View metadata, citation and similar papers at core.ac.uk
brought to you by CORE
provided by Illinois Digital Environment for Access to Learning and Scholarship Repository

Contract Report 598

City of Charleston Public Water Supply: Analysis of Drought Yields from the Charleston Side-Channel Reservoir

by
Sally McConkey and Krishan P. Singh
Office of Surface Water Resources: Systems, Information & GIS

**Prepared for the
City of Charleston, Illinois**

April 1996

Illinois State Water Survey
Hydrology Division
Champaign, Illinois

A Division of the Illinois Department of Natural Resources

CITY OF CHARLESTON PUBLIC WATER SUPPLY: ANALYSIS OF DROUGHT YIELDS FROM THE CHARLESTON SIDE-CHANNEL RESERVOIR

by
Sally McConkey and Krishan P. Singh

Illinois State Water Survey
2204 Griffith Drive
Champaign, IL 61820-7495

Prepared for the City of Charleston

April 1996

CONTENTS

TABLES

Page

INTRODUCTION

The City of Charleston's source of public water supply is the Embarras River. Pumps located upstream of an in-channel dam withdraw river water which is then stored in a side-channel reservoir adjacent to the river. Stored water is subsequently withdrawn from the side-channel reservoir to the water treatment facility. The sustainable yield from the raw water supply may be defined as the maximum water withdrawal rate that can be expected to be delivered during a drought of specific return frequency, such as a 20- or 50-year return interval. Several factors define the sustainable net yield of water for the community, including: the Embarras River streamflow patterns; the volume of the side-channel reservoir; the intake elevation of the side-channel pumping facility; direct runoff to the side-channel reservoir; precipitation on and evaporation from the surface of the side-channel reservoir; and the pumping capacity of the river pumping facility. Long-term planning for Charleston's water needs requires an evaluation of the existing system and an assessment of feasible changes to the system to increase the sustainable yield.

Study Objective

The purpose of this study is twofold: first to carefully determine the sustainable yield from the current raw water supply and pumping system; and second to examine possible alternatives for increasing the sustainable yield from the current source. This assessment will provide the community with information to determine whether the current system is adequate for projected water needs and possible alternatives for increasing the sustainable yield from the raw water source. The study encompasses only those issues related to the raw water supply and does not include evaluation of any aspect of the water treatment process.

POPULATION PROJECTIONS AND WATER SUPPLY USE

The future demand for water is dependent upon several factors including: population, water use per capita (per person), and industrial/commercial demand. Population growth and per capita water use typically increase gradually, creating a gradual increase in water demand. Population may increase slowly, but if demand per person increases dramatically, so will the total demand for water. New industrial and commercial uses can create a rapid increase in water use over a short period of time. Another consideration is the tendency for demand to increase during a drought when the weather is hot and dry. The community's activity in attracting new development will have a significant impact on water use.

County population forecasts performed by the Illinois Bureau of the Budget were used to develop the population projections reported in *Adequacy of Illinois Surface Water Supply Systems to Meet Future Demands* (Sally McConkey Broeren and Krishan P. Singh, 1989, Illinois State Water Survey Contract Report 477). Per capita water use trends were then combined with the population projections to develop the water use forecast. Other population projections for the City of Charleston have been developed over the years using various approaches.

In the interest of examining a suitable range of possibilities, two scenarios of population, per capita water use, and industrial demand were used in this evaluation of the water supply source. These two scenarios demonstrate: 1) a moderate increase in water demand; and 2) the likely maximum increase in demand. The first scenario uses the population and water use projections published by Broeren and Singh (1989). The second scenario was developed using population projections from a regression analysis of census data from 1940 to 1990 (provided by Mark Donnelly, Charleston Water Treatment Plant). The second scenario also incorporates higher per capita water use, projected on the basis of an increase in per person water use observed in 1994 and 1995, and includes a possible increase of 0.75 million gallons per day (mgd) for industrial use.

The values are presented in Table 1. For ease of reference the two water demand scenarios are identified as D1 and D2.

During droughts water use is typically greater than during non-drought periods. Water demand may be as much as 30 percent higher than the annual average demand during a drought with a duration less than 12 months. Drought water demands listed in Table 1 show the potential demand that may occur during a drought if there are no restrictions on water use.

WATER SUPPLY SOURCE ASSESSMENT METHODOLOGY

The City of Charleston obtains its raw water for public use from the Embarras River. Intake pumps are located in the river upstream of Riverview Dam, a low-channel dam at river mile 118.8. The drainage area of the Embarras River at Riverview Dam is 786 square miles (sq. mi.). River water is pumped to the side-channel reservoir, Charleston Side-Channel Reservoir (CSCR).

The reliable yield of raw water for the City of Charleston from the present water supply source is a function of the streamflow characteristics of the Embarras River, the volume of the side-channel storage, minimum instream flow needs, and the pumping systems employed. Pumping systems and storage facilities may be altered, but it is unlikely that the long-term flow patterns of the Embarras River will experience significant modification in the foreseeable future.

The yield of the side-channel system was evaluated following the methodology reported in *Hydrologic Design of Side-Channel Reservoirs in Illinois*, Illinois State Water Survey Bulletin 66, H. Vernon Knapp, 1982 (referenced as Bulletin 66 in this report). The analyses for side-channel reservoir yield are described in detail in that report. Bulletin 66 contains a demand-storage-recurrence relationship developed from 63 years of discharge records for the Embarras River at Ste. Marie. The drainage area of the Embarras River at Ste. Marie is 1516 sq. mi. Various gages have been in place along the Embarras River over the years. The continuous

Table 1. Population and Demand Projections

	1950	*1960*	*1970*	*1980*	*1986*	*1990*	*1995*	*2000*	*2010*	*2020*	*2030*
Population, historical and projected											
Census data	9164	10505	16421	19355		20398					
ISWS forecast					19335	20277		21286	22887	24803	27035
Regression forecast (1)							20500	23758	26542	29327	32113
Water demand											
Raw water, mgd (2)	0.990	0.900	1.560	1.669	1.656	1.527	2.139				
Max. 10-day avg., mgd (2)				2.138	2.017	1.840	2.812				
Ratio				1.281	1.218	1.205	1.315				
Per capita water use, gpcd											
Actual water use	108	86	95	86	86	75	104				
ISWS forecast						92		96	100	104	108
Revised water use per capita (3)								110	115	120	125
Total water demand, mgd											
ISWS forecast						1.87		2.04	2.29	2.58	2.92
Higher forecast using revised population and water use per capita (1 and 3)								2.61	3.05	3.52	4.01
Higher forecast water use + 0.75 mgd increase in industrial use								3.36	3.80	4.27	4.76
Drought water demand projections (=1.3 times average daily), mgd											
D1 = projections using ISWS water use and population								2.66	2.98	3.35	3.80
D2 = projections using revised population and water use and including industrial use								4.37	4.94	5.55	6.19

Notes:
gpcd = gallons per capita per day
mgd = millions of gallons per day
ISWS = Illinois State Water Survey Contract Reports 442 and 477
1) Population projection made on the basis of Census data, 1940-1990
2) Reported by Charleston Water Treatment Plant staff
3) Higher forecast of water use on the basis of per capita demand exhibited in]1994 and 1995

recording gage at Ste. Marie has the longest record. A shorter-term record of flows is available for a gage near Diona, at river mile 103.7 (15.1 miles downstream of Riverview Dam). The drainage area of the Embarras River near Diona is 919 sq. mi. The historical discharge record of the Embarras River at Ste. Marie was compared to the discharge record of the Embarras River near Diona. Yield analyses for withdrawals at Riverview Dam were modified as indicated by comparing the gaging station discharge records, as explained in the following section. Relevant data regarding the Embarras River gaging stations at Ste. Marie and Diona are listed in Table 2.

Table 2. Summary of Embarras River Gaging Station Data

Embarras River station	*USGS number*	*River mile*	*Drainage area (sq. mi.)*	*Period of record*	*Years of record*
Near Diona	03344000	103.7	919	1939, 1945-1947, 1970-1982	17
At Ste. Marie	03345500	48.2	1516	1910 - 1992 *	80

* Period of record used for analysis, data available through current water year (1994)

HYDROLOGY

Analysis of Gaging Station Discharge Records

The method of evaluating the yield of a side-channel storage system described in Bulletin 66 uses nondimensional relations between the storage volume (expressed as days of demand) and raw water demand (expressed as a percentage of the mean streamflow). This nondimensional relationship is used to calculate the reliable yield at other locations along the river. The assumption in a straightforward application of the methodology is that the discharge is directly proportional to the drainage area,

$$Q_1 = (A_1/A_2)Q_2,$$

where A is the drainage area and Q is the discharge for sites 1 and 2, respectively, and

$$R=(Q_1/Q_2)$$

is the ratio between the discharges. The ratio of the drainage areas at the two gage locations is:

AD / ASM = 919/1516 or 0.6062.

A linear relationship between discharge at different locations on the river is generally correct when the discharge of interest is the mean annual discharge. The mean annual runoff for the Embarras River at Ste. Marie (for the period 1910 to 1992) is 1228 cubic feet per second (cfs). Applying the ratio of drainage areas, the mean annual discharge at Diona is 744 cfs, and at Riverview Dam it is 637 cfs. This relationship does not usually apply to low flows.

The linear function describing this relationship between daily flow at Diona and daily flow at Ste. Marie was evaluated by performing regression analysis using the concurrent years of data for the two stations: 1939, 1945-1947, and 1970-1982. The years when the lowest daily average discharges were recorded at Ste. Marie are shown in Table 3. The 63 years of data used in Bulletin 66 for Ste. Marie include all the low flow years noted in the table; 1915, 1954, and 1977 in particular hold most of the records for lowest average daily flow. The gaging station at Diona was in operation in 1977.

The linear correlation coefficient between the daily streamflow at Diona and Ste. Marie is 0.89. The intercept was forced to be zero for the analysis. The least-squares best-fit line has a slope, m_1= 0.57285. This relationship may be written as equation 1 below.

$$\mathbf{Q_D = 0.57285\, Q_{SM}}$$

or

$$\mathbf{R_{linear} = Q_D/Q_{SM} = 0.57285} \tag{1}$$

where Q_D is the discharge at Diona and Q_{SM} is the discharge at Ste. Marie in cfs. This ratio is lower than the ratio of drainage areas.

A similar regression analysis was performed using the base 10 logarithm ($\log_{10}$) of the daily streamflow data, which is expressed mathematically as: logio y = m_2 logio x+ b. The regression analysis was performed allowing a nonzero value for the intercept, b. Regression

Table 3. Year of Record with Lowest Recorded Average Daily Flow, Embarras River at Ste. Marie

Day of month	*Oct*	*Nov*	*Dec*	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*Jun*	*Jul*	*Aug*	*Sep*
1	1915	1915	1915	1915	1977	1954	1954	1915	1954	1954	1954	1954
2	1915	1915	1915	1915	1977	1954	1954	1915	1954	1954	1954	1954
3	1915	1915	1915	1915	1977	1954	1954	1931	1934	1954	1954	1954
4	1915	1915	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954
5	1915	1915	1915	1915	1977	1954	1954	1931	1954	1934	1954	1954
6	1915	1915	1915	1915	1977	1954	1954	1954	1934	1988	1954	1954
7	1915	1915	1915	1954	1977	1954	1954	1954	1934	1988	1954	1954
8	1915	1915	1915	1954	1977	1954	1954	1954	1934	1988	1954	1954
9	1915	1915	1915	1954	1977	1954	1931	1954	1954	1954	1954	1954
10	1955	1915	1915	1954	1977	1954	1931	1954	1954	1954	1954	1954
11	1955	1915	1915	1954	1954	1954	1954	1954	1954	1954	1954	1954
12	1989	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954	1954
13	1989	1915	1915	1977	1954	1954	1931	1954	1925	1954	1954	1954
14	1989	1915	1915	1977	1954	1954	1931	1954	1925	1954	1954	1954
15	1989	1915	1915	1977	1954	1954	1931	1954	1925	1954	1954	1954
16	1989	1915	1915	1977	1954	1954	1931	1954	1930	1954	1954	1954
17	1989	1915	1915	1977	1963	1954	1931	1954	1954	1954	1954	1954
18	1989	1915	1915	1977	1954	1954	1931	1954	1988	1954	1954	1954
19	1989	1915	1915	1977	1954	1954	1931	1954	1930	1954	1954	1954
20	1989	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954	1936
21	1965	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954	1936
22	1965	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954	1954
23	1954	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954	1954
24	1954	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954	1954
25	1954	1915	1915	1977	1954	1954	1931	1954	1954	1954	1954	1954
26	1915	1915	1915	1977	1954	1954	1954	1954	1954	1954	1936	1954
27	1915	1915	1915	1977	1954	1954	1954	1954	1954	1954	1988	1954
28	1915	1915	1915	1977	1954	1954	1954	1954	1954	1954	1936	1954
29	1915	1915	1915	1977	1940	1954	1915	1954	1954	1954	1936	1954
30	1915	1915	1915	1977	...	1954	1915	1954	1954	1954	1954	1954
31	1915	—	1915	1977	...	1954	...	1954	—	1954	1954	

analysis results for the coefficients are: m_2= 1.05802 and b= -0.40826. The expression may be written as a power function:

$$Q_D = 0.39061\ Q_{SM}^{1.105802} \qquad (2)$$

The correlation coefficient is 0.958 for this relationship, which indicates there is a better predictive relationship between the logarithms of the flows than can be provided using a simple linear model.

The average daily flow record at Ste. Marie for the years 1915, 1954, 1977, 1988, and 1989 was used to calculate the corresponding discharge at Diona using equation 2 above. Next, the ratio Q_D / Q_{SM} , between the predicted discharge at Diona (QD) and measured Ste. Marie flow (Q_{SM}), was calculated for each day. Table 4 shows a summary of the ratios calculated, including the minimum, maximum, and mean.

Table 4. Ratio of Predicted Discharge at Diona (QD) to Measured Ste. Marie Discharge (QSM) from Logarithmic Relationships

	Q_D/Q_{SM}		
Year	*Average*	*Minimum*	*Maximum*
1915	0.54	0.39	0.68
1954	0.47	0.40	0.55
1977	0.52	0.44	0.65
1988	0.54	0.45	0.68
1989	0.56	0.44	0.68
All years	0.52	0.39	0.68

The linear relationship between discharges forces a ratio, R_{linear} = QD / Q_{SM} = 0.57285, whereas for low flows the logarithmic relationship yields a typical ratio, R_{log} = Q_D / Q_{SM} = 0.52. The linear relationship would predict higher flows at Diona than the logarithmic relationship. Applying the ratio of the drainage areas, A_D / A_{SM} = 0.6062, would result in even higher estimations of QD.

The demand-storage-recurrence interval relationships presented in Bulletin 66 for the Embarras River at Ste. Marie use streamflow expressed as a percent of mean annual flow. The range of flows of interest are 0.25 to 8 times the demand. For a demand of 5 mgd, this corresponds to streamflows from 2 to 62 eft. Compared to the mean annual flow expected in the Embarras River at Riverview Dam, these are in the low flow range. The relationships in Bulletin 66 may be used with an appropriate adjustment.

The adjustment factor was computed by comparing the ratio of the discharges determined from the logarithmic relationship and the ratio of the drainage areas at the Diona and Ste. Marie stations. Using the drainage area ratio, discharge at Diona would be calculated as Q_D = 0.6062QSM During low flows the better estimate is $Q_D = 0.52\ Q_{SM}$. The factor of 0.52, if applied to the drainage area, would correspond to a drainage area 0.52 times 1516 sq. mi., or 788 sq. mi. This is 86 percent of the actual drainage area at Diona. The final adjustment factor, 0.57285/0.60620 or 0.86, was applied to calculate a modified drainage area and mean annual flow for use in the side-channel analysis along with data from Bulletin 66. Thus, the adjusted drainage area of the River at Riverview Dam is 786 sq. mi. times 0.86, or 676 sq. mi. In the yield calculations an adjusted mean annual flow (549 cfs, or 354 mgd) is used.

Runoff

The mean annual runoff at a point along the river is calculated by dividing the long-term average annual discharge by the drainage area; after applying appropriate unit conversion factors the runoff is expressed in inches. The mean annual runoff for the Embarras River at Ste. Marie for the period 1910 to 1992 is 1228 cfs, or 11.0 inches per year. This value was used in the CSCR yield analysis.

There is direct inflow to the CSCR from 1133 acres, including the 328 acres of the CSCR. A comparison of the pumping records for the river and the side-channel reservoir for the years 1984 through 1995 shows that for all but one year the water withdrawn from the side-channel reservoir exceeded the water pumped from the river, as shown in Table 5. The difference between

Table 5. Comparison of River and Reservoir Pumping

Year	*River pumpage (mgd)*	*CSCR pumpage (mgd)*	*CSCR - River difference (mgd)*	*Finished water (mgd)*	*Difference CSCR -finished (mgd)*
1984	1.086	1.729	0.643	1.680	0.049
1985	0.668	1.657	0.989	1.651	0.006
1986	1.483	1.656	0.173	1.663	-0.007
1987	1.320	1.804	0.484	1.600	0.204
1988	1.948	1.725	-0.223	1.646	0.079
1989	1.389	1.652	0.263	1.370	0.282
1990	0.881	1.527	0.646	1.341	0.186
1991	1.376	1.577	0.201	1.404	0.173
1992	1.529	1.862	0.333	1.582	0.280
1993	0.382	1.925	1.543	1.794	0.131
1994	0.854	2.192	1.338	1.894	0.298
1995	1.897	2.139	0.242	1.816	0.323

Summary

	Year	*River pumpage (mgd)*	*Year*	*CSCR - River difference (mgd)*
Minimum	1993	0.382	1988	-0.223
Average		1.234		0.553
Maximum	1988	1.948	1993	1.543

water delivered to the CSCR and the water withdrawn is attributed to the direct runoff to the CSCR from its watershed. Only in 1988 did water withdrawals from the river exceed pumping from the CSCR.

Rainfall records from a rain gage located near Charleston were obtained from the Midwest Climate Center at the Illinois State Water Survey. The total rainfall for each year from 1984 through 1994 is listed in Table 6. The table lists the difference between the water pumped from the river and the water pumped from the CSCR (from Table 5), expressed in inches on drainage area. The last two columns of the table provide a ranking of the year and the total recorded rainfall in ascending order of rainfall amount. During the period 1984-1994, the lowest total rainfall occurred in 1991, 75 percent of the 1984-1994 average for the gage. More precipitation occurred in 1988 (82 percent of average), but river withdrawals that year exceeded the CSCR withdrawals. On the basis of the Embarras River station data from Ste. Marie, streamflows in 1988 were in the normal range. The annual average discharge at Ste. Marie in 1988 is the 36th highest of 82 years of record, corresponding to about a 2-year return period.

Runoff from the CSCR has a positive contribution during periods of near average and above average rainfall. However, during a severe drought the contribution may not be significant. Extremely high evaporation rates during 1988 may have created the need for greater water withdrawals from the river that year. Total potential evapotranspiration calculated by Midwest Climate Center for their Springfield station is listed in table 6 for the period 1984-1994. Potential evapotranspiration in 1988 is 25 percent greater than in 1991. Precipitation on and evaporation from the surface area of the side-channel reservoir are accounted for in the yield analysis method described in Bulletin 66.

Table 6. Comparison of Difference in River and CSCR Pumping with Precipitation

	Pumping difference			*Annual total precipitation*	
Year	*ac-ft*	***inches(l)***	***total precip.***	***inches***	***% of average***
1984	720.28	7.63	20%	37.42	93
1985	1107.86	11.73	24%	49.06	121
1986	193.79	2.05	6%	35.9	89
1987	542.17	5.74	16%	36.02	89
1988	**-249.80**	-2.65		**33.45**	83
1989	294.61	3.12	7%	42.34	105
1990	723.64	7.66	15%	51.2	127
1991	**225.16**	**2.38**	8%	**30.35**	75
1992	373.02	3.95	10%	39.97	99
1993	1728.45	18.31	36%	50.89	126
1994	1498.81	15.87	42%	37.98	94
1995	271.08	2.87			
Average				40.42	

Year	*Ascending order of total annual precipitation, inches*	*Potential evapotranspiration, Springfield station, inches*
1991	**30.35**	**40.6**
1988	**33.45**	**50.3**
1986	35.90	43.7
1987	36.02	48.8
1984	37.42	42.3
1994	37.98	42.9
1992	39.97	39.1
1989	42.34	40.7
1985	49.06	43.2
1993	50.89	38.3
1990	51.20	41.4

Notes:

1) equivalent runoff in inches for drainage area of 1133 acres

Bold values indicate the two years with lowest recorded precipitation

SIDE-CHANNEL VOLUME AND SEDIMENTATION

CSCR volume and depths as of 1988 are reported in the Clean Lakes Program report *Phase 1 Diagnostic/Feasibility Study of the Charleston Side-channel Reservoir* (City of Charleston, 1992). The sources of sedimentation as well as the rate of sediment delivery to the CSCR are identified in this study. A summary of the sedimentation data from the report is provided in Table 7. The accumulation of sediment in the reservoir causes a reduction in the water storage capacity. Storage volume projections were developed for four possibilities: 1) sedimentation at the current rate; 2) a 20 percent reduction in sediment delivery to the reservoir; 3) removal of 789 acre-feet (ac-ft) of accumulated sediment and sedimentation at the current rate; and 4) removal of 789 ac-ft of accumulated sediment and a reduction in sediment delivery to the reservoir. A 20 percent reduction in sediment delivery was arbitrarily used to illustrate the impact on water supply.

Table 7. Sources of Sedimentation*

	Rate of-watershed total erosion		*Rate of sediment delivered to CSCR*		*Reduction of sediment delivery rate by 20 %*	
Source	*(tons/year)*	*(ac-ft/year)*	*(tons/year)*	*(ac-ft/year)*	*(tons/year)*	*(ac-ft/year)*
Land erosion	1191	1.19	715	0.72	572	0.57
Gully	1420	1.42	1349	1.35	1079	1.08
Shoreline	1000	1.00	1000	1.00	800	0.80
Pumping, Lake Charleston			300	0.30	300	0.30
Total	3611	3.61	3364	3.36	2751.2	2.75

Notes:
* Clean Lakes Program report *Phase 1 Diagnostic/Feasibility Study of the Charleston Side-channel Reservoir* (City of Charleston, 1992).
1 ac-ft of in-lake sediment -1000 tons considering dry density of sediment deposit of 46 lb/ft.^{3}

Water storage above the intake depth constitutes the usable (available) storage volume of the reservoir. The normal pool of the CSCR is 588 feet. The existing intake structure has operational inlets, where water may be withdrawn, at elevations of 583 and 579 feet. A third inlet at an elevation of 576 feet is sealed, but may be opened. Another inlet at an elevation of 572 feet has been abandoned and is below the current sediment level.

The surface area of the CSCR and the incremental storage volumes between selected elevations are provided in Table 8. The accompanying graphs (Figure 1) compare both surface area and storage volume with elevation for the CSCR. The storage volume above various elevations is provided in the column of Table 8 labeled "Incremental volume from 588 feet," and the inlet locations are noted in the last column. The storage volume above the inlets at elevations of 579 and 576 feet was used in separate yield analyses for the reservoir.

The volume of available water storage over time will be affected by sediment accumulation, the elevation of the inlets in the intake structure, and dredging activity to remove sediment. The impact of sediment accumulation on water storage is illustrated in Table 9, which lists future storage volume for two different rates of sediment delivery (from Table 7) for the years 2000, 2010, 2020, and 2030. The Clean Lakes Program report recommends that 789 ac-ft of sediment (about 27 percent of the 1988 volume) be removed. Table 9 also lists the volume of water storage available after dredging 789 ac-ft of sediment, and presents these four combinations as volume projections, identified as V1 through V4.

The volume versus elevation data provided in Table 8 were combined with the sediment accumulation over time in a summary of future water storage available above the inlet elevations of 579 and 576 feet (Table 9b). These estimates of storage were made assuming that sediment would accumulate in the deepest parts of the lake. The water storage above an elevation of 579 feet is not significantly affected by the accumulation of sediment. However, the volume of water storage above 576 feet does change over time.

Table 8. CSCR Surface Area and Volume

Elevation (ft, NGVD 1929)	*Surface area (acres)*	*Cumulative volume (ac-ft)*	*Incremental volume from bottom (ac-ft)*	*Incremental volume from 588 feet (ac-ft)*	*Notes*
588	339	2871	37	0	1
588	328	2833	1852	0	
582	289	981	525	1890	
580	236	456	219	2415	
579	203	237	152	2634	2
578	101	85	56	2786	
577	11	29	10	2842	
576	9	18	18	2853	3
572	0	0	0	2871	4

Notes:
1) With the additional volume and surface area of coves, 37 ac-ft and 11 acres, respectively
2) Elevation of west intake, open
3) Elevation of north intake, sealed
4) Elevation of east intake, sealed and below sediment level

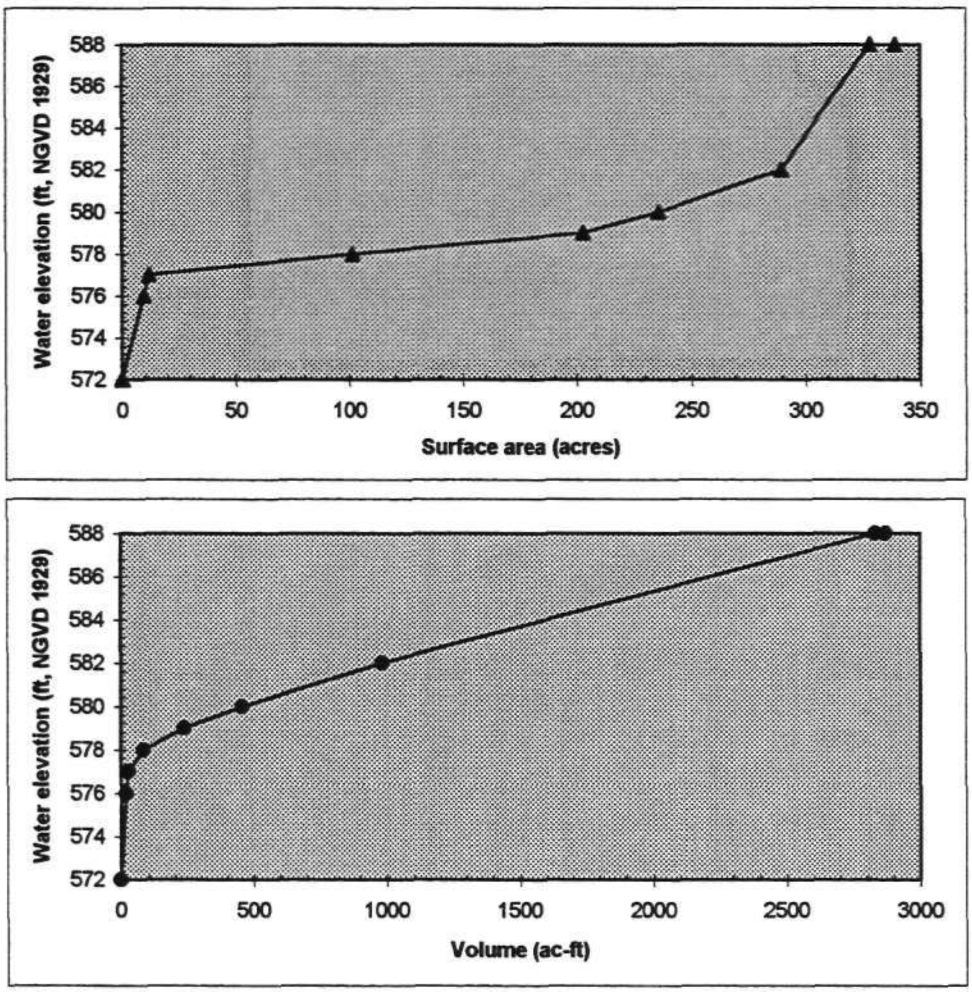

Figure 1. CSCR surface area and volume versus elevation

Table 9a. CSCR Volume:
Projections for Possible Sedimentation Rates

Identification	*Sedimentation rate (ac-ft/yr)*	*Volume, ac-ft*					
		1988	*1995*	*2000*	*2010*	*2020*	*2030*
		Starting capacity volume from 1988 survey					
V1	3.4	2871	2847.2	2830.2	2796.2	2762.2	2728.2
V2	2.8	2871	2851.4	2837.4	2809.4	2781.4	2753.4
		Volume after removing 789 ac-ft (1)					
V3	3.4	2871	2847.2	3619.2	3585.2	3551.2	3517.2
V4	2.8	2871	2851.4	3626.4	3598.4	3570.4	3542.4

Table 9b. CSCR Volume Above Inlet Elevations:
Projections for Possible Sedimentation Rates

Identification	*Description*	*Scenario reference*	*Available water volume, ac-ft*			
			2000	*2010*	*2020*	*2030*
V1,V2,V3,V4	above 579 feet	R1	2634	2634	2634	2634
V1	above 576 feet	R2	2812.2	2778.2	2744.2	2710.2
V2	above 576 feet		2819.4	2791.4	2763.4	2735.4
V3	above 576 feet		3601.2	3567.2	3533.2	3499.2
V4	above 576 feet	R3	3608.4	3580.4	3552.4	3524.4

Table 9c. CSCR Volume Above Inlet Elevations:
Projections for Possible Sedimentation Rates and Effect of One-foot Raise in Spillway Crest (2)

Identification	*Description*	*Scenario reference*	*Available water volume, ac-ft (2)*			
			2000	*2010*	*2020*	*2030*
V1,V2,V3,V4	above 579 feet	R4	2962	2962	2962	2962
V1	above 576 feet	R5	3140.2	3106.2	3072.2	3038.2
V4	above 576 feet	R6	3936.4	3908.4	3880.4	3852.4

Notes:
1) Recommendation in Clean Lakes Program *Phase 1 Diagnostic/Feasibility Study of the Charleston Side-Channel Reservoir* (City of Charleston, 1992)
2) One-foot raise in Charleston Side-channel spillway will increase storage by about 328 ac-ft

Another option for increasing the volume of stored water is to increase the CSCR spillway crest elevation. Table 9c shows the increase in water storage volume achieved by raising the spillway crest elevation along with the options of reducing sedimentation, rehabilitating the intake at 576 feet, and dredging 789 ac-ft from the reservoir. These scenarios of available water storage are identified as R4, R5, and R6.

Six scenarios of available water storage were selected for continued analyses to illustrate a range of possible yields. These options are identified in Table 9 as follows:

R1 - available water storage above the inlet at 579 feet, do nothing to reduce sediment input or accumulation;

R2 - rehabilitate the inlet at 576 feet but do not reduce sedimentation or dredge the CSCR;

R3 - rehabilitate the inlet, reduce sediment delivery, and dredge the reservoir;

R4 - available water storage above the inlet at 579 feet, do nothing to reduce sediment input or accumulation, raise the CSCR spillway crest one foot;

R5 - rehabilitate the inlet at 576 feet but do not reduce sedimentation or dredge the CSCR, raise the CSCR spillway crest one foot;

R6 - rehabilitate the inlet, reduce sediment delivery, dredge the reservoir, and raise the CSCR spillway crest one foot.

YIELD ANALYSIS

The sustainable (reliable) yield from the raw water supply may be defined as the maximum water withdrawal rate that can be expected to be consistently delivered during a drought of specific return frequency, such as a 20- or 50-year return interval. The rate of water withdrawal is an average. The term "return interval" is used to express the likelihood of occurrence of an event. A 20-year drought has a 1 in 20 chance of occurring in any given year (5 percent chance), while a 50-year drought has a 1 in 50 chance of occurring in any given year (2 percent chance). The longer the return interval, the more severe the drought. A drought with a 50-year return interval is more severe than a drought with a 20-year return interval. For a given reservoir storage volume, the greater the demand, the less severe (more frequently expected) drought that could be managed without water shortages; or in other words, the greater the risk of water shortages in any given year.

The reliable yield from the CSCR was evaluated from two different perspectives. First, the demand forecasts presented in Table 1 were considered. The ability of the system to meet these demands was measured in terms of the most severe drought during which the system could still reliably satisfy water demands. The water supply was also evaluated in terms of the raw water demand that could be meet for 20- and 50-year droughts for various pumping configurations.

MinimumFlow Levels

Currently there are no regulatory minimum flow levels; however, in the interest of protecting the downstream ecosystem, pumping from the river should not decrease the flow below some specified minimum level. For the present analysis it is assumed that pumping will only be allowed if flows exceed a certain minimum level. The minimum level is the lesser of either the flow that occurs 75 percent of the time (Q75) or the 7-day 10-year low flow. This is identified as the "primary relationship" in Bulletin 66.

During very low flow periods, little or no water will spill over the dam. Low flow statistics for the Embarras River at the CSCR were developed using the ranked historical 7-day low flows measured at Ste. Marie. The 7-day low flow is the average flow expected during a 7-day period for the specified return period. A summary of the low flows for the Embarras River at Ste. Marie and upstream and downstream of Riverview Dam on the main stem of the Embarras River near the CSCR is provided in Table 10.

Table 10. 7-day Low Flows

Location	*Return period*		
	*10-Year**	*20-Year*	*50-Year*
Ste. Marie	13.0 cfs	9.2 cfs	1.9 cfs
Above Riverview Dam	3.4 cfs	2.4 cfs	0.3 cfs
Below Riverview Dam	1.4 cfs		

* *7-Day 10-Year Low Flows of Streams in the Kankakee, Sangamon, Embarras, Little Wabash, and Southern Regions* (K.P. Singh, G.S. Ramamurthy, and I.W. Seo, Illinois State Water Survey Contract Report 441, 1988).

Evaporation losses from the water impounded behind Riverview Dam are especially significant during low flow periods (see Table 11). Should minimum flow standards be adopted, it will be necessary to consider these losses in an analysis of pumping schedules. Net evaporation for a specified duration and return interval is obtained by subtracting precipitation from evaporation over the water body under consideration.

River Pumping System Configuration

Streamflow varies throughout the year. High flows occur typically in the fall and the spring, and low flows occur during the summer months. The water stored in the CSCR is used to supplement the pumpage from the river when less than the full water demand can be withdrawn from it. Theoretically, to fully use the river flow, the optimum pumping system would need the capacity to pump water from the river to the CSCR over a wide range of river flows. This would require variable-speed pumps and a carefully monitored pumping schedule. However, pumping at very high rates can create high velocities in the river near the intake structure and the dam. It is more common to have several fixed-speed pumps that can be operated individually or jointly to allow pumping during a range of flows. Yields for four river pumping systems were calculated: 1) theoretical optimum system (two variable-speed pumps); 2) existing system (two fixed-speed pumps); 3) three fixed-speed pumps; and 4) combination system with one fixed-speed and one variable-speed pump. There are many possible pumping configurations. However, for this

Table 11. Net Evaporation Losses for Lake Charleston In-channel Storage

Duration months	*Net evaporation (1) inches*	*Net,evaporation losses (2)*		
		ac-ft/day	*mgd*	*cfs*
10-Year return interval				
1	6.38	2.22	0.72	1.12
6	18.75	1.09	0.35	0.55
9	17.67	0.68	0.22	0.34
12	16.41	0.48	0.15	0.24
20-Year return interval				
6	19.80	1.15	0.37	0.58
9	17.94	0.69	0.23	0.35
12	16.69	0.48	0.16	0.24
50-Year return interval				
6	22.15	1.28	0.42	0.65
9	21.06	0.81	0.26	0.41
12	20.20	0.58	0.19	0.30

Notes:
1) Total for return period and duration
2) Surface area of 195 acres at elevation 580 feet (NGVD 1929)

evaluation, it was assumed that the two 4000-gallon per minute (gpm), fixed-speed pumps currently installed would remain. Only pumping system configurations that could be achieved by adding an additional fixed-speed or variable-speed pump to the system were evaluated.

A discussion of the effect of pumping system configuration is presented in Bulletin 66. The demand-storage-recurrence relationship presented for the Embarras River shows the storage requirements assuming that an optimum pumping system is in place. The optimum system is defined in Bulletin 66 as having the capacity to pump continuously over a range of flows from 0.25 to 8 times the raw water demand. For a specific pumping system, the storage required for a given demand and recurrence interval can be described as a multiple of the storage determined from the optimum demand-storage-recurrence relationship. This multiple is termed the "pumping system adjustment ratio." Adjustment ratios are listed in Bulletin 66 for various pumping configurations. Ratios from Bulletin 66 that correspond to pumping systems similar to the current system and possible augmented pumping arrangements were used to determine whether a higher demand could be served when additional pumps were added.

The pump capacity, usually given in gallons per minute (gpm), may be expressed in millions of gallons per day (mgd). The present system has two pumps that individually can pump at a rate of 4000 gpm (5.76 mgd) and together can pump at 7500 gpm (10.80 mgd). Running three pumps yields a capacity of 9600 gpm (13.82 mgd), but the third pump is a back-up pump. The engineering firm of Beam, Longest and Neff reported in February 1981 that the three pumps should not be run simultaneously under any circumstances. The resulting scour from the outlet velocity could possibly undercut the dike (personal correspondence from Alan Alford to Mark Donnelly, November 17, 1995). On the basis of this recommendation, the pumping capacity at the intake structure is limited to about 10.8 mgd.

The purpose of using multiple pumps is to permit pumping over a wide range of flows, from low flows that are less than the demand to high flows that are greater than the demand. The problem with scour limits the maximum pumping capacity. The addition of another fixed-speed or variable-speed pump would allow pumping at flows less than 4000 gpm and/or that part of the flow between 4000 gpm and 7500 gpm. Ratios of the pumping capacity to the demand are listed in Table 12 for demands of 2 to 6 mgd for the existing system and two alternate systems.

The two alternate systems listed in Table 12 are: addition of one 1500-gpm fixed-speed pump and addition of one 4000-gpm variable-speed pump. These alternatives were selected to illustrate the option of modifying the current system. Both options provide a wide range of pumping combinations for demands from 2 to 6 mgd.

Table 12. Pumping Capacity Expressed as Demand and Pumping System Adjustment Ratios

	Pumping capacity		*Pumpinz capacity as a ratio of demand*				
Pumping system	*gpm*	*mgd*	*2 mgd*	*3 mgd*	*4 mgd*	*5 mgd*	*6 mgd*
Present system (3 fixed-speed pumps*)							
One pump	4000	5.8	2.9	1.9	1.4	1.2	1.0
Two pumps combined	7500	10.8	5.4	3.6	2.7	2.2	1.8
Three pumps combined*	9600	13.8	6.9	4.6	3.5	2.8	2.3
Pumping system adjustment ratio			1.6	1.55	1.35	1.35	1.35
Present system with addition of							
one fixed-speed pump	1500	2.2	1.1	0.7	0.5	0.4	0.4
Pumping system adjustment ratio			1.35	1.35	1.30	1.31	1.32
Present system with addition of							
one variable-speed pump	4000**	5.8	2.9	1.9	1.4	1.2	1.0
Pumping system adjustment ratio			1.12	1.16	1.16	1.20	1.20

* Third pump is a back-up pump, engineering recommendation is never to operate all three pumps simultaneously due to the high velocities generated.
** Maximum speed

The methodology presented in Bulletin 66 does not address the situation of an in-channel dam that maintains the water level at the intakes and provides some storage. During low flows very little water flows over the dam, as illustrated by the general relationship between the height of water above a spillway and the discharge over the spillway. Application of the weir equation for discharge with very small approach velocity is illustrated in Table 13 for Riverview Dam across the Embarras River.

Table 13. Riverview Dam Spillway Discharges

$Q_d = C_w x L x h^{3/2}$
Q_d = discharge over the dam, cfs
C_w = 3.0, spillway discharge coefficient
L = length of the spillway, 420 feet
h = height of water surface above the spillway, feet

h (feet)	*Qd(cfs)*	*Qd(mgd)*
0.05	14	9
0.1	40	26
0.5	445	287
0.63	637	411

Note: Approach velocity assumed to be zero in calculations.

During low flows, when the inflow above Riverview Dam is less than demand, pumping may still continue drawing upon stored water as well as the inflow. The installation of a small capacity pump to operate during such low flows may not achieve the full benefit suggested by the tabulated pumping system adjustment ratios from Bulletin 66 and listed in Table 12.

Drought Return Period for Projected Water Demands

The ability of the system to meet demand was assessed in terms of the longest drought that could be endured without water shortages (for the given demand). The two demand projections (identified as D1 and D2) presented in Table 1 were combined with the six possible options for side-channel storage (identified as R1 through R6). The ability of the system to meet the demand is tabulated as the return interval drought that could be managed for the demand projection for the years 2000, 2010, 2020, and 2030, and the volume of available water reserves.

The calculations follow the procedure described in Bulletin 66. The final step in the calculations is to use the graphical relationship shown in Figure 2 from Bulletin 66. For the specified demand (expressed as a percent of the mean annual flow) and the available storage (expressed as days of demand), the corresponding return interval is interpolated from the graph (Figure 2). Results of the analysis for the demand projections and the six possible water storage volume options are listed in Table 14.

The information shown in Table 14 indicates that the present system is adequate to meet the demands (D1) projected by Broeren and Singh (1989) through 2020 for droughts with a return interval greater than 50 years. If nothing is done to modify the volume of stored water available, the return interval decreases to 31 years in 2030.

Through the year 2020, if demand does show a significant increase (D2), and a drought more severe than one expected every 20 or so years occurrs, there may be water shortages if water use is not restricted. However, a more severe drought can be managed if 789 ac-ft of sediment is removed and the intake at 576 feet is rehabilitated.

Storage Volume and Pumping Systems to Meet Various Demands During 20- and 50-Year Droughts

Another way to estimate the demand that may be reliably met from the water supply source is to calculate the volume of storage needed for a given demand and recurrence interval. The storage needed to meet demands of 2 to 6 mgd for droughts having 20- and 50-year return intervals was calculated. The various demand levels and raw water storage needs are listed in Table 15.

The benefit of augmenting the current pumping system is illustrated by the reduced storage volume needs listed in Table 15. However, the tabulated values show that adding a fixed-speed or variable-speed pump provides only a very modest decrease in the storage requirements. A demand of 4 mgd can be met during a 50-year drought using the present pumping system and opening the

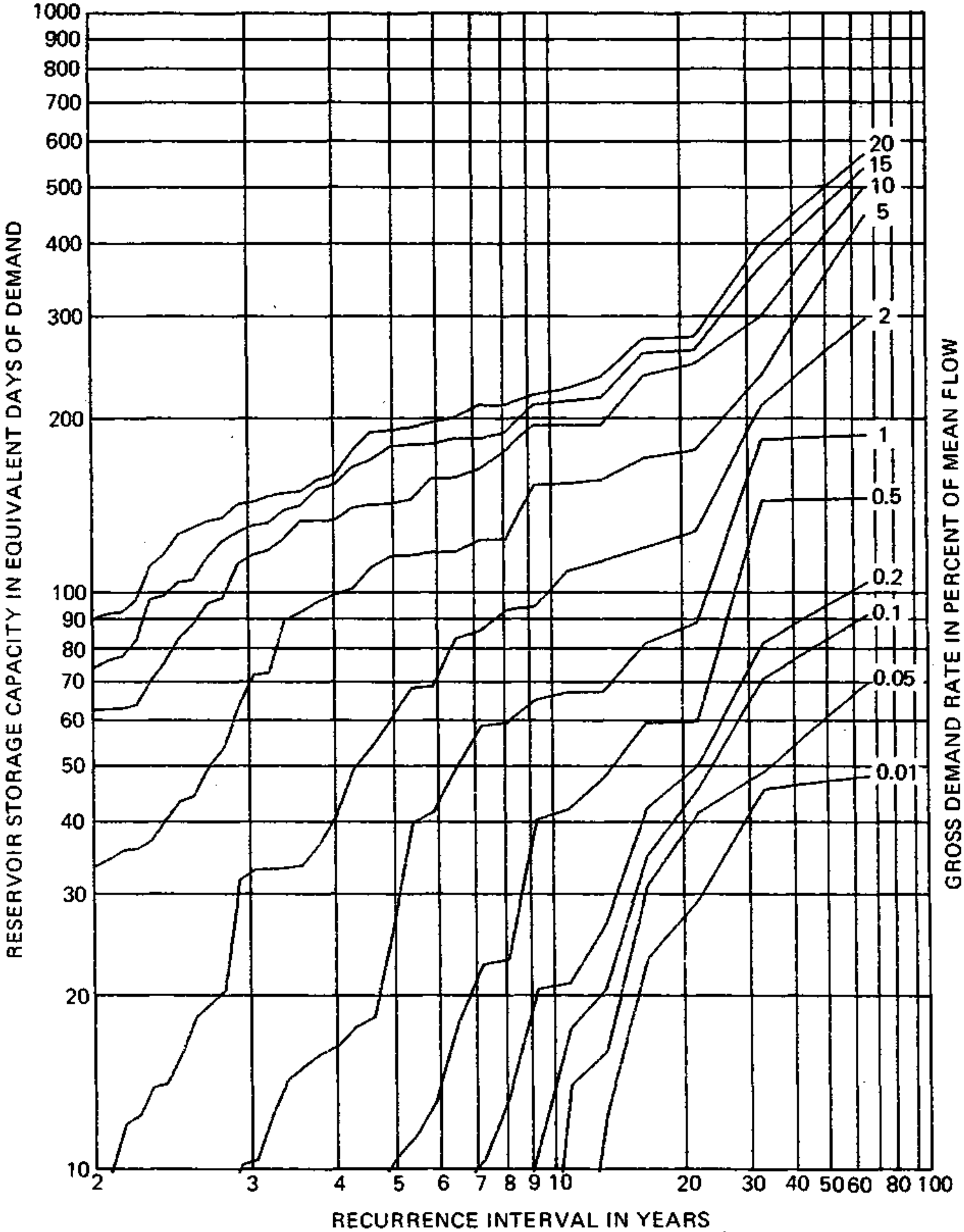

Figure 2. Embarras River at Ste. Marie, demand-storage-recurrence primary relationship (after *Hydrologic Design of Side-Channel Reservoirs in Illinois,* H.V. Knapp, Illinois State Water Survey Bulletin 66, 1982)

Table 14a. Summary of Expected Drought Recurrence Intervals for Combinations of Demand and Water Storage Volume, D1

	2000	*2010*	*2020*	*2030*
Average annual demand:	2.04 mgd	2.29 mgd	2.58 mgd	2.92 mgd
Drought demand, Dl:	2.66 mgd	2.98 mgd	3.35 mgd	3.80 mgd

Reservoir storage scenario	*Drought recurrence interval in years*			
R1	>50	>50	>50	31
R2	>50	>50	>50	31
R3	>50	>50	>50	>50
R4	>50	>50	>50	>50
R5	>50	>50	>50	>50
R6	>50	>50	>50	>50

Table 14b. Summary of Expected Drought Recurrence Intervals for Combinations of Demand and Water Storage Volume, D2

	2000	*2010*	*2020*	*2030*
Average annual demand:	3.36 mgd	3.80 mgd	*Ml* mgd	4.76 mgd
Drought demand, D2:	4.37 mgd	4.94 mgd	5.55 mgd	6.19 mgd

Reservoir storage scenario	*Drought recurrence interval in years*			
Rl	26	22	21	12
R2	28	26	22	14
R3	50	30	26	25
R4	30	26	23	20
R5	31	28	24	21
R6	>50	32	30	27

Table 15. CSCR Storage Requirements for Various River Pumping System Options

Yield (mgd)	Water Storage Requirements (ac-ft) Optimum system	Present system	Additional 1500-gpm fixed-speed pump	Additional 4000-gpm variable-speed pump
		20-year return interval		
2.0	400	700	600	500
3.0	700	1100	1000	800
4.0	1100	1500	1500	1300
5.0	1600	2100	2000	1900
6.0	2200	3200	2900	2700
		50-year return interval		
2.0	1000	1500	1300	1100
3.0	1600	2500	2200	1900
4.0	2400	3200	3100	2800
5.0	3300	4400	4300	3900
6.0	4200	5600	5500	5000

Storage volume in 2030 for various scenarios (from Table 9):

Scenario reference number	*Water storage volume (ac-ft)*
R1	2634
R2	2710
R3	3524
R4	2962
R5	3038
R6	3852

Notes:

R1) Available water storage above the inlet at 579 feet; do nothing to reduce sediment input or accumulation.

R2) Rehabilitate the inlet at 576 feet, do not reduce sedimentation or dredge the CSCR.

R3) Rehabilitate the inlet, reduce sediment delivery, and dredge the reservoir.

R4) Available water storage above the inlet at 579 feet; do nothing to reduce sediment input or accumulation, raise CSCR spillway one foot.

R5) Rehabilitate the inlet at 576 feet, do not reduce sedimentation or dredge the CSCR raise CSCR spillway one foot.

R6) Rehabilitate the inlet, reduce sediment delivery, and dredge the reservoir, and raise CSCR spillway one foot.

Shaded values exceed the maximum storage that can be achieved for the alternatives considered.

CSCR intake at 576 feet. A benefit from installation of an additional pump may be realized if demand grows to 5 or 6 mgd. However, even with an optimum system, additional storage beyond the alternatives explored would ultimately be needed. In Table 15 the highlighted storage volumes exceed the maximum storage that could be achieved for the alternatives considered.

In-channel Water Storage and River Intake Depth

Water impounded upstream of Riverview Dam can augment the water supply. The spillway elevation is 580.75 feet (NGVD 1929), and the bed elevation at the river intake structure is 576 feet. The top of the existing intake structure is about 579.5 feet. Lowering the intake structure may increase the water volume available for pumping. The maximum increase in the intake depth is only about 0.8 feet, which would make the water stored between 579.5 and 578.7 feet available for pumping at the structure. In an emergency situation an auxiliary mobile pump could be used to capture this water.

The in-channel volume upstream of Riverview Dam was measured after the dam failure in 1985 and reported in *Channel Scour Induced by Spillway Failure at Lake Charleston, Illinois* (Misganaw Demissie, William C. Bogner, Vassilios Tsihrintzis, and Nani G. Bhowmik, 1986, Illinois State Water Survey Contract Report 409). About 140 ac-ft of sediment was scoured from the channel after the dam failure. After the scour occurred, the total volume in the first 15,000 feet upstream of the dam, below an elevation of 580 feet, was 366 ac-ft. Assuming the 140 ac-ft created by scouring has been or will be refilled with sediment, the expected total storage volume of the impoundment below 580 feet is 226 ac-ft.

The storage volume between 579.5 and 578.7 feet is about 140 ac-ft or 45 million gallons. Given a demand of 2.5 mgd, this would provide an extra 18 days of supply; given a demand of 4 mgd, this would provide about 11 extra days of supply.

SUMMARY

The risk of experiencing water shortages serves as the standard for assessing the reliability of a surface water supply. An acceptable risk may be a little less than a five percent chance of not meeting demand in any given year (20-year return interval drought). It may be acceptable to impose water restrictions to limit or circumvent elevated water use during dry periods. Imposition of such restrictions would reduce the "drought demand" (computed in this evaluation as 1.3 times the average annual demand).

The reliability of the raw water supply at Charleston may be viewed in terms of its ability to reliably supply adequate water to meet demands into the future. Future demands on water will most likely increase at a moderate rate. However, new industry or commercial activities can increase water demand dramatically in a short period of time. There are numerous combinations and possibilities for water demand increases. Two possible forecasts of water use were used to evaluate the ability of the raw water supply to meet demand. With the information available at this time, the two forecasts show: 1) the impact of moderate increases in water use, and 2) the maximum increase in demand that can be reasonably projected.

Alternatively, the raw water supply may be evaluated in terms of the storage requirements to meet a specific demand This assessment was also performed, and it shows the raw water storage needed to reliably meet demands from 2 to 6 mgd.

Side-channel storage provides many benefits over an in-channel reservoir. In the case of the Embarras River the loss of capacity due to sediment accumulation is significantly less for the side-channel reservoir than in the main channel pool. However, the in-channel dam does enhance the system by maintaining a water level that allows the intakes to be submerged during low flow periods and by providing some water storage. The side-channel system is fairly complex. System parameters that determine the yield of the raw water supply include: river flow patterns, the volume of available stored water, the river pumping system, evaporation losses, and minimum flow requirements. Each of these parameters and their role in determining yields from the system are discussed in the report.

If water demand increases at a moderate rate, the present system can meet demands during droughts with a recurrence interval of greater than 50 years through the year 2020. If demand does show a significant increase (D2), and a drought more severe than one expected every 20 or so years occurrs, there may be water shortages. Shortages can be minimized if water use is restricted. A more severe drought can be managed if 789 ac-ft of sediment is removed and the intake at 576 feet is rehabilitated.

Given the side-channel storage estimated for the year 2030, a raw water yield of 4 mgd can be sustained during a 50-year drought if the CSCR inlet at 576 feet is reopened. Similarly, a raw water yield of nearly 5 mgd can be achieved with modifications to the system, including the addition of a variable-speed pump, dredging, raising the CSCR spillway, and opening the reservoir inlet at 576 feet. Even with an optimum pumping system, a demand of 6 mgd cannot be met during a 50-year drought unless more storage is made available than the options considered.

Charleston's raw water supply system appears to be adequate to meet demands up to about double the current demand. Should demand for water triple in the next 30 or so years, additional raw water storage may be needed or water restrictions should be imposed during very dry periods to safeguard reserve water supplies. If instream flow standards are imposed, it may be necessary to design a pumping schedule, and re-evaluate the raw water storage needs.

ILLINOIS
DEPARTMENT OF
NATURAL
RESOURCES