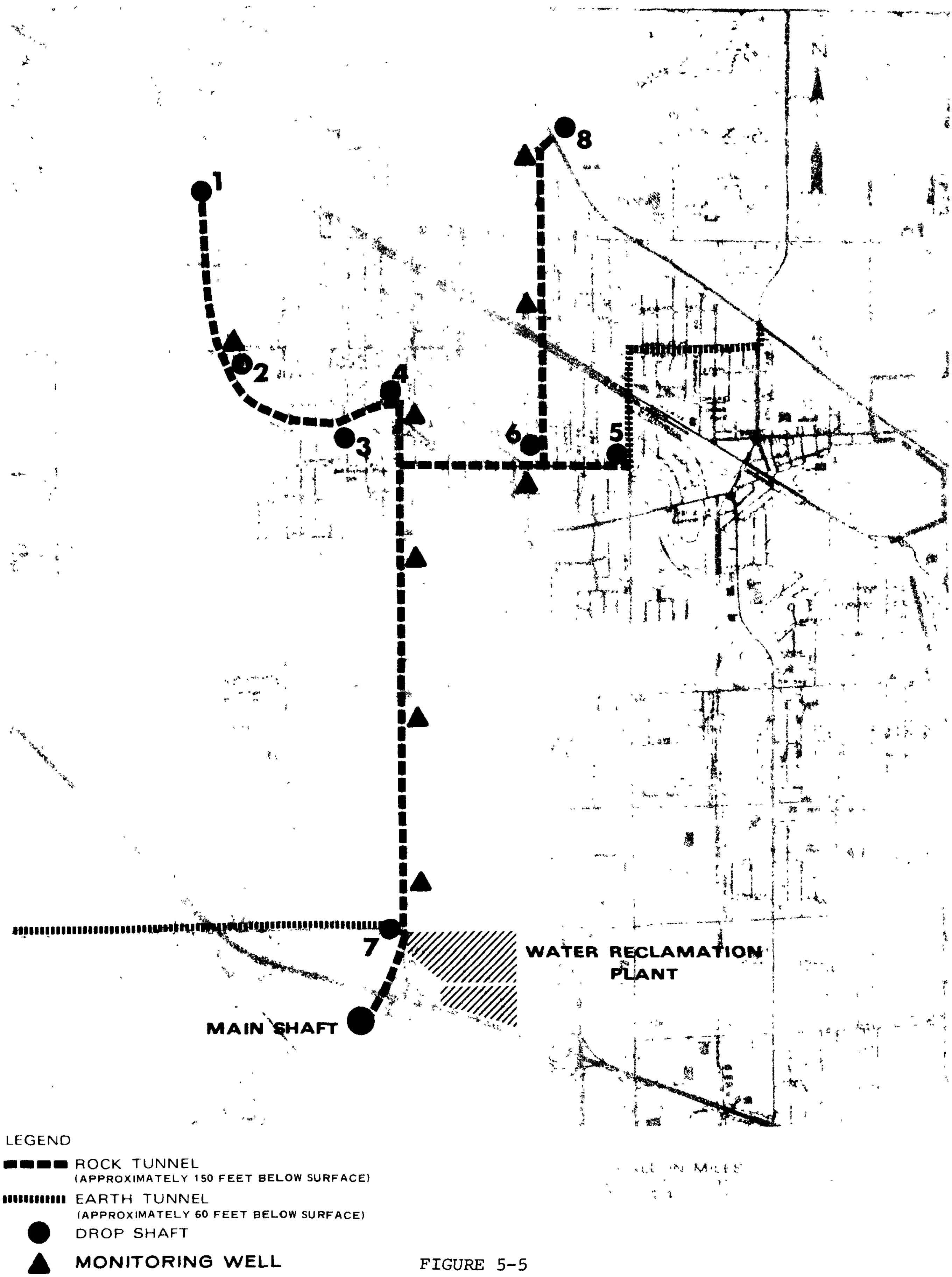


FIGURE 5-5

LOCATION OF MONITORING WELLS

of wells, grouting operations will be stopped until remedial measures have been taken to insure the necessary protection of the public.

D. Land

A map of existing land uses around the conveyance system has been included in Chapter 2.

Without the conveyance system, existing conditions of sewer backups and stream pollution from combined sewer overflows would continue. The entire conveyance system is located underground with drop shafts and access manholes at ground level; tunnelling will require the acquisition of easement rights.

Some surface features of the tunnel project will be located in open space areas. Further information needs to be acquired regarding disturbances during and after construction in park and school properties.

Land use impacts as prepared by MSDGC, are included in Table 5-1. Existing zoning and land use controls in the service area will impact future growth. The anticipated growth of the area has been evaluated by MSDGC and NIPC staff with varying findings. The capacity of the conveyance system to accomodate future proposed growth is sufficient. Industrial development has been forecast by MSDGC from 2000 acres in 1970 to 7300 acres in 2000. At a meeting of the Planning Committee of the Regional Planning Commission (NIPC), the NIPC staff indicated that industrial growth patterns would be half of MSDGC's estimates. However, MSDGC contends that their evaluation of NIPC documents, rate of industrial growth and expected growth would support their projected industrial acreage figure.

NIPC planning papers do not include industrial acreage forecasts. After a presentation by MSDGC to NIPC regarding this issue, the Regional Planning Commission approved the MSDGC design capacity of the conveyance system and WRP.

Factors in addition to conveyance system capacity will result in ultimate industrial development of existing vacant acres.

E. Air Quality

1. Construction Impacts

Construction would require the use of large vehicles and trucks, generally driven with internal combustion engines with the resultant addition of these air pollutants to the atmosphere. Under adverse weather conditions, residents and animal populations in the immediate vicinity of the surface construction activity could notice an air quality change. This would be a temporary condition during construction, lasting no longer than three months for access manholes, and 32 weeks for the drop shafts. Total project construction activity above and below surface and at the main shaft work area would extend over a 42-month period. All of these effects, however, are anticipated to be minor with the continued improvement in vehicle emission control devices. Pollutant levels affecting construction workers, especially those employed in the underground tunneling operations, will be within the limits of the Federal Occupation Safety and Health Act (OSHA) standards.

Dust from construction activities at the surface sites could be significant without proper controls. The mechanical ventilation systems used in underground construction control the quality of exhaust discharged to the atmosphere.

Dust raised by truck traffic will be minimized by using hard paved surfaces and dust control measures.

The amount of blasting and thus the volume of particulate matter that could be emitted from the drop shafts due to blasting will be small and in low concentrations. The rock blasted in the shafts will be wet by groundwater. In addition, it will, by necessity, need to be matted by rubber tire or steel mesh mats prior to detonation of explosives, to prevent release of high velocity particles.

2. Operational Impacts

Operation of the system will not produce particulate matter. No adverse aerosol or water vapor effects are anticipated to be present in the construction phase of any alternatives. During operation, the possibility of aerosol generation and release from the operation of the drop shafts does not appear to be a problem. As presently designed there should be no net movement of air out of the drop shafts. During model studies there were no observations of aerosolization. Should it be found that the actual drop shaft operate differently, the openings to the surface could be easily sealed without hydraulically affecting the structures.

A marked improvement in the residual odors reported in the Weller's Creek area should result from the elimination of the combined wastewater overflows. Construction activities should not create odor problems. No other odor problems are anticipated. The conveyance facilities are designed to maintain self-cleansing velocities to prevent deposition of solids and consequent creation of odors so that there would be no detrimental effects on the population in the vicinity of the drop shafts and manholes.

3. Mitigating Measures

Remedial actions to minimize air pollutants would include effective enforcement of regulations regarding the operation and maintenance of construction vehicles. A continued reduction of emissions from these vehicles would result as provisions of the Clean Air Act are implemented.

The levels of particulate matter resulting from blasting and excavation could, if necessary, be reduced by scrubbing controlled exhausts of ventilating systems, effective use of dust control water and chemicals, and cessation of surface activities during adverse weather conditions.

Reduction of pollutants resulting form the operation of construction equipment will require the constant surveillance to assure all operations are consistent with MSDGC requirements.

F. Biology

The natural ecosystems along the tunnel route have already been extensively altered or eliminated by human activities. The proposed project will have short term adverse impact on terrestrial plants and animals from the construction of dropshafts and manholes. Vegetation removed during the project may be replaced to reverse this impact and restore the habitat for animal life.

The aquatic ecosystems of Higgins Creek may be adversely affected by siltation from construction erosion and by fluctuation of water temperatures produced by the dewatering of tunnels. Detention ponds should greatly reduce these problems. There should be no long term effect on the stream if species are able to migrate into the affected area from upstream areas. The long term effect of the project will benefit water quality and the stream biota by greatly reducing combined sewer overflows.

G. Environmentally Sensitive Areas

Parklands will be affected by the manhole construction of this project. This has been discussed with other impacts to area land use in Section D.

H. Aesthetics

Most of the tunnel construction will occur underground, reducing its visual impact. Surface connections at dropshafts and manholes will cause a temporary adverse visual effect, which will be largely corrected when the construction site is restored. Manhole covers, 26 inches in diameter, will be visable at the ground surface upon completion of the project and replanting of the sites. Dropshafts will each have two 47 inch outside diameter manhole frame and cover castings, and one 3 foot 6 inch by 10 foot open grate visable from the ground surface.

I. Noise and Vibration

Noise levels may reach 120 decibels during construction; however, most noise will be intermittent and generally below 100 decibels. Noise will be an adverse, short term impact. Mitigative measures will be taken to reduce noise from construction equipment and trucks.

The operation of exhaust systems during construction might be considered noisy and objectionable by nearby residents. Both working shafts are not in residential areas, but are adjacent to heavily travelled roads such as the Northwest Tollway and Rand Road; therefore, its impact should be minimal.

The tunnel depth should reduce any blasting vibration during shaft and possible tunnel blasting. Blasting operations will be planned to maintain particle velocities at less than one inch per second and vibration potentials will be within permissible limits depending on the specific sites where construction operations are planned. Information programs will prepare the public for the unavoidable temporary vibrations and noise. Construction schedules would take into account those hours of operation where noise would cause the least disturbance. Use of moles will minimize these problems in the construction of the tunnels themselves.

J. No Action Alternative

A no action decision would result in a continuation of the combined wastewater overflow during peak storms from the 29 outfall points within Weller's Creek and Feehanville Ditch Drainage Basins. See Figure 5-4. This condition, which presently occurs approximately 80 times annually, would eventually be in violation of IEPA Regulations requiring this wastewater to be directed to a treatment plant by December 31, 1977. The flood hazard would continue to increase. The sewage generated within basin would continue to flow to North Side Sewage Treatment Works through overloaded interceptors.

K. Summary

The proposed projects will result in a significant improvement in environmental quality, with respect to the water quality of the local streams. Construction of the projects will, however, result in the irretrievable commitment of certain resources, such as concrete and energy. Once completed however, the operational impacts will be practically non-existent.

The only long-term negative impact resulting from construction will be the very localized soil compaction in the area of the construction shafts. This impact cannot be considered significant when viewed in light of the extensive land development and other construction taking place in the area. Other negative impacts will be of a short-term nature. The turbidity of Higgins Creek will be temporarily and periodically increased due to the water pumped out of the tunnels during construction. In order to mitigate this adverse effect, a detention pond (1/2 hour detention time) will be provided to allow some settling of the materials. Construction vehicle exhaust and dust resulting from vehicle operation will also temporarily affect localized air quality. Given the magnitude of other air polluting sources in the area, the incremental effect of this project will be insignificant. There will be some adverse impacts from the explosives used in the construction of the tunnels. The noise vibrations at the surface will be minimized by careful design and construction controls.

The most significant impact of the projects will be the improvement in the water quality of Weller's Creek and Feehanville Ditch, which now receive tremendous BOD and suspended solids loadings from the 80 or more combined sewer overflow events each year. The proposed projects will reduce the number of overflows to 6 or less each year. While this is a significant reduction, the ultimate objective is to eliminate all combined sewer overflows. The MSDGC is presently examining various alternatives for dealing with the remaining overflows. (See Figure 5-6.)

L. Findings

As a result of this EIS, we believe the following actions would serve to increase the environmental compatibility of the proposed projects:

1. The Upper Des Plaines - O'Hare tunnel conveyance system can be constructed as proposed, provided the necessary environmental safeguards discussed in this EIS are implemented.
2. The MSDGC should take whatever steps are necessary, within their control, to insure that the rock extracted during the construction of the conveyance system is utilized in the most environmentally compatible manner.
3. The MSDGC should take additional reasonable measures necessary to decrease the amount of siltation in Higgins Creek due to the water pumped from the tunnels during construction. The possibility of increasing detention time in the pond should be seriously investigated.
4. Once the conveyance system is in operation the drop shaft openings to the surface should be monitored by the MSDGC for any significant

FIGURE 5-6

MATRIX SUMMARY OF IMPACTS

AREAS IMPACTED	ALTERNATIVES No Action	ALTERNATIVES Rock Tunnel Alternative
BEDROCK GEOLOGY	•	•
SOILS AND SURFICIAL GEOLOGY		
Weathered Soils	•	•
Soil Compaction	•	●
HYDROLOGY		
Surface Water Quality	●	○
Ground Water Quality	•	•
Ground Water Quantity	•	•
Water Quality-Higgens Creek	•	[●]
AIR QUALITY		
Vehicle Exhaust	•	[●]
Odors	●	○
Dust	•	[●]
Aerosols	•	•
ECOSYSTEMS		
Wildlife Habitat	●	○
Rare and Endangered Species	•	•
TOPOGRAPHY	•	•
CLIMATE	•	•
LAND USE		
Landscaping	•	[●]
Traffic Flow	•	[●]
Esthetic Appearance (Surface Structures)	•	•
Permanent Easements	•	•
Combined Overflow Hazard	●	○
NOISE AND VIBRATION		
Noise	•	[●]
Vibration	•	[●]

KEY:

- ● Negative Impact
- • No Impact
- ○ Positive Impact
- [ ] Duration of Impact is Temporary (Construction Period)

odor and/or aerosol releases. If these are found to occur on a regular basis, consideration should be given to the provision of covers.

5. The MSDGC should fully evaluate alternatives to the combined sewer overflow reservoir which may be built on site 2. The cost-effective analysis of alternatives should specifically include the possible interconnection to the main TARP system presently proposed.

CHAPTER 6

FEDERAL/STATE AGENCY COMMENTS

AND PUBLIC PARTICIPATION

(This chapter will be completed after circulation of this Draft EIS and the public hearing. It will be included in the final EIS.)

CHAPTER 7

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APPENDIX A

O'Hare Area Flood Control Activities

Design Criteria

Wilke-Kirchoff Reservoir, Project 70-407-2F
Heritage Park Reservoir, Project 68-815-2F
White Pine Ditch Retention Reservoir, Project 72-313-2F
Buffalo Creek Retention Reservoir, Project 67-803-2F
Willow Higgins Retention Reservoir, Project 68-836-2F
Mount Prospect Retention Reservoir, Project 69-308-2F

WILKE-KIRCHOFF RESERVOIR, PROJECT 70-407-2F

The Wilke-Kirchoff Reservoir is a multi-purpose excavated flood-water retarding, pump evacuated reservoir constructed by the Metropolitan Sanitary District of Greater Chicago, in cooperation with the Village of Arlington Heights, at a cost of $871,000. The reservoir occupies a 16 acre site, is 12 feet deep, and has a storage capacity of 100 acre-feet. It serves a 717 acre tributary area and is designed to accommodate a 100 year storm.

The Wilke-Kirchoff Reservoir is located south of Kirchoff Road, and east of Wilke Road in the Village of Arlington Heights.

The reservoir was designed to serve as a recreational facility, in addition to its primary function of reducing local flooding. Possible winter activities include tobogganing and skiing on a large earth mound in one corner of the reservoir formed with excavated material. Summer activities can include such things as volleyball, basketball, baseball, soccer, football, and a general play area. All recreational activities are supervised by the Arlington Heights Park District.

The reservoir is excavated in a clay soil. The side slopes are 7:1, providing easy access to the bottom of the reservoir for recreational usage. The bottom and side slopes are sodded to prevent erosion and to present an esthetically attractive appearance.

A pumping station, located at the northwest corner of the site contains three variable speed pumps with a capacity of 6.67 cfs to 12 cfs and two low flow pumps with a capacity of 0.33 cfs. These pumps can empty a full reservoir in 6 days. Most storms, however, will not fill the reservoir completely, and the dewatering time will be less than 6 days. An underdrain system is provided beneath the reservoir floor to remove the excess ground and storm water and thereby provide maximum recreational usage of the reservoir bottom.

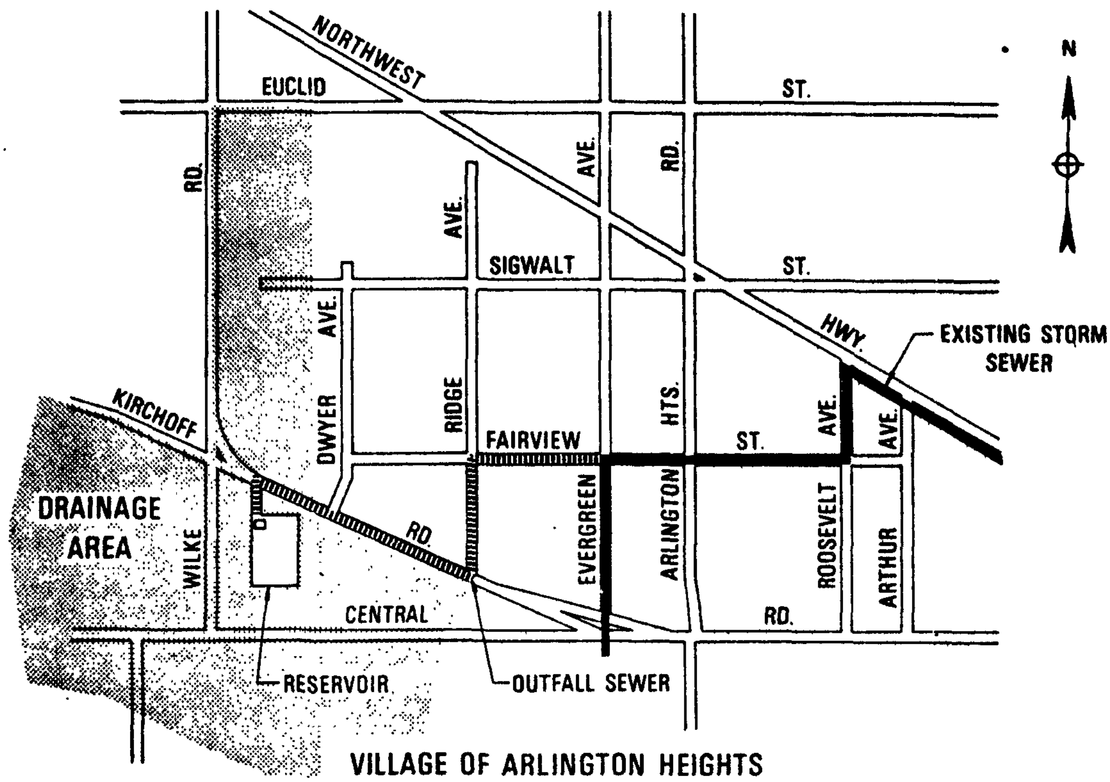
Storm sewers draining the tributary area carry the runoff into the reservoir through two inlet structures. At low flows, the runoff drops through a grate in the inlet structures and is conveyed to the pumping station through the reservoir dewatering system. The multi-purpose use of the reservoir is enhanced by use of the low flow bypass system. The water from the reservoir is pumped through a 30 inch force main to a storm sewer that discharges into Weller Creek, the natural drainage outlet for the reservoir tributary area.

Construction of the reservoir began in August 1972, and was completed in the fall of 1973. The Metropolitan Sanitary District contributed $736,000 of the construction cost and the Village of Arlington Heights contributed $135,000. In addition, the Village of Arlington

Heights paid $195,000 to acquire the reservoir site and also assumed the engineering design costs. The Village will be responsible for the operation and maintenance of the facilities.

WILKE - KIRCHOFF RETENTION RESERVOIR
RESERVOIR - PROJECT NO. 70-407-2F
OUTFALL SEWER - PROJECT NO. 71-310-2F

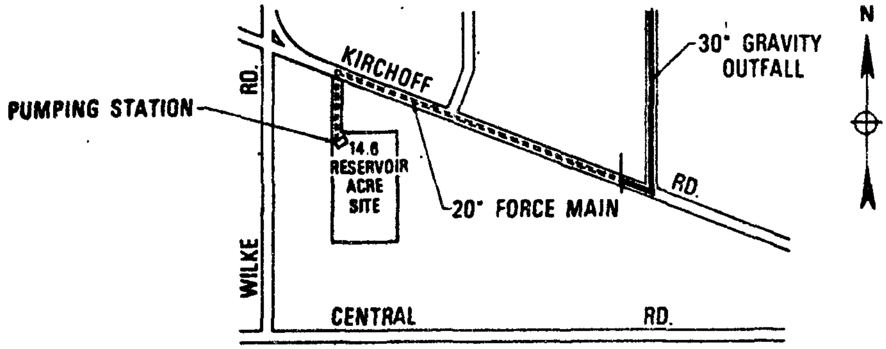
DRAINAGE AREA	717 ACRES
DESIGN STORM	100 YR.
PUMPING STA. CAPACITY	36.7 c.f.s.
CONSTRUCTION COMPLETED	
CONSTRUCTION COSTS	$ 871,000
LAND AREA	14.6 ACRES
LAND COST	$ 232,000



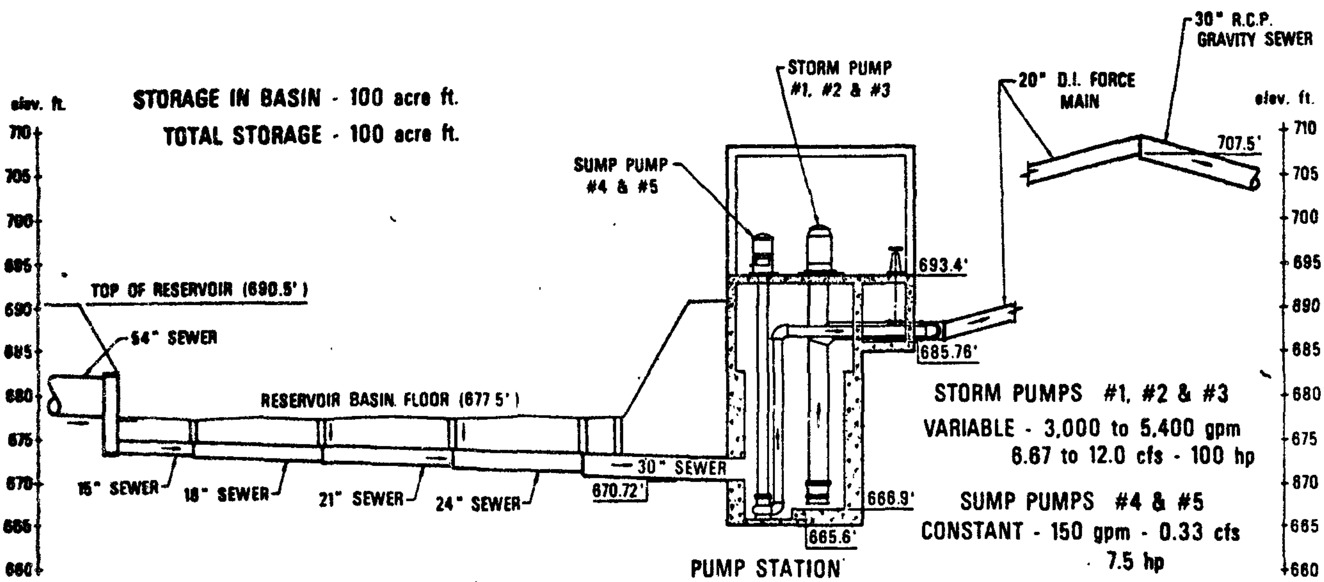
LOCATION MAP

METROPOLITAN SANITARY DISTRICT
OF GREATER CHICAGO
FLOOD CONTROL SECTION
JAN. 1973

WILKE - KIRCHOFF RETENTION RESERVOIR
RESERVOIR - PROJECT NO. 70-407-2F
OUTFALL SEWER - PROJECT NO. 71-310-2F



RESERVOIR LAYOUT



STORM PUMPS #1, #2 & #3
VARIABLE - 3,000 to 5,400 gpm
6.67 to 12.0 cfs - 100 hp

SUMP PUMPS #4 & #5
CONSTANT - 150 gpm - 0.33 cfs
7.5 hp

TOTAL PUMP CAPACITY - 16,500 gpm
36.7 cfs

OVERFLOW @ 689.5' elev.

PUMP CONTROLS - SPARLING - FLOAT

PROFILE

METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO
FLOOD CONTROL SECTION

73 F 539 R2 E.E.W. JAN. 1973

HERITAGE PARK RESERVOIR, PROJECT 68-815-2F

The Heritage Park Reservoir is a multi-purpose flood water reservoir constructed at a cost of $270,000 by the Metropolitan Sanitary District in cooperation with the Village of Wheeling and the Wheeling Park District. The reservoir is on a 25 acre site and has an average depth of 5 feet. It has a usable storage volume of 112 acre-feet which serves a tributary area of 447 acres and is designed to accommodate a 100 year storm.

The reservoir is an excavated storage gravity discharge structure. The reservoir was designed as a multi-purpose facility for recreation, in addition to its primary function of reducing local flooding. A permanent lake, about 8 acres in area and 5 feet deep, is provided to enhance the recreational features of the reservoir and usage for winter activities. Tobogganing and skiing utilize a large earthen hill constructed east of the reservoir with material excavated from the reservoir. The adjacent park areas are utilized for all other seasonal activities. Recreational activities are supervised by the Wheeling Park District.

The reservoir side slopes are 4:1 or less, permitting easy access to the reservoir bottom except for the permanent lake area. The reservoir area is grassed to prevent erosion and enhance the recreational use.

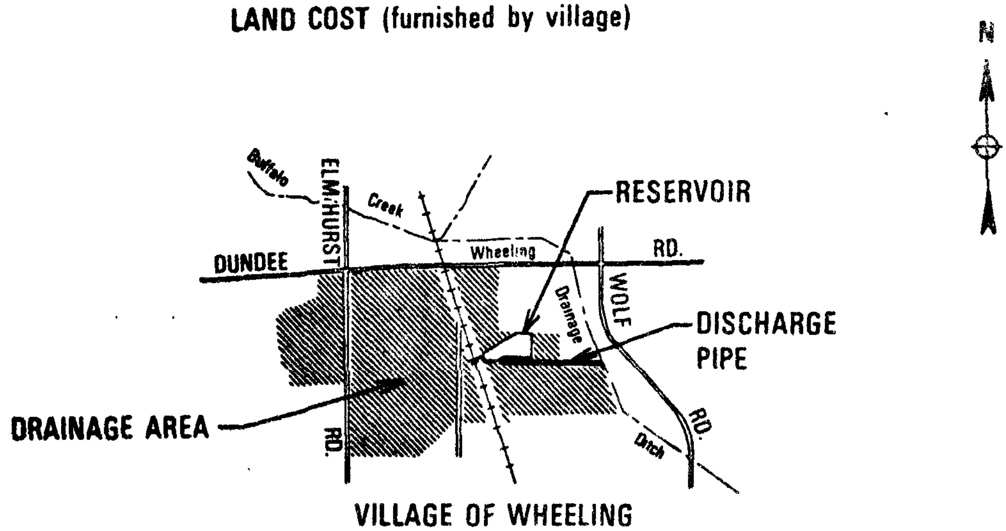
The reservoir is emptied through a 60 inch diameter pipe into the Wheeling Drainage Ditch. A flap gate on the 60 inch discharge pipe prevents water from the drainage ditch entering the reservoir during periods of high water in the Wheeling Drainage Ditch. The flap gate also restricts the flow of storm water out of the reservoir until flow capacity is available in the Wheeling Drainage Ditch.

The reservoir construction was completed in 1970. The Metropolitan Sanitary District contributed $180,000 of the facility's construction cost and the Village of Wheeling contributed $90,000 for the construction and paid the engineering design costs. The land for the reservoir was provided by the Wheeling Park District.

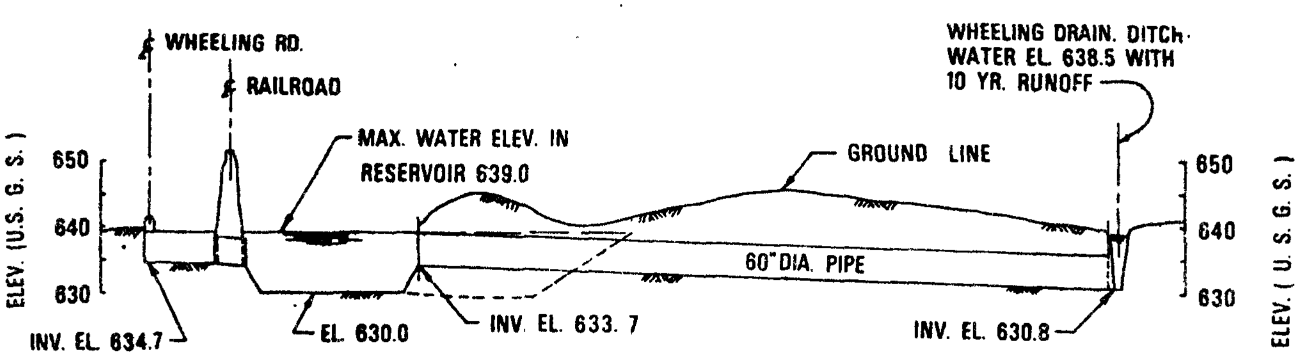
HERITAGE PARK WEST RETENTION RESERVOIR

PROJECT NO. 68-815-2F

DRAINAGE AREA	447 ACRES
DESIGN STORM	100 YEARS
PUMPING STA. CAPACITY	NONE
CONSTRUCTION COMPLETED	2-16-70
CONSTRUCTION COSTS	$270,000 TOTAL (M.S.D. PAID 67%)
LAND AREA	25 ACRES
LAND COST (furnished by village)	



LOCATION MAP



PROFILE

METROPOLITAN SANITARY DISTRICT
OF GREATER CHICAGO

FLOOD CONTROL SECTION

WHITE PINE DITCH RETENTION RESERVOIR, CONTRACT NO. 72-313-2F

The White Pine Ditch Retention Reservoir is a project of inter-agency cooperation that will divert flow from the White Pine Ditch Watershed to control chronic overbank flooding and sanitary sewer back-up caused by storm flows entering and overloading the sanitary sewer system. The need for additional public services to assist people in flooded areas, and the loss of direct access to or around flooded areas with emergency equipment, is costly to the habitants in both life and assets. Diversion of the existing and increased flows from the road improvement and urbanization will convey the flows to a reservoir site with the capacity to store the excess storm runoffs. Along the water-course no site could adequately provide the protection from the 100-year storm event.

The Dundee Road improvement project, developed and under construction by the Department of Transportation, State of Illinois, includes the larger sized storm sewer to divert flows from the White Pine Ditch to the east. The discharge of this sewer and the naturally contributing areas are directed into the retention reservoir of 50 acre-feet storage capacity. The reservoir and White Pine both discharge into Buffalo Creek.

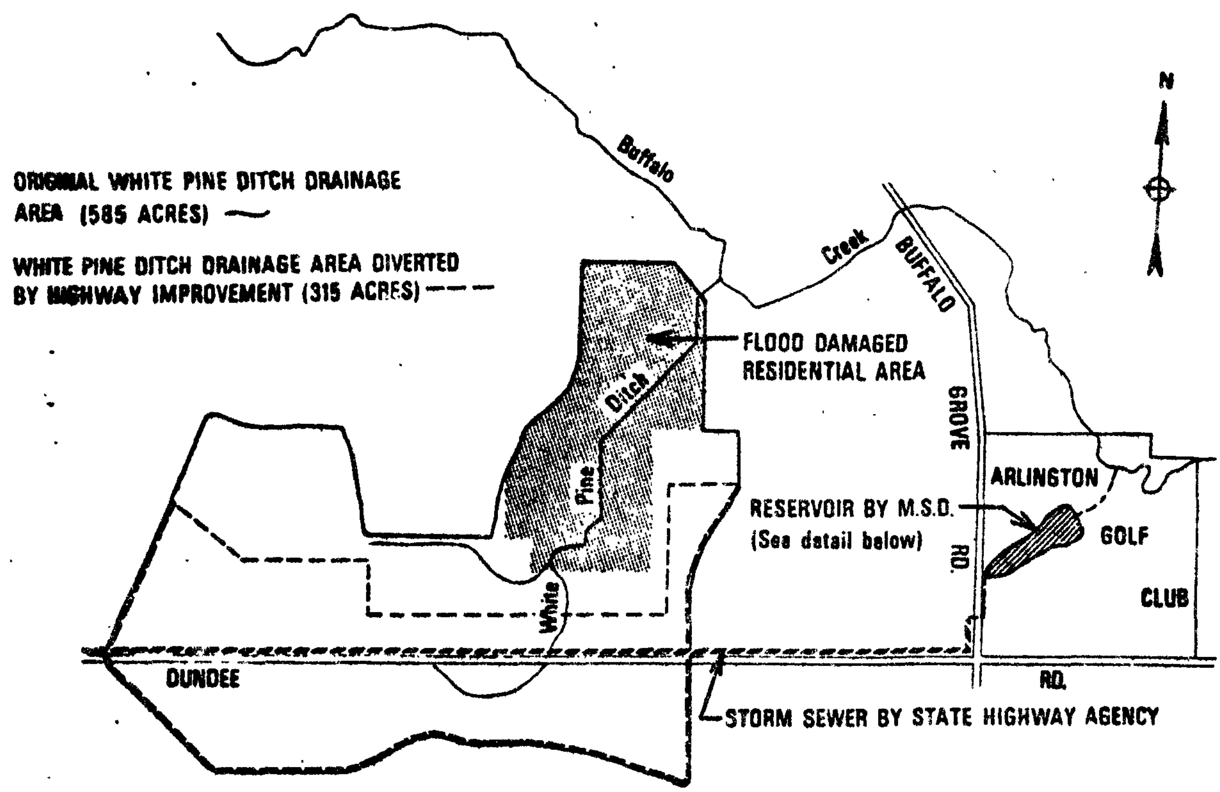
The Village of Buffalo Grove reported the monetary flood related losses for the year 1972 to be $50,700. These losses for the White Pine Ditch area only involved 119 homes.

Cost involvement for the reservoir project are as follows: $120,000 from the Metropolitan Sanitary District, $130,000 from the Department of Transportation of the State of Illinois, and any additional cost by the Village of Buffalo Grove.

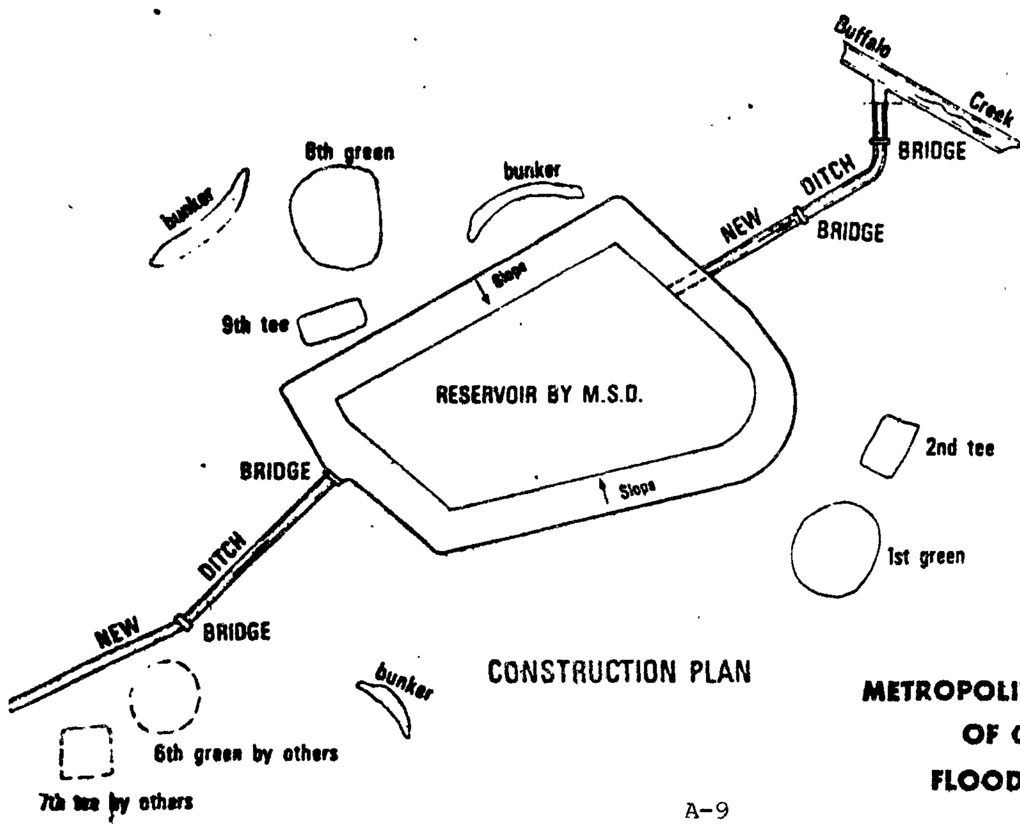
In addition to the construction cost for the reservoir, the Sanitary District will administer the construction contract and the Village of Buffalo Grove will secure the land rights upon which the reservoir is located.

The Sanitary District has authority to undertake this work and commit funds without a general election. Plans and specifications were awarded August 8, 1974. Work will be completed by May 1, 1975.

WHITE PINE DITCH RESERVOIR



LOCATION PLAN



CONSTRUCTION PLAN

EXHIBIT 3

**METROPOLITAN SANITARY DISTRICT
OF GREATER CHICAGO
FLOOD CONTROL SECTION**

BUFFALO CREEK RETENTION RESERVOIR, CONTRACT NO. 67-803-2F

Urbanization of the Buffalo Creek Watershed has increased storm runoff and flooding in areas adjacent to Buffalo Creek and the Wheeling Drainage Ditch in the Village of Buffalo Grove and Wheeling. To reduce this flooding, the proposed Buffalo Creek Retention Reservoir will impound approximately 700 acre-feet of storm water. This valley reservoir will be an earthfill dam located just west of Arlington Heights Road and south of Checker Road in Lake County. A culvert control structure will pass low flows and limit the maximum discharge from the reservoir to approximately 250 cfs. An emergency spillway will be provided to pass storm flows from storm events that exceed the 100-year storm event storage capacity and to protect the dam structure.

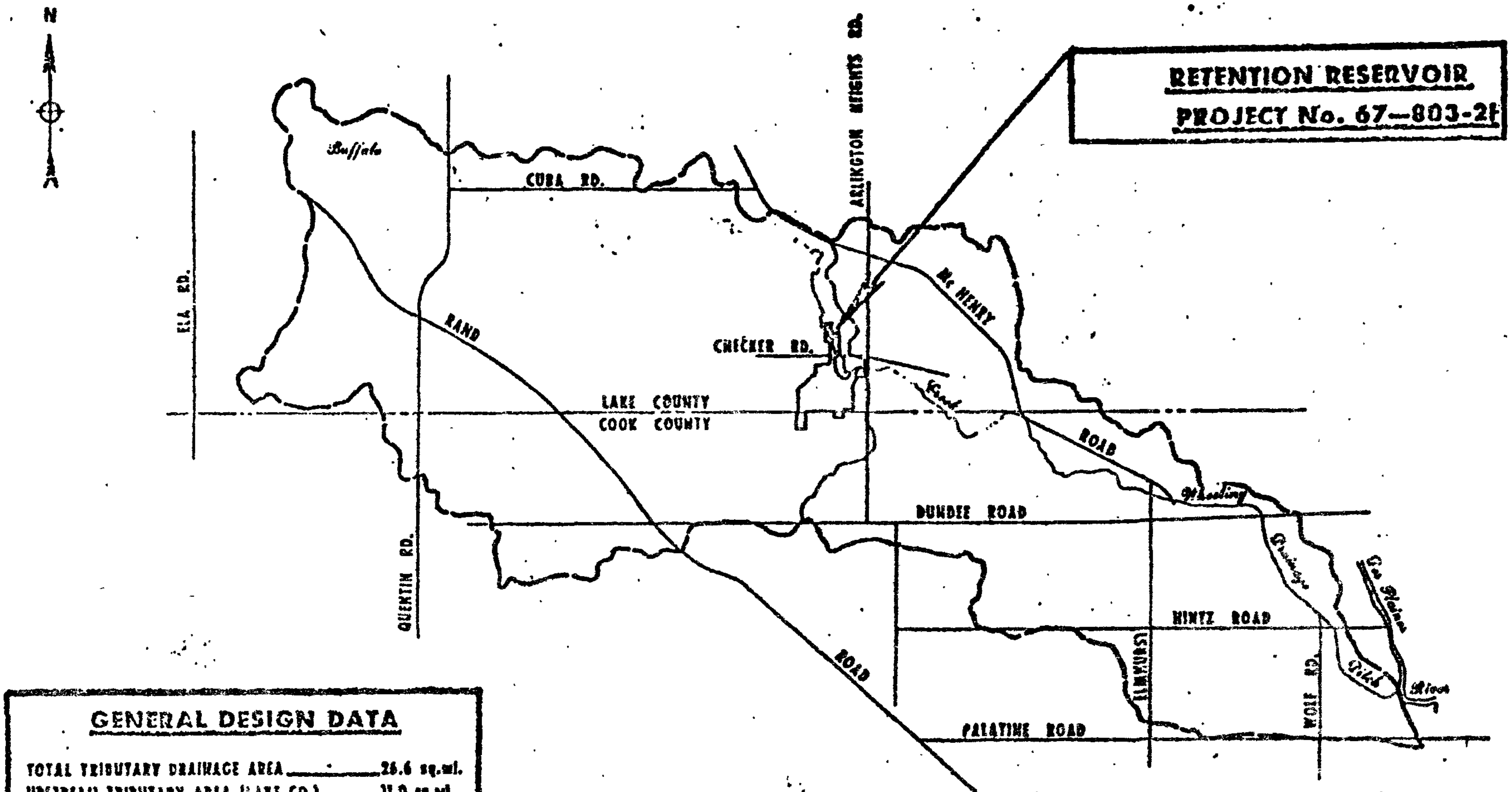
Additional construction work includes a levee or other flood protection method for the private buildings adjacent to the reservoir site north of Checker Road. Checker Road will be raised above the high water elevation as will the new bridge over Buffalo Creek. The reservoir site is approximately 160 acres located west of Arlington Heights Road in Section 31 of Vernon Township, Lake County. The site also included some area in Wheeling Township, Cook County. Total cost to the District is estimated at $2,100,000.

Project implementation will be guided by a Cooperative Agreement between the Lake County Forest Preserve District, Village of Buffalo Grove, and the Metropolitan Sanitary District.

The reservoir site will be a multiple-use facility for open space recreation uses, in addition to the primary function for flood control.

The Sanitary District has authority to undertake this work and commit funds without a general election. Plans and specifications will be available for bid advertisement in March, 1975. The construction contract will be let within 90 days after bid advertisement. Work will be completed by December, 1975.

PROPOSED RETENTION RESERVOIR SITE on BUFFALO CREEK



GENERAL DESIGN DATA

TOTAL TRIBUTARY DRAINAGE AREA	26.6 sq.mi.
UPSTREAM TRIBUTARY AREA (LAKE CO.)	11.0 sq.mi.
UPSTREAM TRIBUTARY AREA (COOK CO.)	4.4 sq.mi.
DOWNSTREAM TRIBUTARY AREA (LAKE CO.)	2.5 sq.mi.
DOWNSTREAM TRIBUTARY AREA (COOK CO.)	8.7 sq.mi.
DESIGN FREQUENCY FOR STORM RUNOFF	100 yr
DURATION	24 hrs
RAINFALL (TOTAL)	6.0 inches
STORM WATER RUNOFF (AVERAGE)	40%

WILLOW-HIGGINS RETENTION RESERVOIR, PROJECT 68-836-2F

The project includes the construction of two storm water retention reservoirs, Willow-Higgins Creek Channel Modifications and Willow Creek relocation to control the flooding in the Willow-Higgins Creek Watershed for the 100 year storm event as shown on Exhibit 1. This provides for storing flows from the O'Hare Water Reclamation Plant that exceeds channel capacity. The reservoirs will be located in the O'Hare Airport runway clear zone areas. Clear zones are provided at the ends of runways because of the high noise level in these areas, the need to control elevation of structures in runway approaches and to provide for aircraft over run conditions and thus are unavailable for individual public use.

The Ravenswood Reservoir site is located approximately 2750 feet from the end of the runway 32R-14L and the Lee Street Reservoir site is located approximately 900 feet from the end of runway 4L-22R. The Willow-Higgins Creek Channel Modifications will be a closed concrete section downstream of the Lee Street Site. The Willow Creek Relocation will consist of both grass lined earth channel and closed concrete sections as physical conditions permit.

Willow Creek will be relocated generally along the western limits of O'Hare Airport south of Old Higgins Road, and then northerly to the Ravenswood Reservoir.

The project will relieve the flooding problems in the Willow Higgins Watershed downstream of O'Hare Water Reclamation Plant for storm events up to the 100 year frequency. Relocation of Willow Creek would facilitate the future development of O'Hare Airport. Also, conveying the flow of Willow Creek drainage area to the Ravenswood Reservoir, will effectively utilize the greater storage capacity available at the Ravenswood site.

The flows added by O'Hare Water Reclamation Plant will be stored at Ravenswood Reservoir when the flow in the downstream channel exceeds the design capacity. These added flows will result from the treatment of flows from an ultimate population equivalent of 439,000 in the service area and also from the treatment of the storm flows from a combined sewer area of 8000 acres located in Weller Creek Watershed, part of Upper Des Plaines Tunnel and Reservoir Plan.

The proposed project would have following long term effects:

1. Eliminate flood damages for storm events up to the 100-year frequency and would provide peace of mind to the citizens in flood prone areas.

2. Provide storage for additional flows from the proposed O'Hare Water Reclamation Plant.

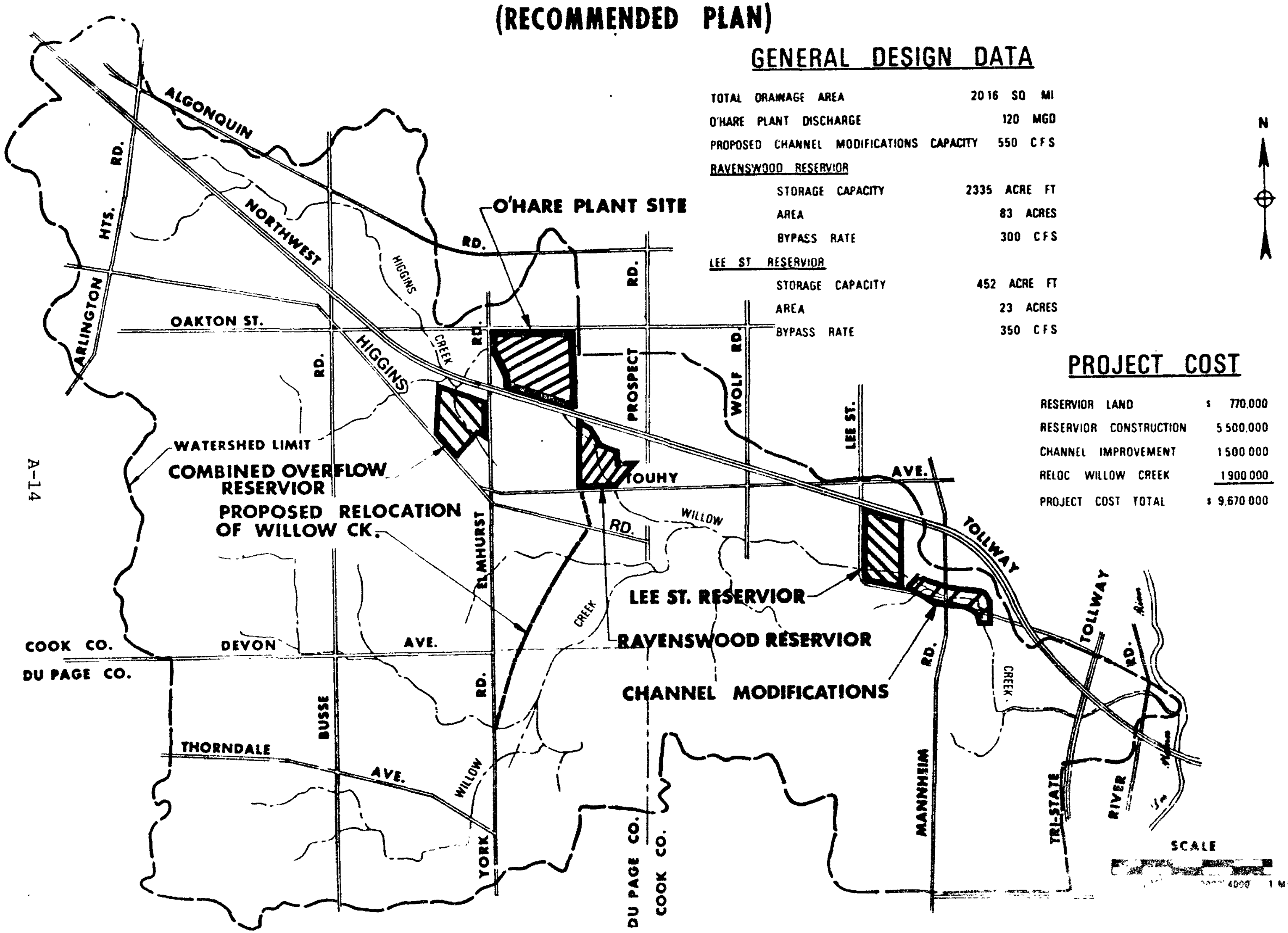
3. Facilitate the O'Hare Airport expansion program.

4. Use land located in clear zones for additional public benefit.

5. Increase property valuation by control of overbank flooding and thereby increase real estate tax revenues, even with the removal of some private land from the tax rolls for the project.

EXHIBIT

WILLOW - HIGGINS CREEK WATERSHED
(RECOMMENDED PLAN)



GENERAL DESIGN DATA

TOTAL DRAINAGE AREA	20.16 SQ MI
O'HARE PLANT DISCHARGE	120 MGD
PROPOSED CHANNEL MODIFICATIONS CAPACITY	550 CFS
RAVENSWOOD RESERVIOR	
STORAGE CAPACITY	2335 ACRE FT
AREA	83 ACRES
BYPASS RATE	300 CFS
LEE ST RESERVIOR	
STORAGE CAPACITY	452 ACRE FT
AREA	23 ACRES
BYPASS RATE	350 CFS

PROJECT COST

RESERVIOR LAND	$ 770,000
RESERVIOR CONSTRUCTION	5,500,000
CHANNEL IMPROVEMENT	1,500,000
RELOC WILLOW CREEK	1,900,000
PROJECT COST TOTAL	$ 9,670,000

F 513 R(P) 1

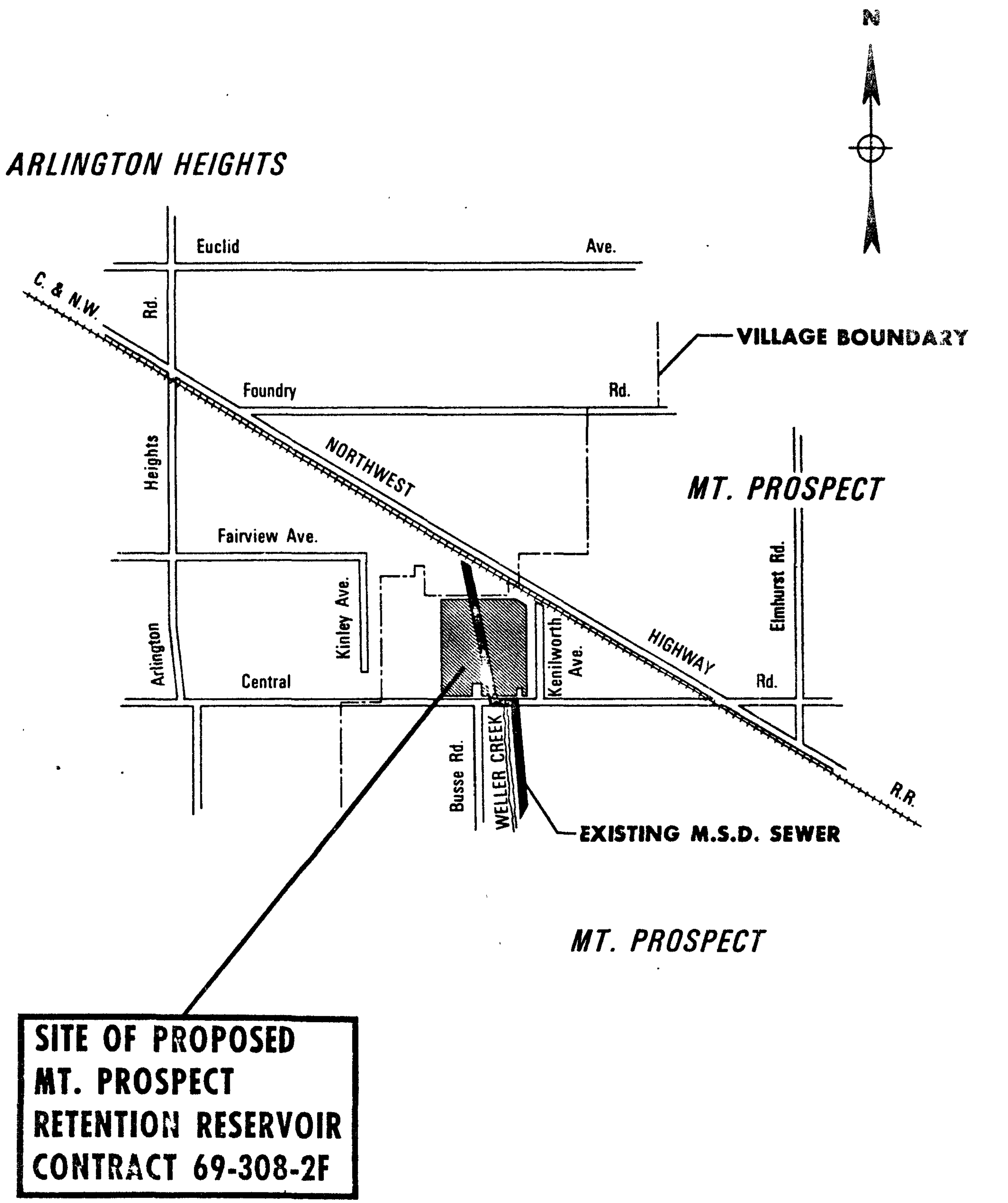
MOUNT PROSPECT RETENTION RESERVOIR, PROJECT NO. 69-308-2F

The reservoir will be an interim facility designed to provide a certain level of protection to the area until such time as the Tunnel and Reservoir Plan is implemented. At that time, the reservoir will be enlarged from 130 acre-feet (interim) to 850 acre-feet (ultimate). The basin will function by gravity. No pumping will be required.

The interim plan involved 130 acre-feet of storage providing relief to the upstream storm sewer system. The Village will be responsible for the measures necessary to convey separate storm flows into the reservoir. Conversion to the ultimate facility will involve enlargement of the reservoir and conveyance facilities to bring combined overflows into the reservoir.

The interim facility will store storm flows only. The ultimate facility will include measures to handle combined flows. The DeLeuw Cather report, "Preliminary Plans for O'Hare Collection Facility", concerns the O'Hare Tunnel and Reservoir system of which the ultimate facility will be a part. The interim facility is not covered in this report. Detailed design and analysis of the interim proposal will commence subsequent to the completion of negotiations with the Village and the purchase of the site.

Drop Shaft No. 1 under the Tunnel and Reservoir Plan for the (O'Hare) Upper Des Plaines Basin will be situated at Central Road and Weller Creek. The 850 acre-foot Mount Prospect combined waste water detention basin will function to limit the flow to Shaft No. 1 to 800 cfs. Based on a fully developed upstream drainage area, and an unrestricted upstream local sewer system (exceeding 100-year design storm frequency), this flow was exceeded 24 times in the 21 year record period, there were 21 times the maximum detention volume did not exceed 100 acre-feet. Twelve times the volume detained did not exceed fifty acre-feet. The maximum time of detention in the study period was 20 hours. This was for a recurrence of the July 1957 storm. In general, the detention period would be a small fraction of the 20 hour maximum even under full development conditions.



THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

ENGINEERING DEPARTMENT

AUG., 1973

APPENDIX B

MSDGC TARP PROGRAM

Combined Sewer Overflow Elimination

The selected plan for eliminating untreated combined sewer overflows or plant bypasses was chosen from various alternatives and was described in the August 1972 "Summary of Technical Reports," presented by the Flood Control Coordinating Committee. Since 1972 refinements to sub-systems of the plan have been made as additional studies and sub-surface exploration work have been performed.

This chapter first describes the August 1972 Recommended Plan and then describes the five revisions that have been made. These revisions do not change the concept of the plan but only present additional development of the project to reflect sub-system optimization.

August 1972 FCC Recommended Plan

Description and Maps

After extensive review of the alternatives, the Flood Control Coordinating Committee unanimously agreed that the Alternatives "G", "H", "J" and "S" Mod 3, are less costly and would be more environmentally acceptable to the community than any of the other plans presented. Detailed studies and layouts along the lines of these plans were then continued to develop the recommended plan.

The system recommended herein, a composite of several of the above Alternatives, is outstanding in its relative storage economy and simplicity. It will capture the total runoff from all of the record meteorological sequences of history, if they were to recur on future ultimate developed drainage basins, except for the peak few hours of three of the most severe storm events. The system will convey these captured combined sewer flows through high velocity, out-of-sight underflow tunnels below the routes of the existing surface watercourses to large pit-type storage reservoirs. Figure M-IX-1 shows the general location of the conveyance tunnel system and storage reservoirs.

The primary storage reservoir is shown located in the area now occupied by the sludge lagoons of the Metropolitan Sanitary District in the McCook-Summit area. This reservoir will be in the form of a 300 to 330 feet deep rock quarry, with a maximum water depth of approximately 200 feet, in the heaviest storm event, and water surface dimensions averaging about 1,000 feet wide by 2 1/2 miles long. Total storage capacity of the reservoir with the water surface as its maximum level of -100 CCD, will be 57,000 acre-feet.

Figure M-X-2 shows the general layout of the reservoir, conduits and pumping facilities. The lower 100 feet of depth of the reservoir will be divided into three basins by transverse dikes, providing two small basins, each with a volume of 5,000 acre-feet for the more frequent small runoff periods. The larger runoff volumes will flood the remaining basin and the water surface will rise in elevation over the entire reservoir.

The dewatering pumping station shown on Figure M-X-2 will discharge from the storage reservoir to the West-Southwest Treatment Plant at an average rate of about 700 cfs. The station's total capacity will be 2400 cfs in order to dewater the conveyance tunnels and Stearns Quarry into the reservoir within two or three days following a storm.

Computer studies indicate that the storage utilized in Basins 1 and 2 will exceed their combined volume (10,000 acre-feet) at an average frequency of six or seven times per year and that these two basins alone will entrap more than 70% of the annual combined sewer spillage containing over 95% of the annual Suspended Solids.

The use of a deep pit storage basin of such magnitude and depth requires that aeration be provided to insure positive odor control by floating equipment. This is necessary because the range of liquid level varies over 200 feet. It is proposed to use submerged turbine aerators provided with a downflow draft tube with air injection below the propeller.

The submerged turbine aerators will be provided with a bar screen to prevent large ice chunks from being drawn into the draft tube and damaging the blades. The aerators will be provided with legs to protect the draft tube and will need a minimum of 20 feet of water to operate. When floating at greater depths, it is considered that active aeration will be limited to the upper 50 feet of the water in storage.

Aerators, in the heaviest rainfall year will be in near continuous operation in or above Basins 1 and 2. A lesser amount of aeration on an intermittent schedule will be required in Basin No. 3.

An aerated reservoir of lesser depth and a volume of 1,800 acre-feet, will be provided near the proposed O'Hare Water Reclamation Plant, to serve the combined sewered area of the suburban communities to the northwest.

Another reservoir will utilize the existing Stearns rock quarry in the vicinity of 28th and Halsted Streets. This reservoir will provide approximately 4,000 acre-feet of storage space and will be used only during record storm events to flatten out the peak discharge through the conveyance tunnels.

Conveyance Tunnels

There are approximately 120 miles of conveyance tunnels intercepting 640 sewer overflow points in the 375 square mile area served by combined sewers. Most of the conveyance tunnels will be constructed in the Silurian Dolomite rock formation 150 to 300 feet below the surface of the waterways. In some areas, the smaller tunnels will be constructed in the clay overburden. See Figure M-X-3, 4 for profiles of the tunnels.

The tunnels will in general be drilled by mining machine (moles), except for the largest sizes which will probably be constructed by the conventional drill and blast method.

Three main conveyance tunnel systems fork out from the primary reservoir facility located in the McCook-Cummit area. See Figure M-X-1. Figure MX-1. The Des Plaines Tunnel System extends north along the Des Plaines River to the Village of Des Plaines, thence northwest terminating at the Village of Palatine. The Mainstream Tunnel System extends under the Sanitary and Ship Canal, the North and South Branches of the Chicago River and the North Shore Channel to the Wilmette controlling works. The Calumet Tunnel System extends south and southeasterly along public right-of-way to the Sag Channel, thence eastward under the Little Calumet, Grand Calumet and Calumet Rivers to near the State Line. The storage space in the conveyance tunnel system is 9,100 acre-feet.

Drop Shafts

The spillages will be delivered to the tunnels by hundreds of vertical drop shafts, capturing the present spillage from the existing riverbank sewer outlets of over five thousand miles of near-surface sewer systems. A typical drop shaft is shown in Figure M-X-5.

The drop shafts will have a split vertical shaft, one side for water and the other side for air. The center dividing wall will have slots to insufflate air in the falling water. This reduces the impact when the air-water mixture hits bottom. An air separation chamber is provided to reduce the amount of air entering the tunnel. At the top, a vent chamber will allow air to escape during filling and to be drawn in during dewatering.

Groundwater Protection and Recharge

The major project elements are sited in rock units of the Silurian System of the geologic strata underlying the Chicagoland area. These limestone and dolomite rock units, together with the hydraulically interconnected overlying glacial drift, comprise the so-called shallow aquifer of the region which is recharged by local rainfall.

Additional data on protection of groundwater and limitation of infiltration into tunnels and storage areas is available in Technical Report No. 4, Geology and Water Supply. This upper aquifer system is used as a water supply for individuals and certain municipalities. However, the water supply for the vast majority of the area is through piped systems using Lake Michigan water.

The preservation of groundwater quality and quantity is achieved by positioning the project elements in the best available rock units, taking advantage of the natural low permeabilities of the rock and augmenting this low permeability by sealing the water bearing bedding planes and joints, thus providing for elimination of the direct connections between the aquifer and the project element.

Additionally, the naturally high piezometric level within the aquifer will provide a positive inward presssure providing additional assurance against exfiltration of flows. In those areas were excessive groundwater withdrawals occur, adversely lowering the groundwater table, the added protection could be provided by artificial recharge to restore high levels around the project element. The identification of this recharge need, however, can only be made after sub-surface exploration, testing and detailed positioning of the project elements.

Benefits

A brief listing of anticipated benefits to be derived from completion of the system of flood and pollution control proposed herein, includes the following:

1. Protection of the valuable water resources of Lake Michigan from flood release of river water as now required through the existing Chicago River, the North Shore Channel and the Calumet River into Lake Michigan.

2. Achieving and maintaining acceptable water quality (in accordance with National Goals and Regulations of the Illinois Pollution Control Board and the Metropolitan Sanitary District) in the open waterways known as the Chicago River and its branches, the Sanitary and Ship Canal, the North Shore Channel, the Calumet-Sag Channel and those portion of the Calumet River, Des Plaines River, Salt Creek and other open waterways, under the jurisdiction and control of the Metropolitan Sanitary District of Greater Chicago.

3. Reduction of surface and basement flooding by underground backwaters or overbank flooding.

4. Improvement of recreational values of all surface waterways.

5. Increase in property values due to general improvement of environment.

Additional Plan Developments

The plan described in previous sections was produced in 1972. The intervening two years have provided the opportunity to incorporate new analysis and information, and improve the Plan accordingly. These up-dates can be grouped under five headings, with the revised Recommended Plan shown on the attached map (Figure M-X-6).

a. Independent Calumet System: The most significant revisions are the separation of the Calumet Area from the Central System and having an independently operating system with dewatering to the Calumet Plant.

The decision to separate the Calumet Area was based on detailed study[2] of operational flexibility and cost. This study originated with the recognition that several potential reservoir sites exist in the Calumet Area, and the cost of the 55,000-foot connection tunnel ($100 million) would be saved by independently operating systems.

The study considered three alternative concepts, each with several variations:

A. <u>Maximum Size Intertie Tunnel Plan</u>: In this plan, no storage would be provided in the Calumet area. All flow would be directed to the McCook-Summit area terminal reservoir. Economies would be realized by concentrating terminal reservoir facilities. These savings would be compared to the extra costs associated with conveyance facilities required to concentrate the storage.

This scheme is similar to the layout shown in Figure M-X-1 (the Recommended Plan from the Summary of Technical Reports) for the project area remaining after exclusion of the O'Hare sub-project area. However, these studies included drainage flow from the communities of Lansing and part of Markham which were not a part of the prior studies made in support of the Summary of Technical Reports. This additional drainage flow is included as well in all other alternatives evaluated in this study.

<u>An Intermediate Size Intertie Tunnel Plan</u>. Storage would be provided in the Calumet area but it would be an amount which would be insufficient to accommodate all of the runoff in the Calumet area during a large storm. In these instances, the Calumet area reservoirs would fill and, subsequently, flow would be diverted through the intertie tunnel to the McCook-Summit area reservoir.

No Intertie Tunnel Plan. In this scheme, storage volume provided in the Calumet area would be sufficient to accept the excess combined sewer flow in that area. The Mainstream and Des Plaines Tunnel Systems would drain to the McCook-Summit area terminal reservoir.

Several variations within each of the above basic concept plans are possible and were examined in this study so that the least-cost representation of each of the above concepts could be identified. This led to development of twelve separate project layouts for comparative evaluation, which included different reservoir locations and tunnel systems.

Initial Evaluation of Project Layouts: The evaluation and cost estimating methods employed in the initial phase of this study are identical to those used in the studies which are reported in the Summary of Technical Reports. The same basic unit costs and cost curves were used as were the computer programs previously developed. These were accepted and used to conduct simulation studies which yielded results concerning performance of the twelve project layouts. A "trial and error" procedure was employed in the development of these layouts wherein tunnel diameters and reservoir sizes were first assumed; then, for selected storm events, the system performance was simulated by electronic digital computer cooperation; and system performance deficiencies were noted upon completion of the computer run. Adjustments were then made in tunnel diameters and reservoir dimensions as indicated by the simulation analysis results. The procedure was repeated until all of the twelve project layouts satisfied the performance requirements. These performance requirements were to limit the overflow quantities during repeat of the largest storms to prevent backflow to the Lake and to treat the captured water at the existing treatment plants at a rate such that the total flow to the plant combined with dry weather flow did not exceed 1.5 times average dry-weather flow. The estimates of costs of construction of the sanitary systems were compiled using the cost parameter data developed for the prior studies.

The general approach employed in the initital evaluation phase of this study is presented here. The prior studies which are described in the Summary of Technical Reports showed that, of the 21 year continuous record of precipitation, a tunnel-reservoir system which functioned adequately in simulation analysis during the events of July 12-13, 1957 and October 3-12, 1954 would also function satisfactorily throughout the remainder of the period of record. Further, the tunnel sizes required in any given layout were dictated principally by conditions which prevailed during the 1957 storm; a storm which yielded the maximum instantaneous peak runoff flow. Also, the prior studies showed that the total reservoir storage volumes required were controlled by the conditions which obtained during the 1954 storm.

Moreover, an approximate correlation existed between the reservoir requirements established by simulation of the 1957 storm and those established by simulation of the 1954 storm.

For these initial evaluation studies, only the July 12-13, 1957 precipitation event was used in simulation analysis step of the work. This was made necessary because of the large number of computer runs required and the especially lengthy run-time of the 1954 event simulation. The July, 1957 analyses yielded the required tunnel diameters of the several schemes. The construction costs of these tunnel networks were computed. The July, 1957 analyses also indicated reservoir volume requirements for satisfactory system performance for this event. These values were extrapolated to approximate total reservoir volume requirements which would be needed for satisfactory system performance during the October, 1954 storm. Reservoir construction cost estimates were then developed.

Cost comparison of the twelve alternatives was made on the basis of the sum of the tunnel and reservoir costs. These were regarded as the controlling project costs since these two project compontents comprise approximately ninety percent of the total construction cost. Additionally, much of the remaining 10 percent of construction costs consist of modification of surface collection facilities and drop shafts, both elements being a common and near-constant cost factor for all proposed systems.

"Least Cost" Alternatives - Detailed Evaluations: The maximum size intertie tunnel plan (Scheme 1A), the intermediate size intertie tunnel plan (Scheme 2E), and the no intertie tunnel plan (Scheme 3A), having been identified as the most economicaly systems for each of the three concepts, were examined in greater detail than the remainder of the alternatives. Each of these schemes include the use of existing quarries as reservoirs. These quarries, already having depths in excess of 200 feet and large volumes available, had distinct advantages over other sites with no significant existing storage volume such as in the sludge lagoon sites. Preliminary reservoir layouts were made for these plans and more detailed construction cost estimates were prepared as shown in Table M-X-1.

The totals show the cost advantage of 3A, separation. Additional advantage is found in the freedom of construction phasing.

TABLE M-X-1 LEAST COST SCHEMES ESTIMATE OF CONSTRUCTION COSTS; MILLIONS OF DOLLARS (Based on a Limitation of Stockpile Height of 200 Feet in the McCook-Summit Area)

Sum of Tunnels and Reservoirs (1972 Costs, Unescalated)

Item	Scheme 1A	Scheme 2E	Scheme 3A
1. McCook-Summit Reservoir[2]	496	351	286
2. Thornton Reservoir[2]	-	56	74
3. Mainstream On-Line Reservoir	15	15	15
4. Tunnels[1]	691	568	552
5. Pumping Stations			
a. McCook-Summit	71	65	64
b. Stearns Quarry	8	8	
c. Thornton-Calumet	-	30	30
TOTAL, without contingencies	1,281	1,093	1,029

[1]Estimates of tunnel cost require a determination of whether or not they are concrete-lined. The final decision concerning concrete lining must be reserved for the design phase of the project and completion of subsurface investigations. This decision cannot alter the conclusions of this optimization study because all project layouts will be similarly affected.

[2]Includes credit for future sale of rock and other future land values.

As a result of these studies, the interconnecting tunnel has been eliminating and planning and subsurface explorations have been concentrated on development of the plan with an independently operating Calumet System.

b. Palatine Tunnel Elimination

An additional revision is the elimination of a tunnel leg into the Village of Palatine. A report titled "Preliminary Engineering Study of Palatine, Illinois for Intercepting and Holding Combined-Sewage Overflows" was completed in September, 1973. Eighteen alternate solutions were studied. The lowest cost alternate for a tunnel and reservoir concept system for Palatine area to connect to the O'Hare System, as originally proposed was $22,700,000. Of the alternates discharging to Salt Creek Water Reclamation Plant, the lowest cost was $17,300,000; however, the recommended tunnel and reservoir concept system was estimated at $18,900,000. It was further established that construction of a separate new five-year storm sewer system for the combined-sewer area would cost $11,100,000 and the construction of a new sanitary sewer system would cost an estimated $12,700,000. Either a new storm sewer system or new sanitary sewer system would eliminate combined-sewer overflows in this drainage basin.

At the regular Board Meeting of April 22, 1971, the Board of Trustees of the District approved the U.S. Soil Conservation Service Upper Salt Creek Watershed Work Plan. This plan, of which the District is a local sponsor, is a comprehensive program which will prevent overflow of Salt Creek in the Palatine area. The District has already committed $4,861,000 for seven projects under the Work Plan. In view of this program, the flood control benefits of the District's Tunnel and Reservoir Plan for the Palatine area will not be required.

The estimated cost of the tunnel and reservoir plan exceeds the estimated cost of separating sewers within that portion of the Village of Palatine which has combined-sewers; and therefore, the tunnel and Reservoir concept is not the cost-effective method for preventing discharge of combined-sewage to the waterways.

Therefore, the Palatine tunnel leg in the Northwest Area of the District has been eliminated and at meetings with the Village of Palatine the Village Officials have been so informed.

c. Mainstream Dual Tunnel System

A third revision or updating includes dual tunnels for the Mainstream System from Summit to Lawrence Avenue. The August 1972 Plan included a 42-foot diameter or equivalent along this reach. In order to maintain a uniform slope, this tunnel would have to be constructed through the Maquoketa shale. This shale formed from clay sized particles is a softer rock than the dolomitic limestone, and presents different stability considerations for both long-term use and during construction. Thus, the construction in this shale will be more costly than in limestone.

During 1974, additional rock borings have been made to further identify the location of this shale formation and additional analysis have been performed. As a result, it has been determined that the most cost-effective plan is to construct a smaller tunnel first at the required slope and at a latter date construct a second tunnel which would be totally in the limestone formations. Each of the dual tunnels would provide one-half the required conveyance capacity.

TABLE M-X-2 TUNNELS MAINSTREAM SYSTEM (McCook to Confluence)

	SINGLE TUNNEL			DUAL TUNNELS			
				Stage I		Stage [illegible]	
LINE	LENGTH	DIA.	COST $ x 1000	Dia.	COST $ x 1000	DIA.	CO[illegible] $ x [illegible]
2	6,930	45	$ 17,000	33	$ 10,570	35	$ 11,[illegible]
3	7,095	42	15,600	33	10,820	35	7,610
4	13,575	42	29,800	33	20,700	35	14,930
5	8,00	42	17,600	33	12,200	35	8,800
6	8,600	42	18,900	33	13,120	35	9,460
7	7,332	42	16,100	30	9,750		
8	8,368	42	18,400	30	11,130		
9	10,992	42	24,100	30	14,620		
10	11,718	42	25,700	30	15,590		
11	8,285	42	18,200	30	11,020		
12	4,095	42	10,800	30	6,520		
13	5,451	42	11,900	30	7,250		
14	4,104	42	9,010	30	3,650		
15	3,823	42	8,390	30	3,400		
16	2,865	42	6,290	30	2,550	35	3,150
48	24,400					35	26,840
49	11,300					35	12,430
50	14,580					35	16,038
SUBTOTAL			$247,790		$152,890		$110,958
TOTAL			$247,790			$263,848	

Cost shown exclude shafts, connecting structure and contingences

The Table M-X-2 compares the construction costs of the single tunnel with the costs of the dual tunnel system. As can be seen the cost difference is $16 million or 6 per cent of the total. This difference is insignificant when considering the total costs of the program. Therefore, also considering the uncertainties of grant funding, the dual tunnel system is chosen because the first tunnel can be constructed at a lower initial cost and provide for the capture of in excess of 80 per cent of the pollutants now discharged to the waterways, and thus obtain an early return on the invested funds. The dual tunnel system also offers time to further optimize the size needed for the second tunnel to meet project objectives.

d. Mainstream On-Line Reservoir

Another revision has been elimination of Stearns Quarry as the location of the Mainstream On-Line Detention Reservoir. This reservoir has been included in the Plan in order to reduce the size and cost of the Mainstream Tunnel System. Without such a reservoir, the lengthy Mainstream Tunnel would have to increase in size from a 42-foot diameter equivalent tunnel to a 55-foot diameter equivalent tunnel[(2)].

The Stearns site is now being filled in and will eventually be used as a park. The previous studies had considered this site only in relation to evaluating different alternatives of solving the combined-sewer overflow problem, and not in terms of the specific details of the site. It has now been determined that the best and highest use of the site is as a park.

In its place, a Mainstream On-line Reservoir at an unidentified site is included. The location of this reservoir can be anywhere along the tunnel between the Stearns site and Wilmette Harbor. Since this reservoir will be used only during the large storms of record to reduce the peak flow rates to the tunnels, it will be one of the last facilities to be constructed. If a suitable site is not found for the reservoir, the alternative does exist to increase the size of the second tunnel of the Mainstream Dual Tunnel System.

e. Des Plaines Watershed

A fifth revision provides for the total capture of the combined-sewer overflows in the Des Plaines River Watershed. The August 1972 Plan included the equivalent of a Mod 3 level of capture for the sizing of the Des Plaines River Tunnel and the O'Hare Northwest System. However, as was stated in the report, total capture would be needed in order to meet the higher water quality standards of the General Use and the Public and Food Processing Water Supply Designated waterways. (See Figure M-X-7.) This was the case in the Little Calumet River and the North Branch of the Chicago River Upstream with its junction with the North Shore Channel. Tunnel sizing is provided so as not to spill into these waterways.

The higher water quality standards for the above waterways includes, among others, the requirement that the dissolved oxygen level shall not fall below five milligrams per liter at any time. The computer simulation of the overflows into the Chicago and Calumet River Systems (designated as secondary contact waters) demonstrates that depression of the oxygen levels below five milligrams per liter will occur. Since these latter waterways have a lower water quality designation and instream aeration is to be provided the Mod 3 level of protection is judged to be adequate. However, it is not judged to be adequate in the Des Plaines River Watershed. A computer model of the Des Plaines River has not been developed as for the above waterways. However, the same level of contaminants would be discharged during overflow and the results can be expected to be the same.

Therefore, the plan now includes reservoir capacity in the O'Hare Area to provide for total capture and increased sizing in the Des Plaines River tunnels to transport a higher rate of flow to the Mainstream-Summit Reservoir.

In the O'Hare Area, the August 1972 Report provides for an 1800 acre-foot reservoir including capacity for Palatine. Subsequent Analysis (3) demonstrates that 1280 acre-feet is required for the equivalent of Mod 3 level storage for the O'Hare sewered area and 2700 acre-feet for total capture. Provision for the latter quantity of storage is being included in the Plan. The sizing of the tunnels is not changed because of the decision to include total capture. The tunnel size has been chosen to transport dry-weather flows to the O'Hare Plant, and on the basis of transporting peak storm flows with an On-line reservoir providing for the detention of peak flow rates so as to control the tunnel surcharge. The cost of both the 2700 acre-foot O'Hare terminal storage reservoir and the On-Line reservoir is included in the Plan.

The Des Plaines River Tunnel has been increased in size to provide for total capture. This size increase is shown on Table M-X-3. The original size was picked such that spillage to the waterways occurred during a repeat of the July 1957 storm, (the storm of record for rate of flow) at approximately the same ratio to total flow as in the Chicago and Calumet River Systems. During the 1954 storm (the storm of record for total volume), there was no spillage to the Des Plaines River and the total flow was transported to the reservoir. Thus, no additional storage volume is needed to provide for total capture and only increased tunnel sizing is required.

TABLE M-X-3 TUNNELS- DES PLAINES RIVER SYSTEM

LINE	LENGTH (ft.)	TOTAL CAPTURE		MoD 3 LEVEL PROTECTION	
		DIA. (FT.)	COST ($ x 1000)	DIA. (FT.)	COST ($ x 1000)
18	8800	36	15,224	30	11,704
19	5040	15	2,974	15	2,974
20	11,280	10	3,948	15	6,655
21	9460	36	16,366	30	12,582
22	12,200	32	17,690	25	12,810
23	11,200	15	6,608	15	6,608
24	8160	32	11,832	25	8,568
25	14,390	32	20,866	25	15,110
26	5100	32	7,395	25	5,355
27	8600	24	8,600	25	9,030
28	7050	24	7,050	25	7,403
29	10,380	24	10,380	20	8,304
30	6470	24	6,470	20	5,176
TOTAL	118,130		135,403		112,279

Cost shown exclude shafts, connecting structures and contingencies

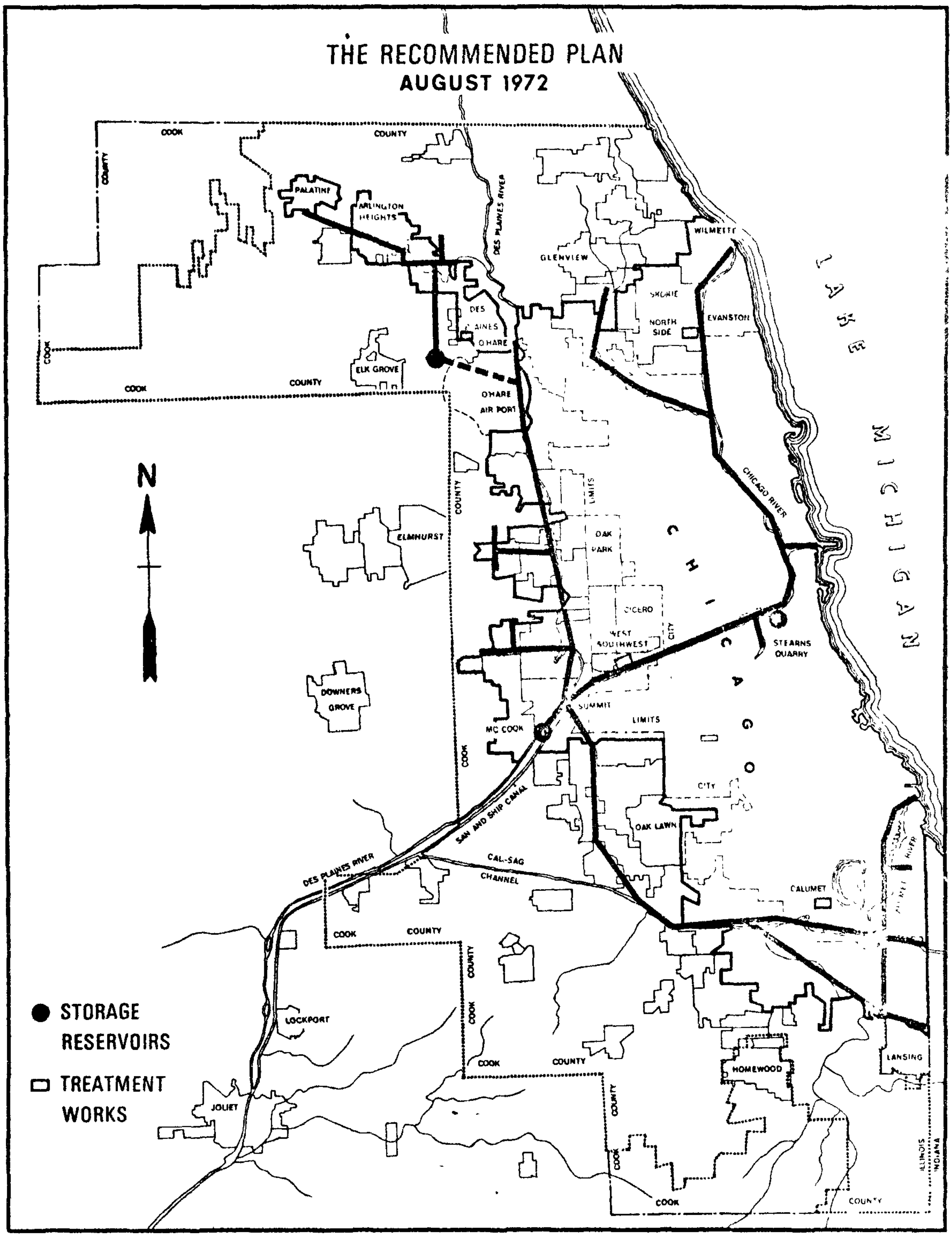
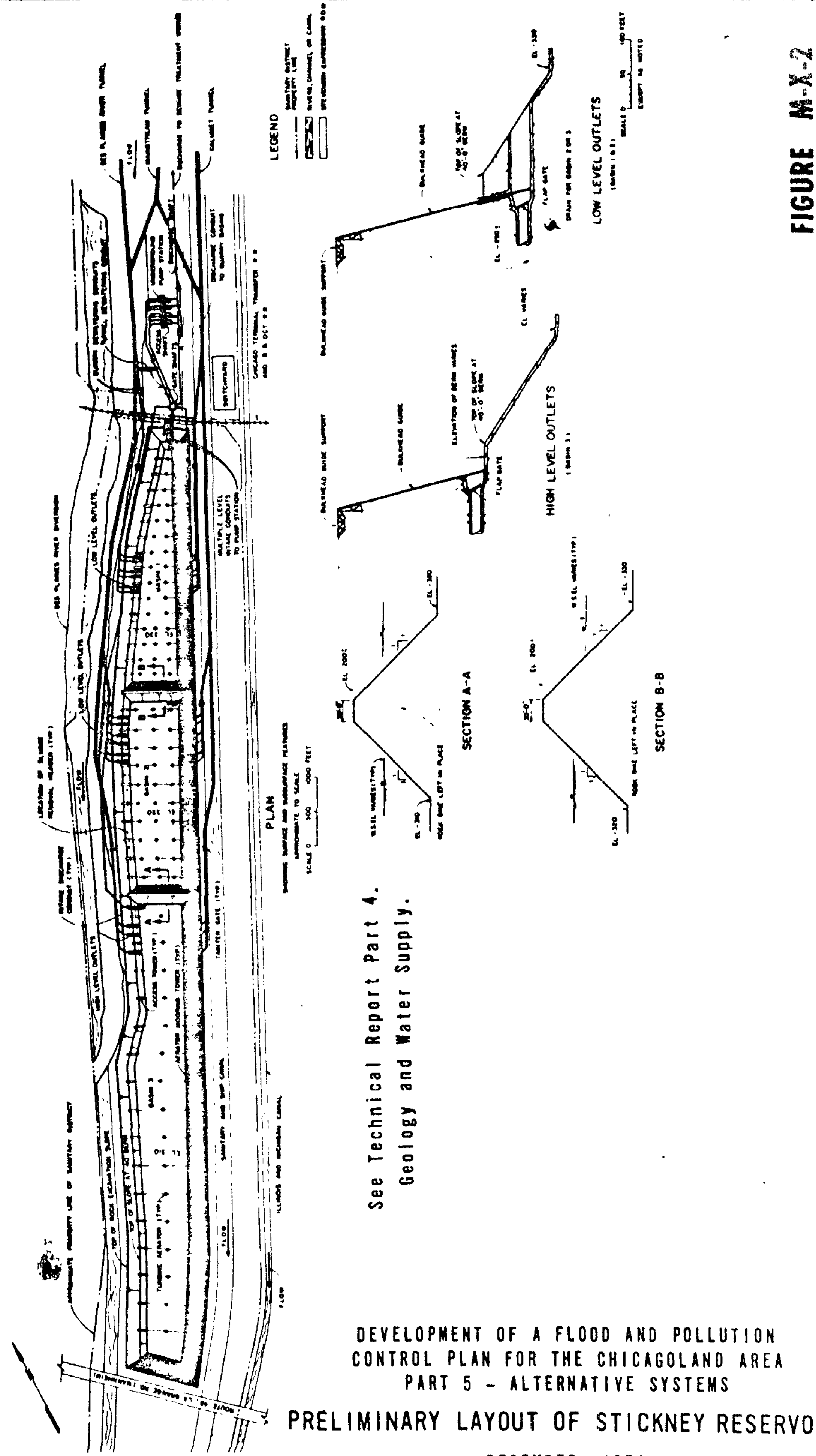

THE RECOMMENDED PLAN
AUGUST 1972
STORAGE RESERVOIRS
TREATMENT WORKS
N
COOK COUNTY
PALATINE
ARLINGTON HEIGHTS
DES PLAINES RIVER
GLENVIEW
WILMETTE
SKOKIE
NORTH SIDE
EVANSTON
DES PLAINES
O'HARE
ELK GROVE
O'HARE AIR PORT
LAKE MICHIGAN
CHICAGO RIVER
ELMHURST
OAK PARK
CHICAGO
CICERO
WEST SOUTHWEST
CITY LIMITS
STEARNS QUARRY
DOWNERS GROVE
SUMMIT
MC COOK
SAN AND SHIP CANAL
OAK LAWN
CAL-SAG CHANNEL
DES PLAINES RIVER
CALUMET
LOCKPORT
JOLIET
HOMEWOOD
LANSING
ILLINOIS
INDIANA

FIGURE M-X-1

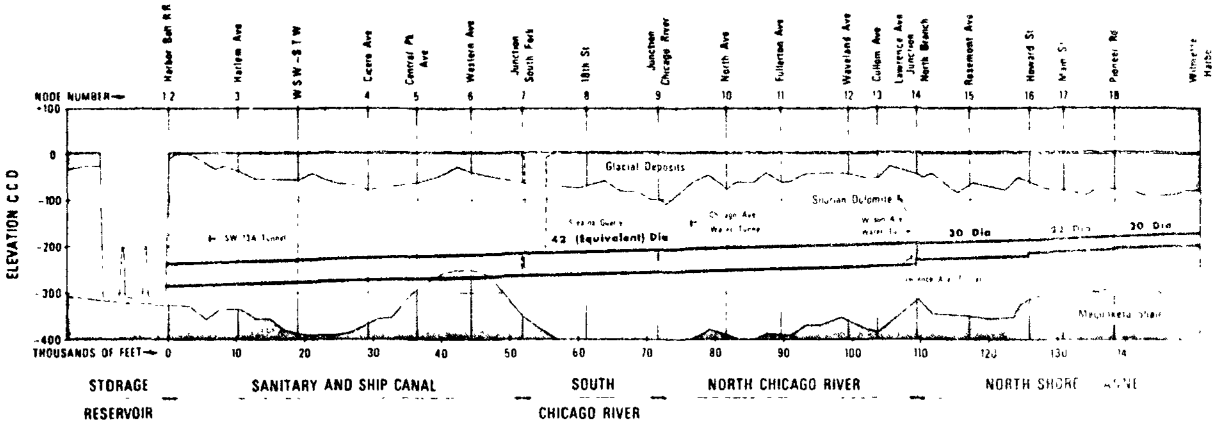


See Technical Report Part 4.
Geology and Water Supply.

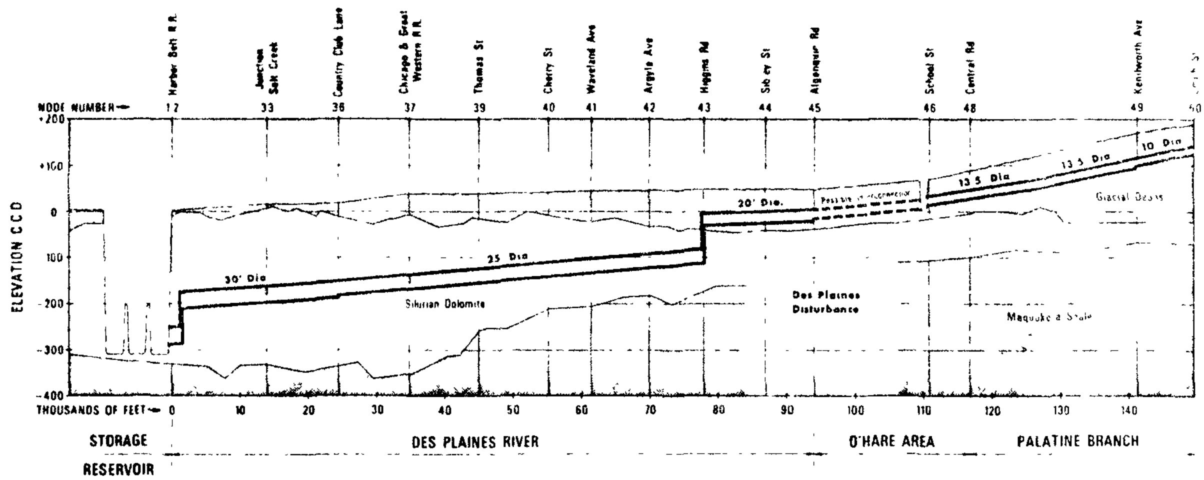
DEVELOPMENT OF A FLOOD AND POLLUTION
CONTROL PLAN FOR THE CHICAGOLAND AREA
PART 5 - ALTERNATIVE SYSTEMS

PRELIMINARY LAYOUT OF STICKNEY RESERVOIR

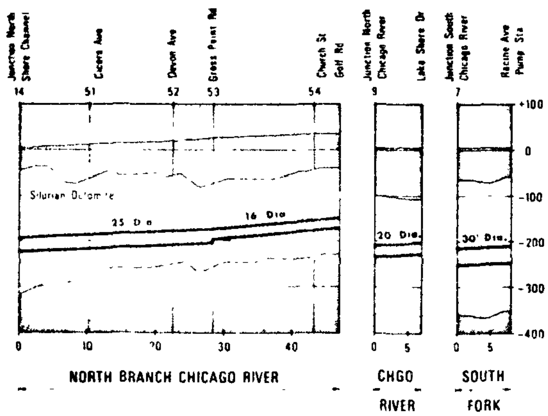
DECEMBER, 1972



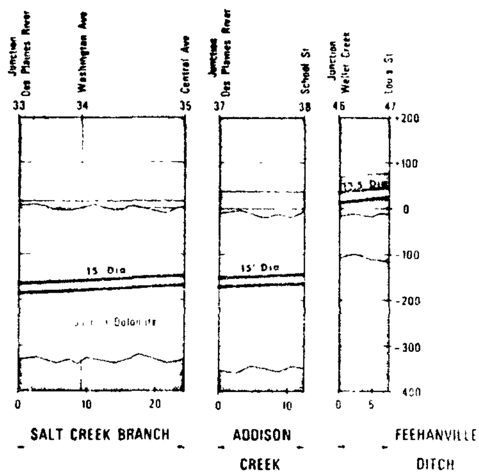
MAINSTREAM SYSTEM



DES PLAINES RIVER SYSTEM



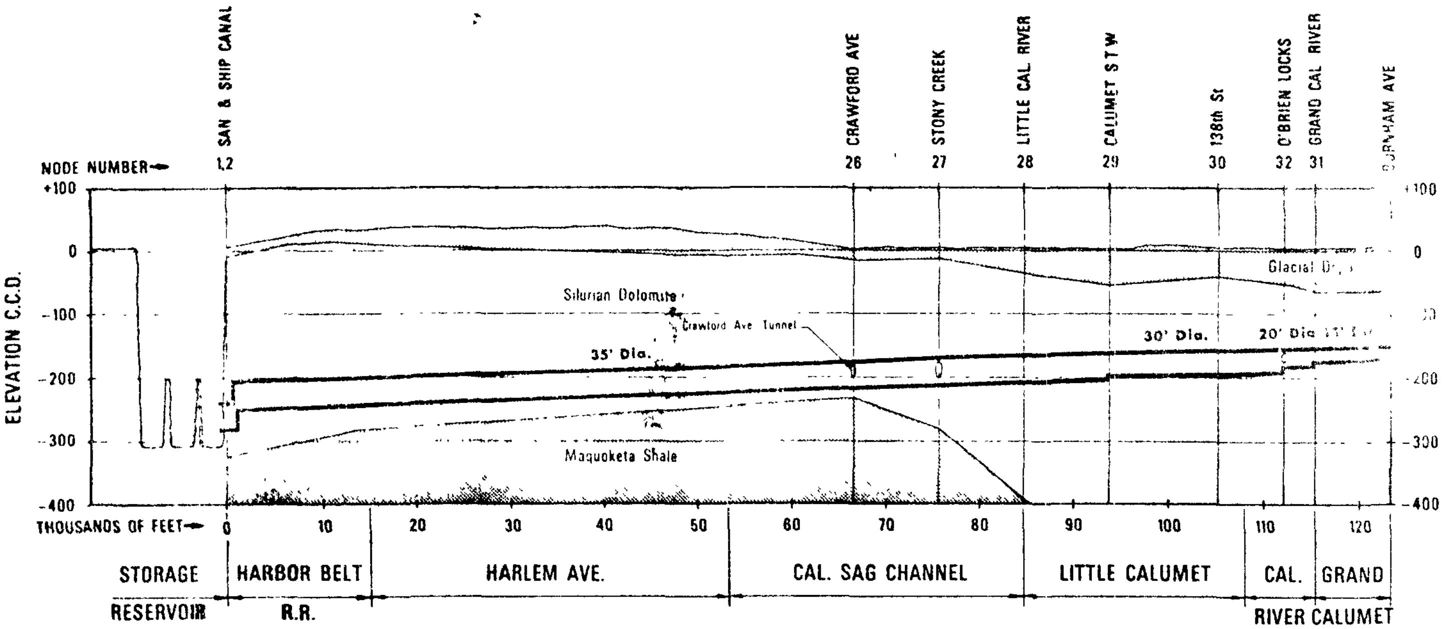
MAINSTREAM BRANCHES



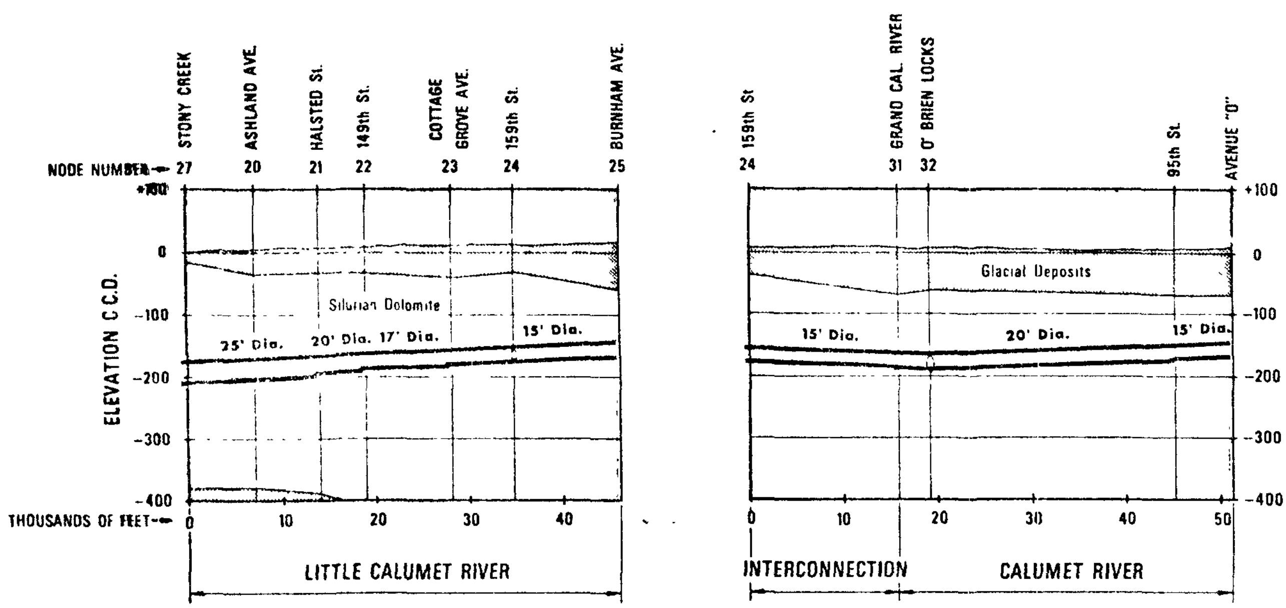
DES PLAINES RIVER BRANCHES

TUNNEL PROFILES

FIGURE M-X-3



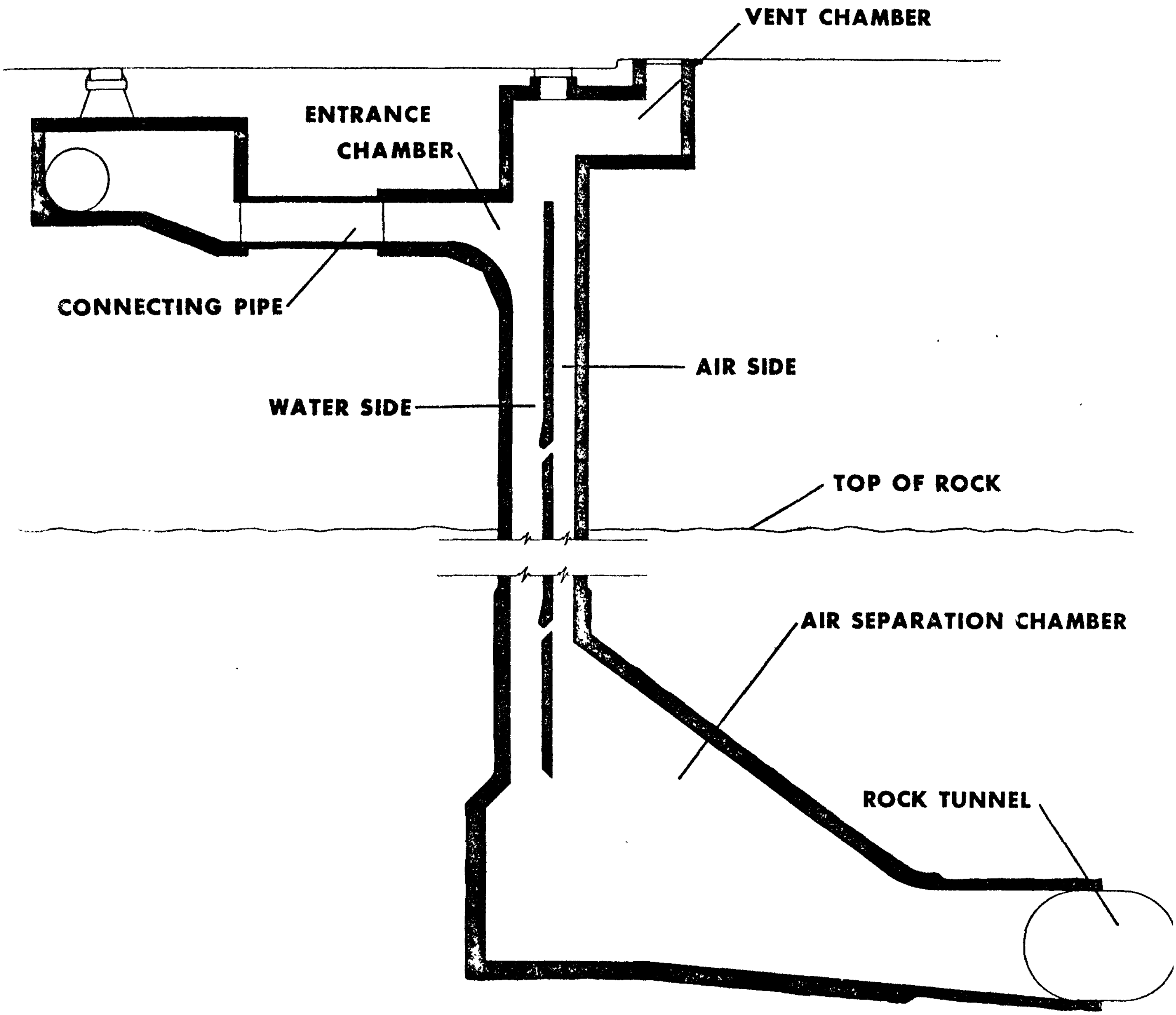
CALUMET SYSTEM



CALUMET BRANCHES

TUNNEL PROFILES

FIGURE M-X-4



TYPICAL DROP SHAFT STRUCTURE

FIGURE M-X-5

TUNNEL AND RESERVOIR PLAN

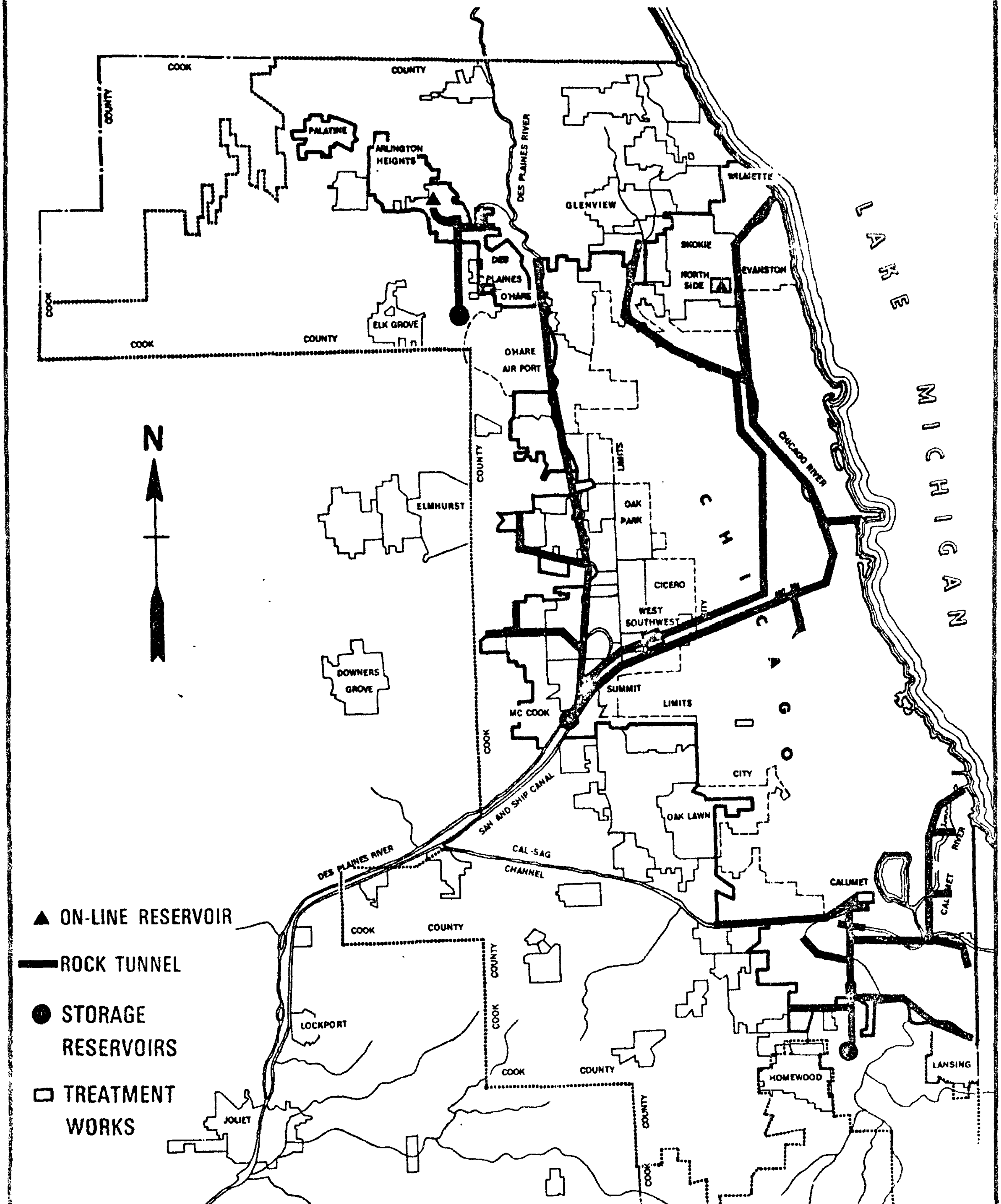


FIGURE M-X-6

NOV. 1974

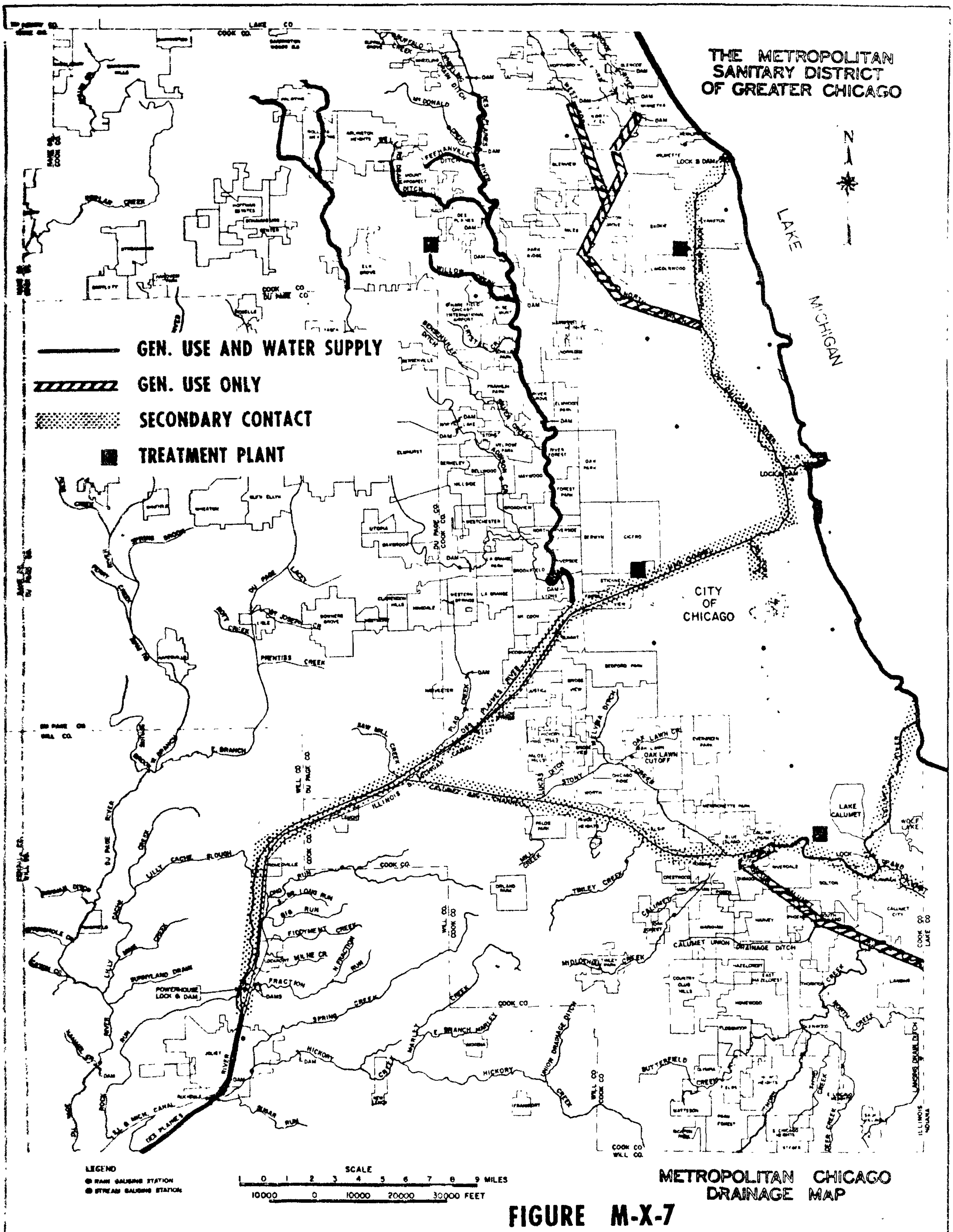

THE METROPOLITAN
SANITARY DISTRICT
OF GREATER CHICAGO
GEN. USE AND WATER SUPPLY
GEN. USE ONLY
SECONDARY CONTACT
TREATMENT PLANT
LAKE MICHIGAN
CITY OF CHICAGO
LAKE CALUMET
DES PLAINES RIVER
DU PAGE RIVER
CALUMET UNION DRAINAGE DITCH
POWERHOUSE LOCK & DAM
LEGEND
SCALE
0 1 2 3 4 5 6 7 8 9 MILES
10000 0 10000 20000 30000 FEET
METROPOLITAN CHICAGO
DRAINAGE MAP

FIGURE M-X-7

APPENDIX C

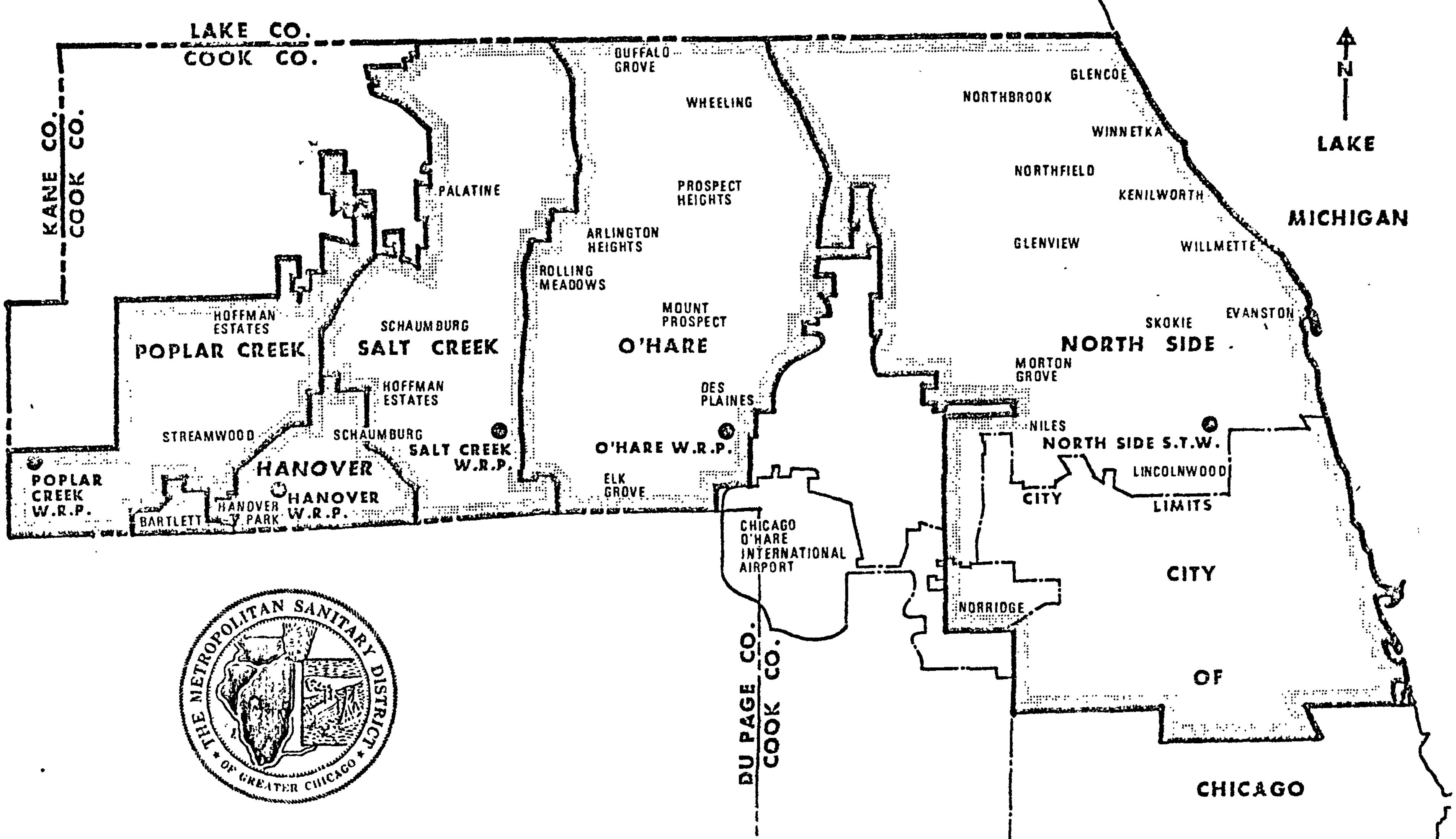
THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

POSITION PAPER ON SELECTION OF UPPER DESPLAINES SERVICE BASIN PLAN OVER OTHER SUGGESTED ALTERNATES

The O'Hare Facility Area (Upper DesPlaines Service Basin) is a 58 square mile area in the northwest region of the Metropolitan Sanitary District. At present, all sanitary sewage and the combined sewage finding its way into District interceptors through regulated control structures, is diverted through existing interceptors to the District's Northside Sewage Treatment Works for treatment. Initially, following annexation into the District of most of the area in 1956, it was planned to transmit sewage flows for the District's northwest area to the West-Southwest Sewage Treatment Plant in Stickney for treatment. However, further study indicated the cost-effectiveness and desirability of dividing the northwest area into four (4) service areas. This Paper gives, in detail, the planning history and rationale of dividing the northwest area of which the O'Hare Facility Area is a part, into four (4) facility (service) areas.

Collection and treatment of sewage generated in this basin has been the subject of many studies and reports. In 1961, the Sewer Design Section of the Metropolitan Sanitary District recommended that the Northwest Intercepting System be constructed to relieve existing sewers in the northwest portion of the District to provide service for the ultimate development of that area (Ref.1). The northwest area comprises the O'Hare, Salt Creek, Hanover Park and Poplar Creek (Elgin) Facility Areas as shown in Figure 1. The consulting firm of Greeley and Hansen was retained to investigate this proposal. Based on their report, submitted in 1962, a tentative decision was made to convey all sewage from the area to the West-Southwest Treatment Plant (Ref.2). Further investigation of this proposal indicated that the cost and magnitude of the project would require such time and resources as to necessitate construction of temporary plants in the northwest area (Refs.1,2). Additional studies and investigations carried out primarily due to the trend toward higher standards for disposal of treated effluent, indicated the advisability of collecting and treating the sewage from each facility area in the northwest area separately, since construction of temporary tertiary treatment plants, of the magnitude indicated, would not be cost-effective (Ref.3). Furthermore, the District considered that diversion of substantial quantities of water from the northwest area would not be conducive to water reuse. The utilization of tertiary quality effluents for stream augmentation, within the area, was considered to have

NORTHWEST REGION — SERVICE AREAS



THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO environmental and recreational benefits. The Northwest Intercepting Sewer proposal would have diverted all sewage flows from the area for treatment at West-Southwest Treatment Plant and treated effluent would be discharged into the Sanitary and Ship Canal at Stickney.

As a consequence, the Northwest Area was divided into Facility Areas corresponding to existing drainage basins: Upper DesPlaines Service Basin, O'Hare Facility Area; Upper Salt Creek Service Basin, Salt Creek Facility Area; Upper DuPage Service Basin, Hanover Park Facility Area; and Poplar Creek Service Basin, Poplar Creek (Elgin) Facility Area.

A preliminary design concept for the O'Hare Water Reclamation Plant and intercepting sewers was prepared in report form by Brown and Caldwell, consulting engineers, in 1968 (Ref.4). The contract plans for O'Hare Water Reclamation Plant have been prepared by Consoer, Townsend and Associates, consulting engineers, and are presently under review by the District. Preliminary plans for two (2) collection facilities systems were prepared in a report by DeLeuw, Cather & Company, consulting engineers (Ref.5). One system would divert total sanitary sewage flow only within the basin to the O'Hare Water Reclamation Plant; the second system would convey all sanitary sewage to the plant but would, in addition, eliminate combined sewer overflows by collecting and storing for treatment, all combined sewer overflows presently discharging to waterways within the drainage area. They are presently developing contract plans for the O'Hare Tunnel System as part of the second system.

In 1973, The Corps of Engineers published the Chicago-South End of Lake Michigan Study (Ref.6). The investigation included the O'Hare Facility Area and the O'Hare Water Reclamation Plant was defined in three of the five alternates presented in the Report.

Northeastern Illinois Planning Commission (NIPC) included the District's plan for the O'Hare Facility Area in its "Regional Wastewater Plan" in 1971 (Ref.7). This plan has been revised a number of times since (Refs.8,9,10,11,12) but revisions have not affected the O'Hare Facility Area except to include the District's Tunnel and Reservoir Plan as a regional solution to the combined sewer overflow problem. The first three NIPC Plan revisions have been certified by the State of Illinois and the Federal Government in accordance with 40 CFR 35.565 (Refs.8,9,10).

The review of the planning history indicates that the division of the District's Northwest area into four facility areas is cost-effective and environmentally sound and has been recognized by NIPC, the State of Illinois and the Federal Government. The District's comprehensive plan calls for collection and treatment of sewage within each facility area separately. For the O'Hare Facility Area, the District has planned a collection system designated as O'Hare Tunnel System and a treatment facility, O'Hare Water Reclamation Plant.

It is, therefore, clear that the selection of the Upper Des Plaines Service Basin Plan over other alternates suggested was a sound judgment, based on a number of detailed engineering studies concurred in by a multitude of governmental agencies. Based on the results of the studies and concurrence of the applicable regulatory and planning agencies, as well as the majority of the affected communities, decisions have been made and actions taken over a number of years which weigh even more heavily in favor of continuing the proposed course of action as expeditiously as possible. The initiation of new studies and reconsideration of numerous alternates suggested by individuals untrained in the relevant fields of endeavor and uninformed in the history of past decisions and the multitude of facts and data drawn upon in making these decisions is unwarranted.

REFERENCES:

1. MSDGC, "Recommendation for Site Acquisition for Additional Sewage Treatment Plants for Northwest Section of Cook County, Salt Creek and DesPlaines River Areas," June 25, 1964

2. Greeley and Hansen, "Proposed West and Northwest Sewers," MSDGC, 1962

3. Greeley and Hansen, "Report for Northwest Area", MSDGC, 1968

4. Brown and Caldwell, "Design Report, O'Hare Reclamation Plant", MSDGC, 1968

5. DeLeuw, Cather & Company, "Preliminary Plans for O'Hare Collection Facility", MSDGC, 1972

6. Corps of Engineers, "Chicago South End of Lake Michigan Study", Chicago District, 1974

7. Northeastern Illinois Planning Commission, "Regional Wastewater Plan", March, 1971

8. Northeastern Illinois Planning Commission, "Regional Wastewater Plan", Revised September, 1971

9. Northeastern Illinois Planning Commission, "Regional Wastewater Plan", Revised October, 1971

10. Northeastern Illinois Planning Commission, "Regional Wastewater Plan", Revised January, 1972

11. Northeastern Illinois Planning Commission, "Regional Wastewater Plan", Revised July, 1972

12. Northeastern Illinois Planning Commission, "Regional Wastewater Plan", Revised October, 1972

APPENDIX D

BEDROCK GEOLOGY

This description of the general bedrock geologic conditions in the study area is complemented by the analysis of field borings as reported in The Geotechnical Report on the Upper Des Plaines Tunnel and Reservoir Plan, Volume 1, Bedrock Geologic Investigation, 1974, De Leuw, Cather & Company.

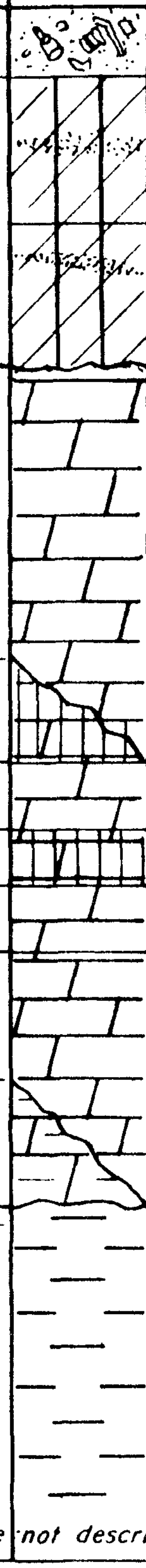
The limestone and dolomite rock units, together with the hydraulically interconnected overlying glacial drift, function as an aquifer.

Rocks in the project area date back to the Upper Ordovician Period. They consist of mudstones, argillaceous dolomite, pure dolomite and unconsolidated or semi-consolidated glacial deposits. Stratigraphic hiatuses (disconformities) occur between Middle Silurian and Pleistocene, and between the Lower Silurian and Upper Ordovician age rocks. Local lensing may totally remove some rock units (Waukesha Formation). See Exhibit II-1.

The lowering of sea level which produced the disconformity at the end of the Ordovician period resulted in local valleys cut as much as 150 feet deep in the underlying mudstones. With the re-advance of the sea, these valleys were subsequently filled with the shaley dolomite of the Edgewood Formation, which was not

System	Series	Formation/Member		Column	Description
QUATERNARY	Recent				
QUATERNARY	Pleistocene	WADSWORTH MEMBER			Till and outwash deposits. Clayey silt with sand lenses. (Gravel lenses possible but not probable - described in soils report.)
QUATERNARY	Pleistocene	WEDRON FORMATION			Bouldery till, clayey silt with sand lenses, gravel, boulders common near base and at unconformity. (Described in soils report.)
SILURIAN	Niagaran	RACINE (0-300')			Gray-brown, argillaceous, fine grained, thin bedded dolomite containing reefs of pure, gray, massive, vuggy, dolomite.
SILURIAN	Niagaran	(WAUKESHA) (0-20')			Gray, fine grained, silty dolomite. (Generally absent in northern area.)
SILURIAN	Niagaran	JOLIET (40-70')	Romeo		Light gray, pure, porous dolomite.
SILURIAN	Niagaran	JOLIET (40-70')	Markgraf		Light gray, silty, very fine grained dolomite
SILURIAN	Niagaran	JOLIET (40-70')	Brandon Bridge		Red or greenish gray dolomite and interbedded shale.
SILURIAN	Alexandrian	KANKAKEE (20-50')			Light brown, fine grained dolomite with prominent wavy clay partings.
SILURIAN	Alexandrian	(EDGEWOOD) (0-100')			Brown to gray shaley dolomite. (Cherty near top. Not recognized in project area)
ORDOVICIAN	Cincinnatian	Maquoketa Group	NEDA (0-15')		Oolite and red shale (Generally absent)
ORDOVICIAN	Cincinnatian	Maquoketa Group			Oolite and red shale. (Generally absent)
ORDOVICIAN	Cincinnatian	Maquoketa Group	BRAINARD SHALE (0-100')		Green to brown fossiliferous mudstone

Base not described



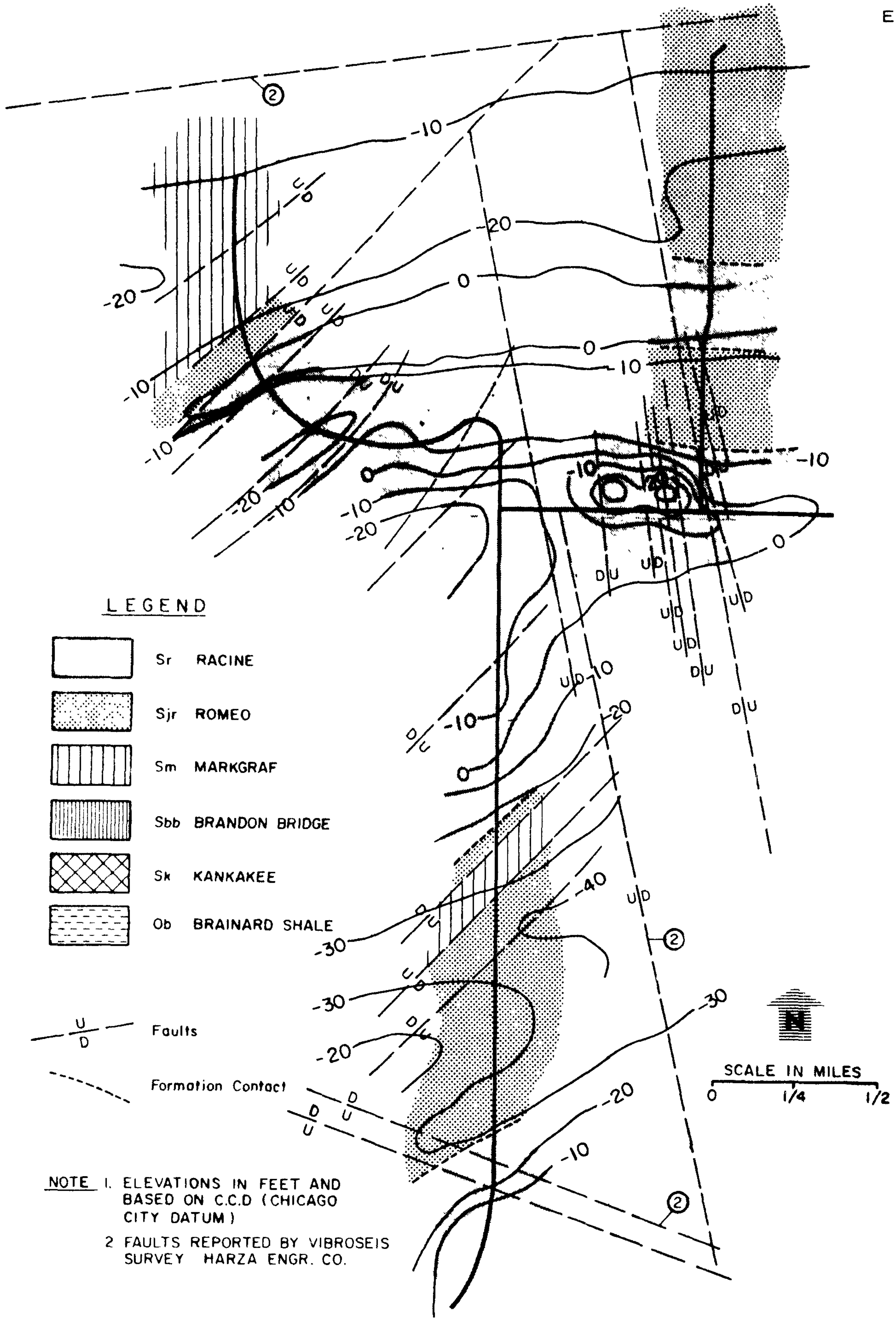
STRATIGRAPHIC SEQUENCE

recognized in any of the rock cores recovered during the explorations reported herein, but which may exist in local areas of the project.

The stratigraphic sequence used in this report has been developed from rock cores taken along the project alignment. No surface rock exposures are available for study. The rock units used follow as closely as possible formational units as described in the literature (Willman, 1971 and 1973), but vary somewhat in the designation of formational member units as the contacts between member units are gradational and thus subject to personal judgment.

Bedrock Surface

The bedrock surface is covered by glacial till throughout the project area; the bedrock contours shown in Exhibit II-2 are based totally on interpretation of boring data and are generalized. The



CONTOURS ON TOP OF ROCK AND BEDROCK GEOLOGY

bedrock relief in the project area is over 60 feet.

The rock units at the till/rock contact are the Niagaran age dolomites of the Racine and Joliet Formations. The top of rock surface is usually broken and open for five to ten feet and occasionally carries significant quantities of water.

Stratigraphy

A detailed discussion of the stratigraphy of the study area can be found in the Foundation Science Report, A Geotechnical Report on the Upper Des Plaines Tunnel and Reservoir Plan, prepared for De Leuw, Cather & Company.

Structure

The geologic structure described in this report is based solely on interpretation of boring data. No exposures of bedrock are available for study in the project area. Therefore, the situation presented in maps and sections must be used in its broad diagrammatic sense only.

Faults. A number of faults of ten to 30 feet vertical offset are postulated. The faults are interpretive and have not been physically observed, but they do further explain stratigraphic facts developed by the boring program. None of the borings actually intersected a major fault zone. In many cases, stratigraphic offsets of a single fault are probable--the result of more than one fault of smaller proportion stair-stepped to produce the total offset.

With borings spaced at 1,000-foot centers, as in this program, it is also possible for fault blocks of greater or lesser proportion than those shown to exist between borings and not be represented on geologic maps and sections. The general structural trend for faults is expected to be NE-SW and NNW-SSE. The actual fault trends indicated on the accompanying map are interpreted from apparent bedrock drainage displacements and may or may not in fact represent actual conditions.

Three angle borings were drilled to try to sample a fault zone. No significant amounts of faulted or gouged rock could be identified in any of these holes although a few core loss zones occurred which may represent small shear zones. However, the rock on either side of the core loss zones does not appear bisected as might be expected if a major fault zone were nearby.

The Des Plaines Disturbance. The project is immediately west of a geologic anomaly known as the Des Plaines Disturbance. This disturbance is a five- and one-half-mile-diameter area having a very complex system of high-angle faults. Fault displacements along the edges of the disturbance range from 50 to 300 feet, and in the interior reach 900 feet. Thus, faulting may be aptly described as severe, and the fault system may be expected to extend some considerable distance away from the boundary of the disturbance. Shear zones associated with the fault system would cause support problems and would yield high volumes of water. Because of the anticipated areal faulting, water infiltration problems would probably be severe. Also, because of the offsets across individual faults, it would be difficult to avoid the contacts between rock members, again adding to water inflow and support problems.

Jointing. Joints, although not numerous in the rock, have a significant influence on its permeability and local tunnel stability. The open joints act as a conduit to carry groundwater from the overlying glacial till to the bedding planes in the upper rock units (Racine and Joliet Formations). The numbers of the open joints are relative indicators of the amount of groundwater to be expected in a given rock. These data clearly demonstrate the more open and permeable natures of the Racine and Upper Joliet (Romeo) Formations.

Earthquakes. In the past, damage from earthquakes has not been extensive or severe in Northern Illinois. Past disturbances have ranged in intensity up to VII on the Mercalli scale, but a VII intensity was recorded only once in history. Presently, the study area is in Seismic Risk Zone 1, an area predicted to experience only minor damage from earthquakes that have the epicenters outside the state. Intensities of the disturbances are predicted to range from V to VI on the Mercalli scale. A V intensity disturbance is felt by nearly everyone and breaks glassware and windows. A VI intensity disturbance is felt by everyone; objects are upset; and chimney and plaster damage occurs.

SOILS AND SURFICIAL GEOLOGY

The basic drainage patterns, landforms and soil parent materials are related to the Wisconsin glaciation. Glacial deposits may approach depths of 60 feet in this area. The textural composition of these morainal till deposits range from clay to clayey silt, with varying amounts of sand, gravel and boulders. At the earth tunnel depth, the soils range from clayey silts to silty clays, with occasional sand and gravel. In this area, the Tinley Moraine directly overlies the Valparaiso Moraine. The Tinley Moraine, predominately a silty clay, may contain waterbearing sand layers.

REGIONAL WATER RESOURCES AND NEEDS

SECTION A

Sources of Water Supply

2.01 GENERAL At the present time, Lake Michigan and groundwater are the sources for public water supply in northeastern Illinois. Groundwater is developed from four aquifer systems: 1) sand and gravel deposits in the glacial drift; 2) shallow dolomite formations; 3) the Cambrian-Ordovician aquifer; and 4) the Mt. Simon aquifer. For purposes of this report, the sand and gravel deposits and the dolomite formations are collectively referred to as the shallow aquifers, while the Cambrian-Ordovician aquifer is referred to as the deep sandstone aquifer. The Mt. Simon aquifer is considered separately and discussed to a lesser degree because it is virtually unused at present. While there are several surface streams flowing through the region, only the Kankakee River has been seriously considered as a source of public water supply.

2.02 LAKE MICHIGAN Lake Michigan is the most extensively used water source. It provided 1,105 million gallons per day (MGD) for public water supply in 1970, or about 85 percent of the region's total public water supply needs.(1) From a purely physical standpoint, the Lake offers an almost limitless amount of water that can be readily treated to acceptable drinking water quality. It should be noted that no water flows naturally from the Lake to the region. That which is withdrawn for water supply and other purposes constitutes a diversion. Currently, this diversion is limited to 3,200 cubic feet per second (cfs), or about 2,080 MGD, as a result of a 1967 U.S. Supreme Court Decision.

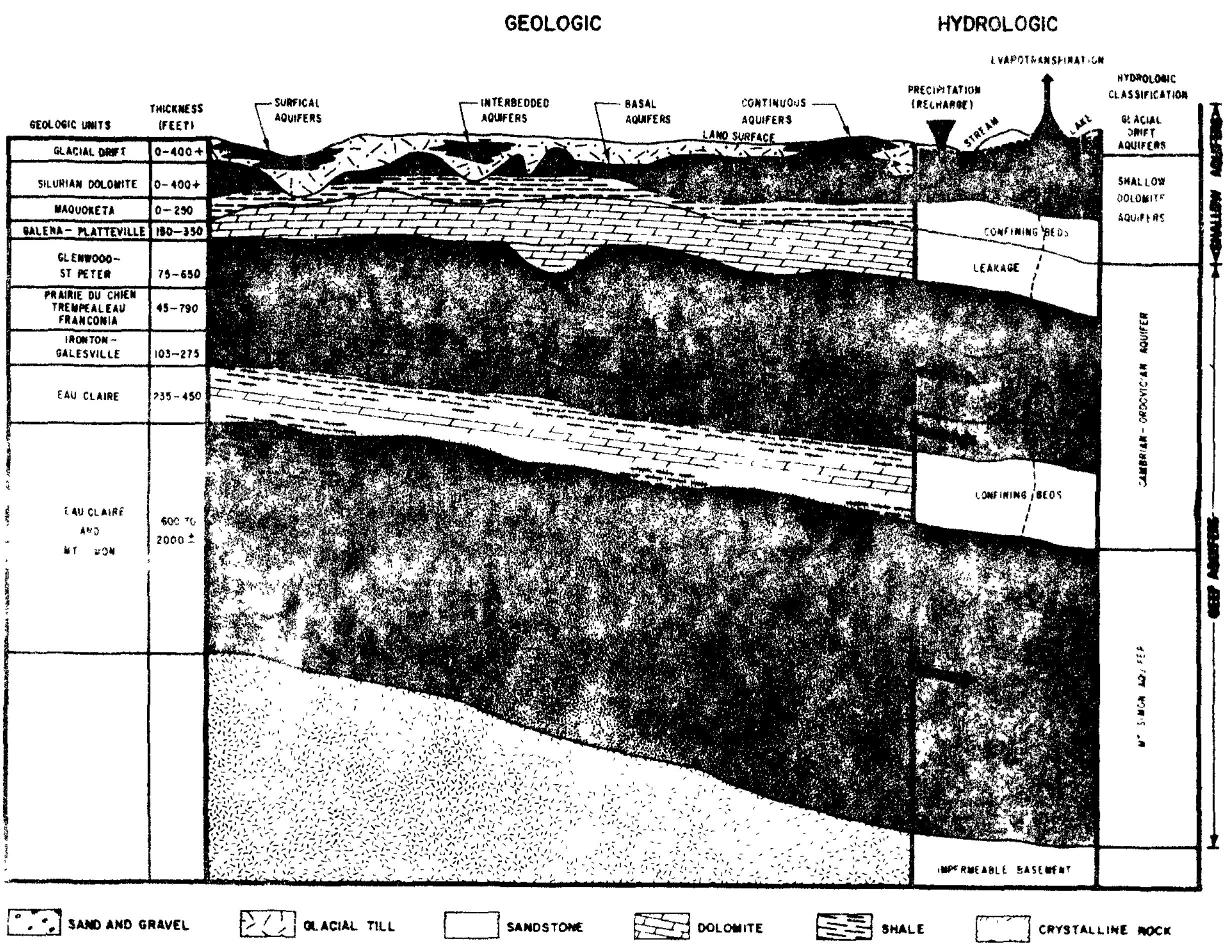
The City of Chicago is the largest user of Lake Michigan water, withdrawing amounts to meet its own needs as well as those of 72 suburban communities in Cook County which purchase water under contract. Additional water is withdrawn by fourteen other public water supply systems located along the Lake Michigan shoreline in Cook and Lake Counties; service by those systems is usually limited to one or two communities. Given the expected future development of the region, coupled with decreased groundwater availability in certain areas, the Lake will be more heavily relied upon for public water supply in the future.

2.03 SHALLOW AQUIFERS The shallow aquifer system in northeastern Illinois is comprised of unconsolidated sand and gravel deposits of the glacial drift and dolomite formations, mainly of Silurian age. Although these aquifers are hydraulically interconnected, their characteristics are sufficiently different to warrant separate discussion.

a. **Sand and Gravel Aquifers** The sand and gravel aquifers randomly underlie approximately 50 percent of the region at depths ranging from near the land surface in certain areas to more than 400 feet in others. (Figure 2-1 is a cross-sectional illustration of

(1): Ref. 1, pg. 3

Figure 2-1 **The Groundwater Aquifers of Northeastern Illinois**



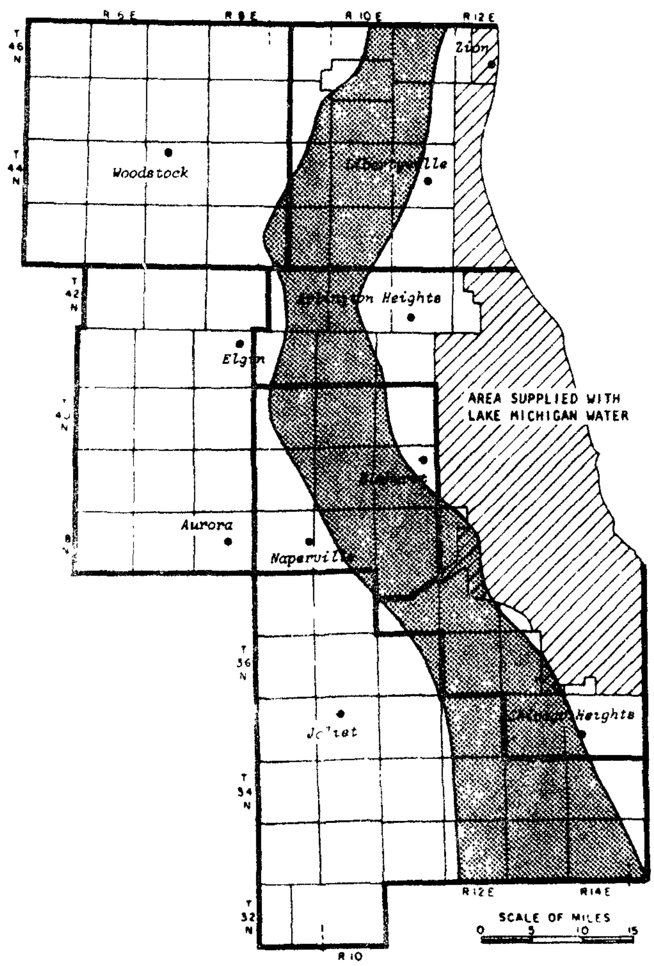
the entire regional aquifer system.) Extensive surficial sand and gravel deposits are found in parts of DuPage, Kane, Lake, McHenry and Will counties, while deeply buried deposits are found widely scattered throughout Kane and McHenry counties, western Lake County, northeastern Cook and DuPage counties and central Will County. Generally, the greatest chance for successful well penetration of a productive water yielding sand and gravel formation is within subsurface valleys cut into the bedrock by preglacial and glacial geological processes.

Because of their irregular occurrence, the sand and gravel aquifers are more difficult to locate than the deeper, more extensive sandstone aquifers. They are also more difficult to develop for large water supply systems since they are more directly affected by the vagaries of rainfall and drought. On the other hand, the glacial drift aquifers are generally more rapidly recharged, are more permeable than the deep aquifers, and involve lower drilling costs. Locally they provide good sources of supply to municipalities and private individual users, with some wells yielding in excess of 1,000 gallons per minute (gpm). In 1970, approximately 31.4 MGD were pumped from the sand and gravel aquifers in the six-county region. This amounted to approximately 12 percent of the estimated total groundwater pumpage in that year, which was 261.2 MGD.(2)

The hardness content of raw water is extremely variable but usually ranges between 100 parts per million (ppm) and 450 ppm. The iron content which can affect the taste, appearance and use of water averages about 2 ppm and is higher than that of the deep aquifers and Lake Michigan. Water temperatures average about 52 degrees, which is considered to be cool and refreshing.

b. Dolomite Aquifer Underlying much of the region at depths varying from ground surface to 450 feet deep is the shallow dolomite aquifer. In this aquifer groundwater is found in joints and fractures, and it moves through an interconnected network of these openings. Since these water-bearing cavities are unevenly distributed both horizontally and vertically, the yields of wells drilled into the dolomite vary greatly from place to place. Successful development for water supply depends upon a well intersecting a large, water-filled fracture which is capable of sustaining heavy pumpage over time. Some wells drilled into dolomite yield in excess of 1,000 gpm, while others result in very low yields. Figure 2-2 shows the general area of highest yields from this formation.

Figure 2-2 **Area of High Yield from the Shallow Dolomite Aquifer**



Adapted from map prepared by Alvord Burdick and Howson Engineers

The dolomite aquifer is an extensively used source of water supply for many municipalities, particularly in DuPage County and southern and northwestern Cook County. In 1970, total pumpage from the dolomite was estimated at 90.7 MGD, which was approximately 35 percent of the region's groundwater withdrawal.(3) In several areas, the aquifer is being pumped in excess of recharge, and there have been significant declines in water levels and well yields.

c. Estimates of Potential Yield Potential yield is defined as the maximum amount of water that can be developed from a reasonable number of wells and well fields without creating critical water levels or exceeding the rate of groundwater recharge. The Illinois State Water Survey has estimated the potential yield of the shallow aquifers (sand and gravel and dolomite combined) at 507 MGD, assuming they are fully developed in the six-county area.(4) According to the total shallow aquifer pumpage figures noted above, only 122.1 MGD were withdrawn in 1970. Thus, on a regionwide basis, the shallow aquifers are currently producing only about 25 percent of their potential yield, and there is greater opportunity for increased development of them for future water supply.

2.04 CAMBRIAN-ORDOVICIAN AQUIFER The Cambrian-Ordovician (or deep sandstone aquifer) is regarded as the best bedrock aquifer in Illinois because of its consistently high yield. It extends continuously throughout the region and is uniformly productive. This aquifer is actually a vertical series of water-bearing rock formations, of which the Glenwood-St. Peter and Ironton-Galesville sandstones are the principal producers. The latter is considered to be the most productive and supplies over 50 percent of the aquifer's total yield. Because this aquifer has a regional southeasterly dip of about 10 feet per mile, the top of the Ironton-Galesville sandstone lies about 800 feet below the land surface in the northwest corner of the region and increases to a depth of about 1,800 feet in the southeastern part. The saturated thickness of this aquifer varies from approximately 100 feet to about 275 feet, while the average collective thickness of the geologic formations comprising the aquifer is about 1,000 feet. It is significant to note that while some recharge of the deep sandstone

(2): Ref. 2

(3): Ref. 2

(4): Ref. 3

occurs in western Kane and McHenry counties, most takes place in areas outside of the metropolitan region, including Kendall, Boone and DeKalb counties of Illinois, and in certain areas of southeastern Wisconsin. It is important that future urbanization and land use in those areas not have an adverse impact on recharge.

The Cambrian-Ordovician aquifer is the most heavily pumped aquifer in the region; it furnished approximately 53 percent of the total groundwater used in 1970. Since 1958, withdrawals from this aquifer have exceeded the practical sustained yield, which is defined as the maximum amount of water which can be continuously withdrawn from existing pumping centers without eventually dewatering the most productive unit (i.e., the Ironton-Galesville standstone). The practical sustained yield of the Cambrian-Ordovician aquifer has been estimated at only 46 MGD. Pumpage data for 1970 indicates that actual withdrawals approximated 139 MGD, or about three times the estimated sustained yield.(5) This withdrawal of water at rates in excess of natural recharge (termed "mining") has been reflected by a progressive decline in water levels, increased pumping lifts and increases in pumping costs. During the period 1966-1971, annual water level declines in wells in the Cambrian-Ordovician aquifer averaged nine feet.

Table 2-1 lists the increases in pumpage from the deep sandstone aquifer during the period 1966-1971.

TABLE 2-1: PUMPAGE FROM DEEP WELLS IN NORTHEASTERN ILLINOIS, 1966-1971 (IN MGD)(6)

	Public Supplies		Industrial Supplies		Total	
County	1966	1971	1966	1971	1966	1971
Cook	31	42	24	17	55	59
DuPage	11	16	1	1	12	17
Kane	23	26	3	2	26	28
Lake	2	5	1	2	3	7
McHenry	2	2	1	1	3	3
Will	12	14	16	14	28	28
Total Region	81	105	46	37	127	142

While industrial pumpage declined over the five-year period, total pumpage actually increased 15 MGD as a result of greater withdrawals for public supply. Particularly noticeable are the increases in public pumpage in Cook and DuPage counties.

Despite the problem of overpumpage, the Cambrian-Ordovician aquifer will continue to be an important source of supply. Water in this system is naturally free of bacterial pollution. The hardness content is from 200 to 250 ppm in the northwest part of the region, and increases toward the east as the aquifer increases in depth. The iron content of the water is usually less than 0.4 ppm. Temperatures range from 54 to about 62 degrees and increase with depth. The Cambrian-Ordovician aquifer is generally well suited for large, municipal water systems; yields in excess of 1,000 gpm have been recorded. Mining of this aquifer cannot be continued indefinitely. Eventual provision must be made for transfer to an alternative or supplemental source in areas where water levels and well yields are declining.

2.05 MT. SIMON AQUIFER The Mt. Simon aquifer underlies the Cambrian-Ordovician system and is the deepest in the region. The top of this aquifer ranges from 1,400 to 1,600 feet below the ground surface in the northwest, and from 2,200 to 2,400 feet in the southeast. Its average thickness is approximately 1,600 feet, with the materials consisting primarily of fine to coarse grained sandstone. The cleaner parts of the sandstone yield moderate quantities of water, although the aquifer's potential is limited by a number of factors. The most significant limitation is brackish water beginning at depths below 1,300 feet mean sea level, necessitating costly treatment prior to use. The aquifer also is not consistently permeable. Furthermore, deep and expensive wells are involved.

The practical sustained yield of the Mt. Simon aquifer has been estimated at 14 MGD, although development of this source has been virtually nonexistent to date. In 1973, the Illinois State Water Survey completed a feasibility study of developing and desalting water from the Mt. Simon aquifer. Reverse osmosis and freezing processes were considered feasible for 1 MGD capa[illegible]atment plants, while distillation was considered feasible for [illegible] plants. Costs (including wells, transmission lines, desalting facilities and brine disposal) ranged from $1.33/1,000 gallons for [illegible] 1 M[illegible] reverse osmosis plant to $1.85/1,000 gallons for a 5 M[illegible] tion plant.

2.06 SURFACE WATER RESOURCES Unlike many other large metropolitan areas, no inland lakes, rivers or streams are presently used for public water supply in northeastern Illinois. There is, however, substantial industrial use of water from the Sanitary and Ship Canal, the Calumet River, the Des Plaines River and, to a lesser extent, the Fox River.

a. Limitations There are several factors which have mitigated against the use of surface watercourses. Certainly one reason (at least until recently) has been the readily available supply of groundwater which could be developed at low cost. But the major deterrent has been the general poor quality of the region's surface waters, a problem which necessitates thorough, expensive treatment. While water treatment technology has advanced to the point where virtually any water can be made potable, the cost of such treatment may be excessive, especially when compared with the cost of developing alternative sources. Waterways such as the Des Plaines River have been discounted as viable sources of municipal water supply because of their lack of dependable flow, high concentrations of bacterial and viral organisms, high solids and heavy metals content, and undesirable tastes and odors.

Nevertheless, the suitability of these waters should be periodically reevaluated in light of changing needs and conditions. As improved methods of wastewater treatment are employed and as nonpoint sources of pollution are reduced, surface waterways may become economically feasible and attractive water sources. In the interim, greater attention could be given to increased use of these waters for non-domestic purposes whenever possible in order to alleviate competitive pressures on water resources which are suitable for public supply.

b. Kankakee River It should be noted that the Kankakee River is an exception to the foregoing discussion and does offer potential for development as municipal supply. The river's raw water quality is reasonably good, and its large flow volume would eliminate the need to construct expensive storage reservoirs. In addition, it is proximately located to the Joliet area where there is concern for the long-term availability of groundwater.

c. Fox River At the present time it is not advisable to use the Fox River for domestic purposes since a high percentage of its flow consists of wastewater treatment plant effluent which presents a risk of viral or chemical contamination.(7) However, the Fox River may offer some potential for future use as a public water supply. Indeed, state water quality standards have designated the river for "domestic and food processing water supply," and pollu-

(5): Ref. 2

(6): Ref. 4, pg. 8

(7): Ref. 5

tion abatement efforts necessary to achieve that standard are underway. After the desired level of water quality has been attained, the river might be used for this purpose. One possible approach would be to reduce deep well pumpage in the Fox Valley area to the rate which can be sustained without mining. Demands which could not be satisfied by groundwater under this condition could be compensated for through withdrawals from the river. During seasonal low streamflows, well pumpage could be increased beyond sustained yield on a short-term basis until normal flows are resumed.

d. *Other Streams* Finally it is significant to note that while there are a large number of tributary streams in northeastern Illinois, few present public water supply opportunities. In addition to quality problems, frequent periods of low flow would necessitate the construction of storage reservoirs, a condition for which the topography of this region is poorly suited. There are also problems of leakage in these reservoirs through permeable surficial materials or fractured bedrock, and excessive sedimentation. Sites which have been identified as being potentially suitable for water supply reservoirs are discussed in Appendix D.

SECTION B

Water Supply By County

2.07 GENERAL This section contains summary descriptions of water supply conditions in each of the six counties in the region. Particular emphasis has been placed on groundwater, since areas now supplied by groundwater are expected to face the most serious problems in both the immediate and distant future. The groundwater data used in this section were furnished by the Illinois State Water Survey; the information is thought to be the most accurate presently available. Total pumpage figures are cited, including amounts used for the following purposes: public (which includes municipal, subdivision and institutional); industrial, domestic; irrigation; and livestock. The amounts used for the last two purposes are difficult to quantify and are not thought to be substantial. Estimates of the potential yields of the shallow aquifers in each county were also made by the Water Survey.

2.08 COOK COUNTY Lake Michigan is the predominant source of supply for Cook County. The City of Chicago withdraws water from the Lake to supply its own needs and in addition furnishes water on a contractual basis to a number of suburban communities. In 1970, the City provided an average of 1,035 MGD to supply 4.52 million people in its service area. There are six other independent municipal systems located along the north shore which withdraw Lake water. Lansing, in the southeastern part of Cook County, obtains Lake water via the Hammond, Indiana, system.

A significant portion of Cook County is supplied with groundwater. In 1970, pumpage amounted to 99.6 MGD, which was approximately 38 percent of the six-county total groundwater pumpage of 261.2 MGD in that year. Of the 99.6 MGD, 59.6 MGD (60 percent) were taken from the deep sandstone aquifer, 36.4 MGD (36 percent) from the dolomite, and 3.7 MGD (4 percent) from the shallow sand and gravel.

Deep sandstone pumpage in Cook County is more than twice that in any other northeastern Illinois county. Water well levels have declined in response to heavy drawdowns, particularly in the northwest and southern portions. There is also significant pumpage from the shallow aquifers, especially in the southern sector. Chicago Heights, and to a lesser extent LaGrange, have been identified as areas where pumpage has exceeded recharge. Countywide, the estimated shallow aquifer potential yield is 98 MGD.

2.09 DU PAGE COUNTY DuPage County is supplied exclusively with groundwater, and pumpage in 1970 averaged 51.7 MGD. The dolomite accounted for 33.9 MGD of the total withdrawal (66 percent), while the deep sandstone yielded 15.5 MGD (30 percent) and the sand and gravel aquifers 2.3 MGD (4 percent).

There is considerable concern for the long-term adequacy of groundwater supplies in DuPage County. The potential yield of the sand and gravel and dolomite aquifers is estimated at 42 MGD and by 1972, pumpage from these aquifers had increased to 39.7 MGD. In some areas (most notably in the vicinities of Hinsdale, Clarendon Hills, Addison, Downers Grove, Wheaton and Glen Ellyn) the dolomite is already being pumped in excess of recharge, and there has been permanent lowering of the water table and reductions in well yields. In an effort to compensate for these declines, increased numbers of deep sandstone wells have been drilled. Extensive mining is being practiced, and water levels in the deep wells have been declining steadily for several years.

2.10 KANE COUNTY Kane County is supplied primarily with groundwater, although there is some minor industrial use of the Fox River. The western two-thirds of the county is largely rural, and no particular water supply problems are being experienced. However, in the more urbanized Fox River Valley area, the Cambrian-Ordovician aquifer is heavily used, and steady water level declines have been observed, particularly in Aurora and Elgin.

The importance of the deep sandstone aquifer is illustrated by the fact that it provided 27.9 MGD (or 74 percent) of the 37.5 MGD total pumpage in 1970. The sand and gravel and shallow dolomite aquifers produced 6.2 MGD (17 percent) and 3.4 MGD (9 percent) respectively. Their potential yield is estimated as 31 MGD, which is the lowest of the six counties.

2.11 LAKE COUNTY Lake Michigan is the primary source of supply in eastern Lake County, while groundwater is used in the central and western portions. In terms of total groundwater pumpage, development of the three aquifer systems has been approximately equal. According to 1970 pumpage figures, withdrawals amounted to approximately 19 MGD. The sand and gravel aquifers produced 6.9 MGD (36 percent) of the total, followed by 6.1 MGD (32 percent) from the dolomite and 6.0 MGD (32 percent) from the deep sandstone.

Water level declines in the deep wells are being experienced in certain areas (primarily Libertyville and Mundelein), although this situation is not as severe as that in Cook and DuPage counties. There appear to be considerable opportunities for greater development of the shallow aquifers, where potential yield is estimated at 51 MGD.

2.12 MC HENRY COUNTY Of the six counties, McHenry is in the most favorable position with respect to water supply. The sand and gravel aquifers are by far the predominant source and their use is increasing. They supplied 9.4 MGD (or 63 percent) of the county's 15 MGD total pumpage in 1970. By way of contrast, the deep sandstone produced 3.0 MGD (20 percent) and the dolomite produced 2.6 MGD (17 percent).

It is significant that the combined potential yield of the shallow aquifers is estimated at 96 MGD. Thus, while the shallow aquifers provided 12 MGD (or 80 percent) of McHenry County's total 1970 groundwater demand, this still amounted to only about 13 percent of the total quantity potentially available to the area from the shallow system.

raising Lakes Michigan and Huron by 4.4 inches, excluding the effects of the Illinois diversion. The effect of the present Illinois diversion (exclusive of the diversion into Lake Superior) is to lower Lakes Michigan and Huron 2.7 inches The net effect of these in-and-out diversions is to raise the levels of Lake Michigan and Lake Huron by 1.7 inches. To put this in perspective, artificial diversion *into* the Great Lakes presently exceeds diversion *out* of the Lakes by approximately 1,800 cfs. It therefore would appear that diversion by Illinois could be increased without having any critical effect on the Great Lakes Basin as a whole. Indeed, such an increase would allow a better inflow-outflow balance to be achieved.

SECTION C

Groundwater Mining

8.12 GENERAL There are two basic approaches to groundwater development. The first regards aquifers only as systems through which water moves, and favors limiting well withdrawals to the practical sustained yield. The second approach, mining, favors continued withdrawal of water from the aquifers at a rate which exceeds that of recharge At the present time, approximately 96 MGD of the 112 MGD pumped from the deep sandstone aquifer in this region are mined.

8.13 ADVANTAGES AND DISADVANTAGES Mining is a debatable issue. The most common argument against the practice generally has been that since it removes water held in storage, it deprives future generations of the right to obtain adequate water at low cost The extension of this reasoning is that present pumpage ought to be reduced to sustained yield, with any deficiencies to be made up through the development of alternative supplies, including remote surface sources In this way, water held in aquifer storage would be kept in permanent trust for future use. The counter argument in favor of mining is that the water in storage is of no value unless it is used In addition, mining allows large capital investments in surface water supply projects to be deferred to a later date. In the interim, changing technologies and alterations in water use patterns conceivably could reduce the need for importing large quantities of water.

One of the central objectives of water management is to provide adequate service with the maximum net benefit to all. Clearly, if the same benefits can be derived from any of several alternatives, the least cost alternative will result in the maximum net benefit. Since the cost of mining water is usually less than the cost of obtaining water from an outside source, it follows that mining may need to be conducted until it is no longer economically feasible, at which time the next "lower cost" source would be developed.

There are a number of other reasons why mining of the deep sandstone aquifer might be continued on a managed basis in northeastern Illinois First, if mining were not practiced and withdrawals were limited to the rate of recharge, a number of townships in the region would become deficient in groundwater by 1980 Given existing legal limitations on diversion of Lake water, these deficiencies could not easily be satisfied by importation from that source. Furthermore, considering the existing investment in wells and pumping facilities, coupled with the large amount of water held in aquifer storage, it may be expedient to continue or accelerate mining, at least on a short-term basis. It should also be noted that the dewatering of the aquifer as a result of mining probably would not cause serious damage to the aquifer's water storage or transmitting properties. Indeed, if after a period of mining, pumpage were reduced to sustained yield, water levels would rise and the capacity of the aquifer to transmit water would eventually return to its original state.

Water Conservation, Recharge, and Recycling

8.14 GENERAL One means of helping to avert water shortages is to institute water conservation and/or reuse and recycling techniques. Conservation measures employ technical, economic, educational or legal tools to control water usage in such a way as to balance it with supply Recycling seeks to maximize the use potential of any given quantity of water. The primary objective of both of these approaches is to manage existing sources more efficiently and effectively as an alternative to developing new sources.

8.15 WATER CONSERVATION TECHNIQUES A detailed discussion of water conservation (particularly domestic conservation techniques) is contained in Appendix E Th which follows is primarily concerned with water metering and leakage control with brief attention given to reduction of in-house water waste

a. Metering Metering water consumption is one method of encouraging thrift and normalizing water demand in a community. Metering allows consumers to be charged according to the amount of water they use, thus providing an economic incentive to minimize waste. For example, greater use of meters has been cited as a contributing factor to the reduction in per capita consumption in the City of Chicago, where average water use decreased from 302 gpcpd in 1930 to 249 gpcpd in 1972.

Metering is regarded as one of the most fundamental precepts of modern water management. Yet, a number of public water systems in the region do not meter consumption and prefer to charge a flat rate for water provided regardless of the amount used With a flat rate system in operation, there is virtually no economic incentive for consumers to practice water conservation

It must be recognized that the cost of purchasing, installing maintaining and reading water meters is substantial Thus, it m not be economical to meter all water users that are presently unmetered, particularly in light of the relatively low rates charged for water in most communities. However, as water becomes a more valuable resource in the future, greater metering (at least of new and large users) will probably be practiced.

b. Control of Leaks Leakage from water distribution systems can create a substantial demand on water supplies without providing any corresponding benefits Excessive leakage reduces the amount of water available for domestic purposes and increases overall costs. A number of factors influence leaks, including age of the system; materials used in construction; physical and chemical properties of the soil; properties of the water, pressures involved; and the degree of proper maintenance.

While no system is absolutely tight and some leakage will inevitably occur, leaks should be reduced to the greatest practical degree. In the construction of new distribution facilities or in the replacement or addition to older facilities, leakage control can be achieved through proper sealing of joints and testing for tightness Control in existing systems can be achieved through an ongoing detection and correction program. However, the savings derived from such a program must be balanced against the costs of its operation. Total elimination is seldom justifiable economically, but it can proceed to the point where the cost of salvageable water lost equals the cost of a repair program. Any additional rehabilitation beyond this point would not be economical since the cost of repair would exceed the incremental benefits derived from the water savings.

The appropriate magnitude of a leakage detection and repair program is thus dependent upon a number of factors, the most important of which are the rate of loss of salvageable water within the system, the cost of supplying water; and the cost of system maintenance and repair. Individual communities contemplating a leakage control program should evaluate their particular systems in light of these conditions to determine the extent of corrective action warranted. Those having serious leakage problems may benefit considerably from increased water savings, especially if water costs are high or supply is inadequate. Conversely, communities that have relatively minor leakage problems, low water costs and abundant supplies probably need not undertake extensive control programs.

c. Water Conservation in the Home Several steps can be taken to reduce water consumption and/or waste in and around the home. Maintenance and repair of leaky plumbing fixtures can save large quantities of water over time which otherwise would be lost. Use of water conserving devices such as shallow trap toilets, washing machine "suds savers" and restricted flow showerheads can also reduce in-house water consumption. Substantial reductions can also be effected by taking care that lawns are not overwatered and that too much water is not used for such activities as washing automobiles. Conscious efforts to eliminate waste not only conserve water but also result in economic savings in the form of reduced water bills.

8.16 ARTIFICIAL RECHARGE Intensive development of groundwater has created considerable interest in the possibility of artificially recharging the aquifers. Replenishment of water in areas of concentrated pumpage, if feasible, would reduce the rate of water level decline and improve the yield capacity of wells. Consequently, the lives of existing wells could be prolonged and the aquifer could continue to provide a dependable water supply.

a. Sources of Recharge Water The most readily available source of water for artificial recharge is the seasonal high flow in surface streams. The diversion of high flows from stream channels for artificial recharge would also make available additional storage space in these channels for the temporary storage of flood peaks. Sophisticated stormwater drainage systems provide efficient means for the collection and temporary detention in basins of water that also can be used for artificial recharge of the shallow aquifers. If the highly polluted initial flush from urban areas is bypassed, the remaining stormwater, if treated, may be suitable for artificial recharge. However, the feasibility of this technique needs to be more thoroughly investigated. Other possible sources include cooling water, certain industrial wastewaters, and conceivably, treated domestic wastewater.

b. Methods The three principal methods of direct artificial recharge are water spreading, seepage pits and injection wells. Induced infiltration from streams caused by pumpage from nearby wells is an indirect method of artificial recharge. Whatever the method, artificial recharge requires agencies and facilities to: obtain, treat (if necessary) and transport the water to the recharge area, infiltrate or inject the water, and provide for the disposal of any excess water. The development of the area affects the capital costs of the project. High land costs in the urbanized parts of the region favor the use of the pit and injection well methods which require less land. Spreading basin methods require more land and would more likely be used in rural areas. Economics will strongly influence the degree to which artificial recharge operations are initiated in the future.

c. Potential Recharge Areas The Illinois State Water Survey has identified ten areas in northeastern Illinois which would probably be suitable for the pit method of artificial recharge. These areas were selected because there was a well-defined cone of depression in the water level surface of the aquifer under consideration; there was a surficial sand and gravel deposit in the area, and there was a perennial stream in the immediate vicinity to serve as a source of water with which to recharge. The ten sites so identified are listed in Table 8-2, generally in order of decreasing need.

8.17 ENHANCEMENT OF NATURAL RECHARGE Management measures can also be taken to protect or enhance natural recharge. For example, in rural areas, natural recharge rates can be increased through the use of basic soil conservation techniques such as contour plowing and terracing. In urban areas (as well as in undeveloped rural areas) natural recharge can be sustained by restricting the construction of buildings, pavement and other impermeable surfaces in prime recharge areas. If these areas are reserved as open space and protected from intensive urbanization, they can continue to function in their recharge capacity. On the other hand, if they are substantially developed, recharge will not be able to keep pace with withdrawal and groundwater shortages may develop. Figure 8-1 depicts the prime natural recharge areas in the region.

TABLE 8-2: POTENTIAL ARTIFICIAL GROUNDWATER RECHARGE AREAS IN NORTHEASTERN ILLINOIS(14)

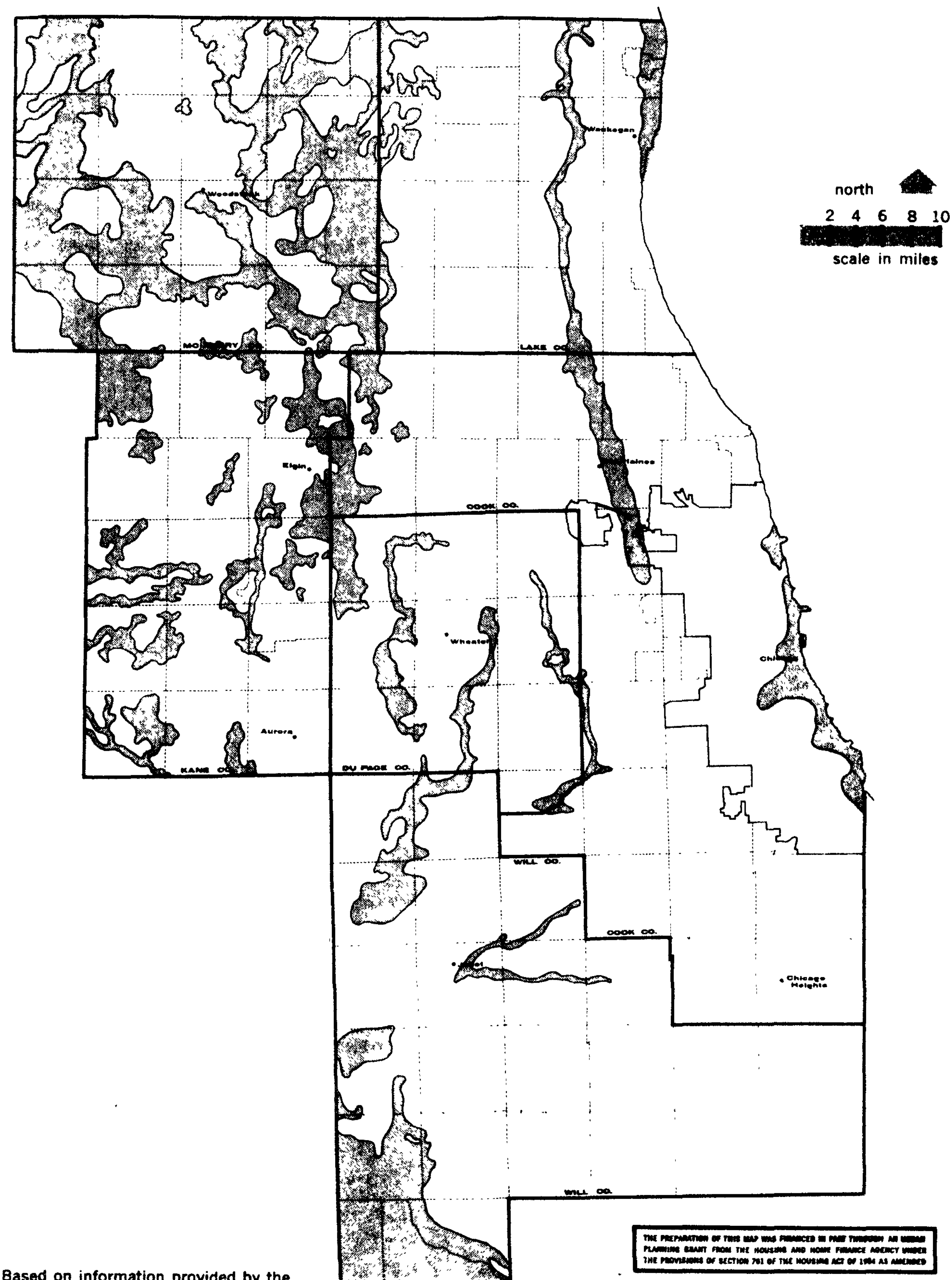
County	Potential Recharge Area	Recharge Source	Aquifer
Cook-Will	Park Forest-Chicago Heights	Thorn Creek	Dolomite
Will	Joliet (Hadley Valley)	Spring and Hickory Creeks	Sand and Gravel
Lake	Libertyville-Mundelein	Des Plaines River	Sand and Gravel Dolomite
DuPage-Cook	Western Springs-Hinsdale	Salt Creek	Sand and Gravel Dolomite
DuPage	Glen Ellyn-Lombard	East Branch of DuPage River	Sand and Gravel Dolomite
Cook	Wheeling	Des Plaines River	Sand and Gravel Dolomite
Kane	East Dundee-Carpentersville	Fox River	Sand and Gravel
Kane	Elgin-South Elgin	Fox River	Sand and Gravel
McHenry	Marengo	Kishwaukee River	Sand and Gravel
DuPage	Lisle-Downers Grove	East Branch of DuPage River	Sand and Gravel Dolomite

8.18 WASTEWATER RECYCLING AND REUSE Land disposal of wastewater has recently received a great deal of publicity and attention. In this approach, treated effluent is spread on the ground surface, usually with spray irrigation equipment. Nutrients are removed through vegetative uptake, and the effluent is further treated by filtration through the soil. Originally, it was intended that the water would continue to percolate through the soil and eventually become a part of the groundwater supply. However, pending more extensive investigations, concern for the protection of public health prohibits the use of such treated wastewater as a source of public water supply. Indeed, spray irrigation projects undertaken to date have employed underdrain systems to prevent pollution of the aquifers.

It also is not likely that large scale, direct recycling of wastewater effluent for use as public water supply will become a

(14). Ref. 8

Figure 8-1 **Prime Natural Recharge Areas in Northeastern Illinois**



Based on information provided by the Illinois State Water Survey realistic alternative in northeastern Illinois, at least in the immediate future. It is true that sophisticated methods of waste treatment are presently available which allow near total reduction in the biological and chemical contaminants of wastewaters. When thus purified, the effluent is suitable for industrial or agricultural purposes. However, the cost of such treatment, when coupled with health concerns and probable aesthetic objections, does not presently favor the use of recycled wastewater for municipal supply.

During 1973, this Commission reviewed two separate but related applications for federal funds which involved testing the feasibility of recycling wastewater effluent for use as potable water supply. The applicants were the Village of Bensenville and the Hinsdale Sanitary District, both of which are located in eastern DuPage County where there is considerable concern for the adequacy of local groundwater supplies. The basic concept of both these proposed research and development projects involves the incineration of municipal solid waste to produce heat, which can then be used to distill treated wastewater plant effluent. Depending upon the outcome of test results, the distillate could be used to directly augment present water supplies or to increase local groundwater recharge. Both projects are presently being reviewed by the USEPA. Their futures are uncertain at this time due to the paucity of federal funds for projects of this nature.

SECTION E

Organization and Administration

8.19 FRAGMENTATION Perhaps the most conspicuous shortcoming of the present institutional framework for water supply is the extensive fragmentation of authority and responsibility. Several federal, multi-state, state, regional, and county agencies conduct specialized programs which have significance in water supply planning and management. At the local operational level, there are approximately 260 municipalities and numerous special purpose districts which are empowered to furnish water and engage in related activities. Then, too, there are a number of private utility companies authorized to provide water, principally in subdivisions and other unincorporated areas.

At the present time, this Commission is the only governmental unit conducting a comprehensive water resources management planning program in the six-county northeastern Illinois region. On the operational level, the trend continues toward the creation of more separate and independent systems which deal with problems on a piecemeal basis. Waterworks have been constructed and expanded without benefit of areawide planning, coordination or controls. Slight attention has been paid toward developing a water supply system for the region as a whole, with the view of providing for needs beyond the immediate future. Instances of coordinated, interlocal efforts have been few. Indeed, there are cases in which there has been keen competition between communities for available water, a situation which has at times interfered with the optimum development of the resource.

There are, of course, examples of successful intergovernmental cooperation. The arrangement by which the City of Chicago provides water to suburban Cook County communities is the most notable. Some of the lakeshore communities north of Chicago provide water to neighboring inland municipalities on a similar though more limited basis. There are also four public water commissions or districts which were organized for the purpose of obtaining and furnishing water to customer municipalities on a sub-regional scale.

The Great Lakes Basin Commission has noted that although it may be difficult, more emphasis should be put forth in developing plans for areawide utilities and cooperate efforts.(15) Problems such as well interference could be solved by preventing the proliferation of small water systems while favoring larger utilities which cross corporate boundaries and which develop the best available source of water rather than relying heavily on wells in the immediate area. The GLBC recommends that the preparation of such plans, before population pressures and increased water use necessitate independent crash programs, should begin immediately and be worked out with local, county and regional planning commissions. Implementing plans for areawide utilities may require the creation of additional laws and regulations.

The most pressing future water supply need will be that of providing adequate substitute sources for those areas of the region where groundwater deficiencies are expected to occur. Given the fact that the areas of projected shortage are generally located some distance from Lake Michigan, it is not feasible for individual municipalities to construct their own independent systems. Some type of multi-community approach may have to be taken in order to achieve economies of scale and to minimize conflicts and inefficiencies.

8.20 ORGANIZATIONAL ALTERNATIVES There are several alternative organizational structures which might be established for this purpose, varying both in scope of authority and area of jurisdiction. A number of these possible alternatives are highlighted below to illustrate the range of management opportunities available.

a. Maintain Existing Arrangements This is a continuance of the status quo in which no major changes in agency structures or prerogatives would be effected. Water supply development and use decisions would continue to be made at local levels, generally without regard for broader area needs and problems.

Water supply has traditionally been viewed as a local responsibility, and attempts to drastically alter this approach may not withstand the test of implementation. Therefore, expansion and coordination of the water supply programs of existing local units may be the most politically feasible and realistic method for dealing with water supply problems on a regional scale. The potential for duplication of effort, waste of funds, and competition and conflict would remain.

b. Metropolitan Water Authority At the opposite end of the institutional spectrum would be the creation of a six-county metropolitan water authority. If authorized, this agency would assume primary responsibility for furnishing water on a "wholesale" basis throughout the region, or for significant portions thereof where economies of scale might favor such an arrangement. Source development, treatment, and primary transmission would fall within its purview. Individual municipalities would retain responsibility for constructing and operating local distribution and storage systems.

It would also be possible to expand the role of the water authority to include other important aspects of water resources management. This has been done in the Detroit metropolitan area where a single agency was created to deal with the water supply, wastewater, and stormwater drainage problems of Detroit and 88 neighboring municipalities. With respect to water supply alone, significant cost savings have been realized as a result of the metropolitan utility approach.

Such an agency would allow for the systematic expansion and operation of all public water facilities in the region. It would of course be necessary to base such functional program on a comprehensive plan for the region to ensure orderly and efficient growth and development. Other issues requiring careful consid-

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(15): Ref. 9, pg. 278