Lake Michigan Diversion Accounting

Lakefront Accounting
Technical Analysis

Introduction

1. In response to a request from the Great Lakes Mediation Committee (meeting date December 11-13, 1995), the Chicago District has completed a period of record analysis of long term runoff and an analysis of the consumptive loss of water supply for use in the accounting of Lake Michigan waters diverted by the State of Illinois.

2. Currently the primary measurement point for diversion accounting is at Romeoville, Illinois on the Chicago and Sanitary Ship Canal (CSSC). At this point, approximately 94% of the diversion flows accountable to Illinois are measured through an acoustic velocity meter (AVM). Included in this measurement are component flows which are not part of the diversion. These flows include groundwater (domestic supply and Tunnel and Reservoir Plan (TARP) seepage), Indiana water supply, Lake Michigan water pumped by Federal facilities, and runoff from the Des Plaines River watershed. Although the AVM at Romeoville measures approximately 94% of the diversion, flows accountable to Illinois also bypass this measurement point. These flows are Lake Michigan water supply pumpage to communities whose sewage effluent is discharged to streams or rivers which are not tributary to the CSSC and, thus, the AVM. To compute the diversion of water by the state of Illinois the above flows must be determined either through measurement or simulation and then subtracted from (not accountable flows) or added to (bypassed flows) the AVM gage record.

3. While this procedure for computing Illinois' diversion is prescribed in the Supreme Court Decree, it has proven to be somewhat cumbersome and time consuming due to the large amounts of data that are required and the extensive computer simulations that are involved. Additionally, the hydrologic and hydraulic characteristics of the watershed are not static, but instead are ever changing. Consequently, computer simulation models require periodic updating, and reconnaissance missions are required to verify and update the accounting procedures. Sources of revisions to the accounting system include: new industrial water users; changes in the domestic water supply uses and distribution; modifications to sewage treatment plant service areas or sanitary flows; the opening of new local sanitary treatment plants; changes to TARP with respect to new tunnels or interceptor connections; and numerous other dynamic aspects which must be accounted for with respect to their impact on diversion accounting.

4. As part of the Great Lakes Mediation process, modifications in the methodology for computing the diversion are being considered. The primary measurement point for diversion accounting would be moved to the lakefront. This move would involve the installation of AVMs at the Chicago River Controlling Works (CRCW), the O'Brien Lock and Dam on the Calumet River, and the Wilmette Controlling Works on the North Shore Channel. If the lakefront AVMs prove to provide accurate and consistent results during low flow conditions, they will provide a direct measurement of the direct diversion flows: lockages, leakages, discretionary flow, and navigation makeup.

5. Through the use of lakefront AVMs, the revised procedure for computing the diversion could consist of the additions of direct diversions, water supply, and a negotiated value of runoff, followed by the subtraction of a negotiated value of consumptive use. Direct diversions would be measured at CRCW, O'Brien Lock and Dam and Wilmette Controlling Works. Lake Michigan water supply pumpages from primary (first order) users would be summed and federal pumpages subtracted along with an agreed upon consumptive use. Runoff diverted from the Lake Michigan watershed would be an agreed upon constant value based on an average runoff determined through a period of record simulation. The consumptive use credit would be negotiated and could be either a fixed value or a fixed percentage of the water supply. This report details the period of record methodology used in determining a constant average diverted runoff, and the continuous period methodology used to explore potential consumptive use values.

Period of Record Analysis

6. The hydrologic and hydraulic modeling developed to determine the period of record runoff is detailed in appendix A. The total runoff from the Lake Michigan watershed is computed by summing the following five elements:

- The total inflow and infiltration components of interceptor and overflows for all 137 Special Contribution Areas (SCAs) found within the Lake Michigan watershed and within the three Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) water reclamation plant (WRP) service areas.

- The total runoff, sewered and unsewered, from the 84 square mile "ungaged" Calumet watershed.

- Runoff from streamflow separation techniques applied at two streamflow gages on the North Branch Chicago and Little Calumet Rivers.

- Runoff from streamflow separation and a simulation analysis for the Grand Calumet River.

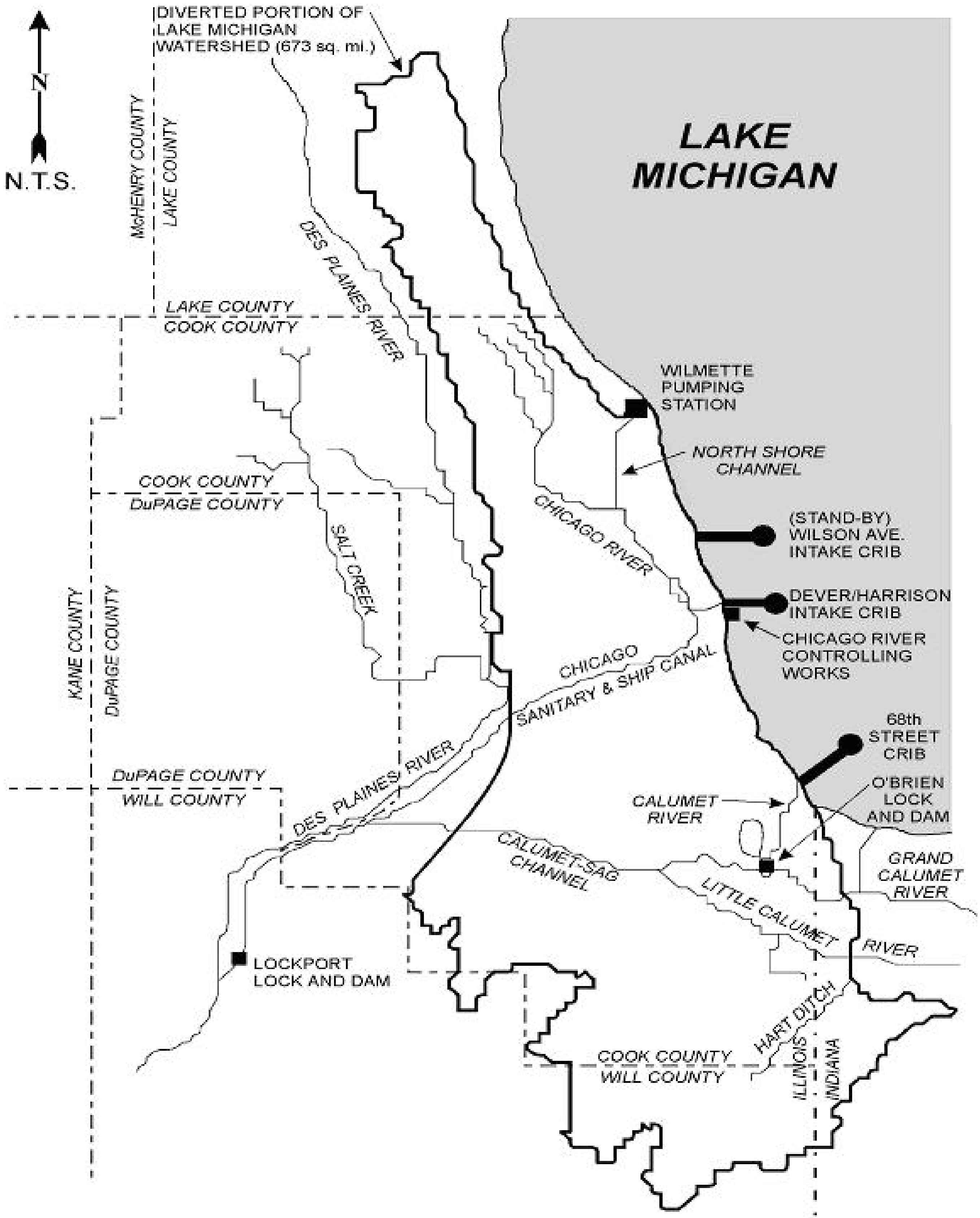
- The baseflow entering the canal and watershed channels between the gages and the downstream end of the diverted watershed.

7. The first two items above are computed using the WY 90 diversion accounting simulations models. The models have been calibrated against the three MWRDGC WRP influent pumpage records. Statistical analyses at the three MWRDGC WRPs, for each Water Year, show a good correlation, both with respect to the correlation coefficient and the simulated to recorded ratios. However, prior to WY 90 the total simulated flows were somewhat less than the total recorded inflows at the three water reclamation plants. The revised models used for WY 90 and thereafter show total simulated flows that are slightly higher than the total recorded inflows at the water reclamation plants. Specific details can be found within the individual accounting reports published within the Lake Michigan Diversion Accounting Annual Reports. Additionally, the simulated ungaged Calumet Watershed has been found, during WY 90 modeling revisions, to contain significantly more impervious area than was modeled prior to WY 90.

8. The runoff from remaining areas of the Lake Michigan watershed was based primarily on stream gage records. These areas include the runoff from the northern and southeastern extents of the watershed. In these areas, a streamflow separation technique was used, in which estimated sanitary discharges upstream of a stream gage are subtracted from a stream gage record to determine the portion of streamflow that is runoff. The total simulated area is approximately 361 square miles, while the total area using streamflow separation techniques is approximately 312 square miles. Some areas overlap in that they fall within both the simulated area and the stream gaged area. These areas are separately sewered where the sanitary sewers convey flow to the water reclamation plants while the storm sewers discharge into streams to be measured by the gages. Overlapping areas were generally classified as gaged areas. See the map on the next page for the locations of simulated, gaged and ungaged areas.

9. The Chicago District and the Illinois Office of the United States Geological Survey undertook an analysis of the ground water discharge to the canals and watercourses downstream of the gages within the diverted watershed. The subject streams include portions of: the Chicago River, North Branch Chicago River, South Fork of the South Branch of the Chicago River, the Chicago Sanitary and Ship Canal, the Calumet River, the Grand Calumet River and the Cal-Sag Channel. This total annual baseflow is 4.0 cfs.

10. The annual runoff from the diverted watershed was computed by summing the simulated flows (for the SCAs and the ungaged Calumet watershed), the gaged flows (from the North Branch Chicago, Little Calumet and Grand Calumet Rivers) plus the baseflow. The result of the WY 51-94 continuous period simulation of the diverted Lake Michigan diversion accounting runoff over the period of record is 785.2 cfs.



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SALT CREEK
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CHICAGO RIVER CONTROLLING WORKS
KANE COUNTY
DuPAGE COUNTY
CHICAGO SANITARY & SHIP CANAL
68th STREET CRIB
DuPAGE COUNTY
WILL COUNTY
DES PLAINES RIVER
CALUMET RIVER
O'BRIEN LOCK AND DAM
CALUMET-SAG CHANNEL
GRAND CALUMET RIVER
LITTLE CALUMET RIVER
LOCKPORT LOCK AND DAM
HART DITCH
COOK COUNTY
WILL COUNTY
ILLINOIS
INDIANA

Runoff Sensitivity Analyses

11. To gain a better understanding of the long-term runoff values a series of comparisons and analyses have been undertaken (see appendix B for the detailed documentation). These evaluations include:

- A comparison was made between the Chicago District's period of record analysis and the analysis of long term average runoff values conducted by the Northeastern Illinois Planning Commission (NIPC). The runoff study by NIPC resulted in an annual runoff of 636 cfs while the Chicago District's study resulted in an annual average of 785 cfs (including 4.0 cfs for baseflow). A comparison of methodologies used provided a rationale for the difference in results. Four primary differences in methodology were evaluated: the period of record; the model parameters; the determination of runoff from streamflow areas; and the precipitation data employed in the models

- A comparison of the period of record runoff values with those computed for the diversion accounting reports was undertaken. Statistical analyses were presented, contrasting the differences in the results. Also changes in the accounting procedures were presented to both contrast the differences in the techniques, and to provide a perspective on the development of the runoff models.

- A series of trend analyses consisting of comparison of the increases over time in station rainfalls, modeled rainfalls, and modeled runoffs. Generally, the results showed that rainfall and runoff were increasing over time, but at a diminishing rate.

- A sensitivity analysis of the rainfall gages utilized was performed. The average runoff computed over the 5-year period, WY 90-94, was 866.2 for the 3-gage period of record study. Using only the Midway gage (the procedure NIPC used) resulted in increasing the average annual runoff for the 5-year period to 887.4 cfs for a 2.4 percent increase. Using 20 of the 25 gages currently employed in the accounting of Lake Michigan diversion resulted in increasing the average annual runoff for the 5-year period to 916.0 cfs for a 5.6 percent increase.

- A sensitivity analysis of the effects of imperviousness was undertaken, and the average runoff computed for the 5-year period, WY 90-94, using the impervious and pervious breakdowns applied in the period of record study was 866.2 cfs. Increasing the impervious areas by 10 percent resulted in increasing the average annual runoff for the 5-year period to 885.9 cfs for a 2.3 percent increase. Decreasing the impervious areas by 10 percent resulted in decreasing the average annual runoff for the 5-year period to 846.5 cfs for a 2.3 percent decrease.

- A comparison with the historic record at Lockport was made using a rather involved procedure. The results included, for the period of record WY 51-95, estimated AVM flows, estimated long term diverted flows and estimated long term deviations. The long term average estimated diverted flow is 3,537 cfs. Diverted flows and imbalances were also estimated for WY 93-95, based on the best current data, the results are:

Water Year	Diverted Flow	Imbalance
1993	3,946 cfs	217 cfs
1994	2,960 cfs	-58 cfs
1995	3,107 cfs	22 cfs

- Mass balances were prepared for total rainfall versus total runoff plus groundwater and evapotranspiration, total runoff versus the runoff components, sewer inflows versus sewer outflows, and total overflows versus flows to TARP plus flows to the canals and rivers.

12. A summary of the sensitivity results is presented below in the form of a table (similar to the type of table HEC used in its comments on the draft report). Each item has a qualitative sense of concern, as well as the impact that item may have on the period of record results.

Issue	Concern	Impact
Chicago's analysis versus NIPC's	Moderate	Low
Period of record versus accounting flows	Moderate	Low
Trend analyses of rainfall and runoff values	Moderate	Low
Sensitivity analysis of the rainfall gages	High	High
Sensitivity analysis of imperviousness	High	High
Comparison with the record at Lockport	Moderate	Low
Mass balance of rainfall and runoff	Low	Low

13. The high concern and high impact issues, the sensitivity analyses of the range gages and the imperviousness, generate the largest uncertainty in the runoff value. Clearly, it would be desirable to use more rainfall gages for computing the period of record; however the three that were used are the only long term gages available in the basin. A further review of the rain gage sensitivity analysis could serve to diminish the concerns. The uncertainty in the correct values of imperviousness has been discussed in detail in the runoff appendix (with respect to the ungaged Calumet area and the overflows). The impact of this is significant for the period of record analysis, but not for the diversion accounting reports. Unfortunately, the solution to this problem would involve the evaluation of hydraulic connectivity of impervious areas for all contributing areas, which is outside the scope of this analysis.

Consumptive Loss of Domestic Water Supply

14. The methodology used to estimate consumptive loss is described in detail in appendix C, and consists of subtracting the water reclamation plant (WRP) influent from the total water supply (the sum of the pumpage from Lake Michigan plus ground water). The major difficulty in accomplishing this computation is the isolation and removal of the stormwater discharge that flows through the sewer system. Stormwater discharge is a persistent problem in that the resulting inflow and infiltration (I&I) into the sewer system can last for long periods of time. The I&I complicates the computation of consumptive loss because it is part of the runoff and not part of the water supply, yet it masks the quantity of sanitary flow that is influent to the WRPs.

15. The procedures used to compute consumptive loss required the use of the rainfall - runoff models described in appendix A. With the use of these models a variety of techniques were employed in attempting to eliminate the effects of stormwater discharge. The main approach required limiting of the comparisons of WRP influent to water supply (i.e. computing consumptive loss) for only dry weather periods. As a sensitivity analysis, a second approach involved merging the results of the dry weather analysis into the entire continuous period analysis. Finally, an additional series of sensitivity analyses were carried out to develop probable bounds on the value of consumptive loss.

16. In general, it is difficult to even select a potential range of consumptive use values from the analysis. However, if extreme values are discounted, a potential range of 8% to 12% can be derived. It should be noted that this range is low compared to other "accepted" ranges of 10% ("book" value) to 16% (International Joint Commission). As a final note, a consumptive use value was required for the computations of the imbalance in the sensitivity analysis (appendix B). Without any recommendation, the results from merging the dry-weather flows into the continuous analysis (8.7 %) were utilized.

Monte Carlo Simulation
Comparison of Lockport vs. Lakefront Measurements

17. The Chicago District (with technical support from the United States Geological Survey) has undertaken an error analysis of the accounting flows (see appendix D). The intent of the analysis is to provide a comparison of the existing Lockport based Lake Michigan diversion accounting system with potential Lakefront based accounting systems. In performing this analysis three "Monte Carlo" simulations were completed. These simulations can be used to compare the existing accounting system (LOCKPORT) with a direct movement of the accounting system to the lakefront (LAKEFRONT), and with a possible future lakefront accounting system (FUTURE). For further explanation of the different accounting systems please refer to appendix D.

18. The simulations were completed to compare errors in the existing diversion accounting system with errors from a direct movement to the lakefront, and with errors in a possible future accounting system. The standard deviation for the existing system is

184 cfs, for the lakefront system it is 309 cfs, and for the future system it is 190 cfs. Two general conclusions can be reached from this analysis of errors:

- If the two systems that account for all of the flow past Lockport are considered (i.e. LOCKPORT and LAKEFRONT), then the present accounting system is clearly more accurate. This is primarily due to the fact that errors are smaller when computing runoff and consumptive use values than in assuming fixed values.

- However, if a complete revision to the accounting can be negotiated, and a new total flow at the lakefront adopted (i.e. replacing the 3,200 cfs), then an accounting system equivalent to the FUTURE system would be in effect. In comparison to the LOCKPORT system, this new possible system would not significantly alter the level of accuracy.

Responses to Comments on Draft Report

19. The draft report of this document, dated 31 January 1996, was submitted to the Corps of Engineers' Hydrologic Engineering Center (HEC), and the parties of the Great Lakes mediation process for review. Comments were received from HEC, the State of Illinois and the State of New York. This report provides copies of the comments and the District's responses (appendix E).

Conclusions

20. The major results from the report are:

- Th computed period of record runoff value was 785 cfs.

- The two major issues with the runoff number concern the impacts of the rainfall gages and the selected values of the imperviousness.

- The continuous period modeling did not produce an adequate value of consumptive use. The effects of stormwater runoff could not be removed from the computations. A possible of range of 8% to 12% was mentioned.

- The error analysis showed that for a complete accounting of flows diverted from the basin, the current Lockport based accounting system is the most accurate. However, if runoff and consumptive use values can be negotiated and removed from the analysis, a lakefront based accounting system would produce smaller errors.

21. The overall conclusion from this report is that the values or ranges of runoff and consumptive use that have been produced have some level of uncertainty. However, the level of uncertainty in all likelihood is low enough to enable negotiated values to be adopted and thus allow a switch to lakefront accounting to occur. Finally, if the runoff

and consumptive use values are excluded from the new accounting system, the errors in diversion accountable to the State of Illinois will be consistent with the current accounting system.

22. A change to lakefront accounting would also significantly alter the Corps of Engineers diversion accounting mission. The Corps' role would change from one of primarily computer simulation, to one of quality assurance / quality control (QA/QC). The most important aspects of this new QA/QC program would be:

- Monitoring the lakefront AVM's
- Inspecting and monitoring the water supply measurements
- Inspecting and monitoring the treatment plant outfalls
- Preparing backup systems for redundant measurement capabilities

Appendix A

Period of Record Runoff Analysis

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3. The runoff from remaining areas of the Lake Michigan watershed was based primarily on stream gage records. This area includes the runoff from the northern and southeastern extents of the watershed. In these areas, a streamflow separation technique was used, in which estimated sanitary discharges upstream of a stream gage are subtracted from a

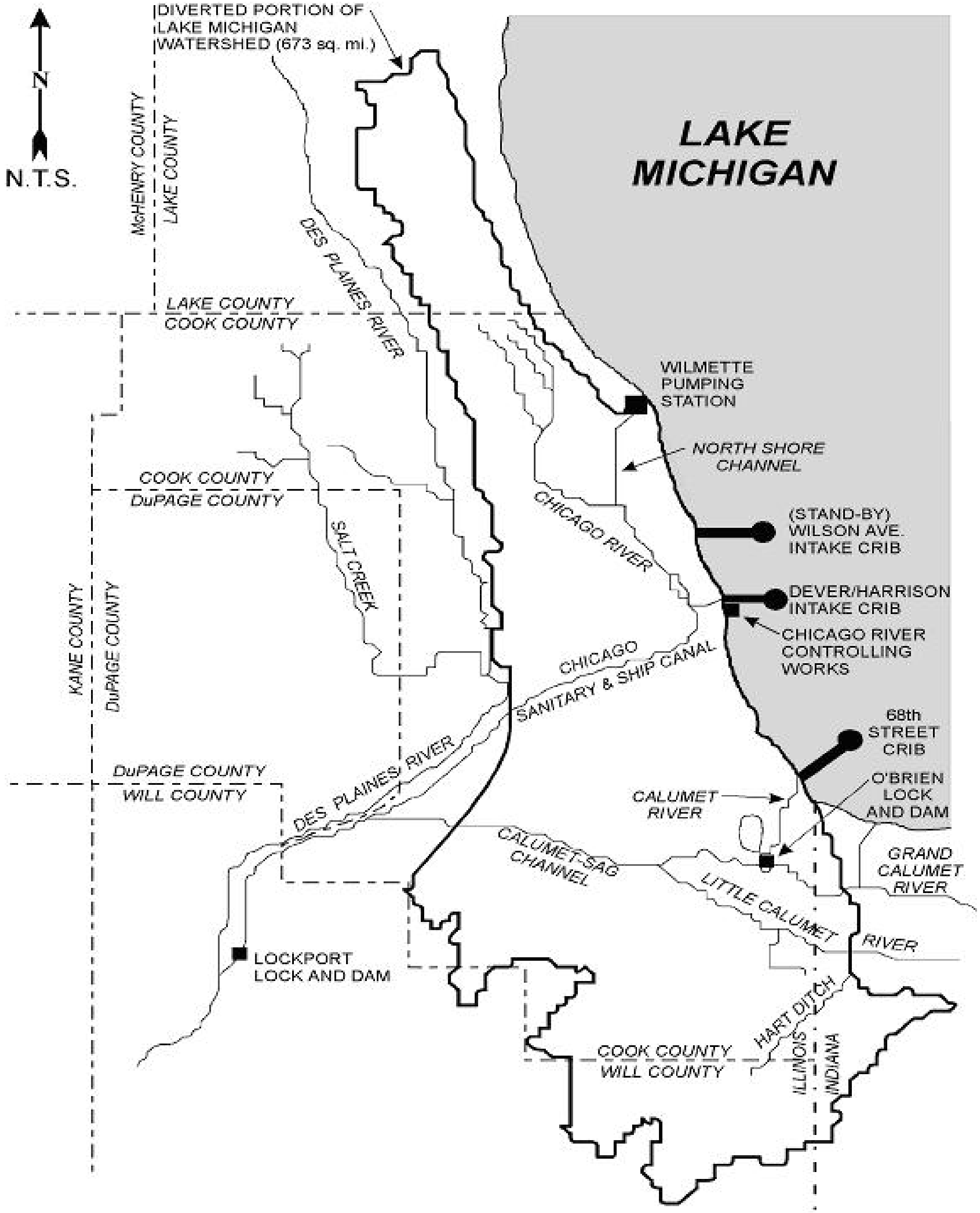
stream gage record to determine the portion of streamflow that is runoff. See the map on the next page for the diverted Lake Michigan watershed. The total simulated area is approximately 361 square miles, while the area for which the flows were calculated using a streamflow separation technique is approximately 312 square miles. Some areas overlap in that they fall within both the simulated area and the stream gaged area. These areas are separately sewered where the sanitary sewers convey flow to the water reclamation plants while the storm sewers discharge into streams to be measured by the gages. Overlapping areas were generally classified as gaged areas.

Simulated Watersheds

4. Areas that were simulated include the service areas of three Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) water reclamation facilities: West Southwest (Stickney), North Side, and Calumet, as well as the 84 square mile "ungaged" Calumet watershed (the sum of the first two runoff elements in paragraph 1). Based on the availability of precipitation, meteorological, and streamflow data, it was decided that the period of record could be determined for Water Year 1951 (WY51) through Water Year 1994 (WY94). The majority of the Lake Michigan Watershed runoff was simulated using the Hydrologic Simulation Program - FORTRAN (HSPF) as well as a hydraulic sewer routing model, the Special Contributing Area Loading Program (SCALP).

5. All simulations were based on models used in the computation of WY90 diversion. Prior to WY90, a 13-gage precipitation network was employed for diversion accounting, along with models based on approximately 1980 land use conditions. Beginning in WY90, the modeling incorporated a new 25-gage precipitation network implemented and maintained by the Illinois State Water Survey (ISWS) for the purpose of diversion accounting. Also incorporated into the WY90 accounting were revised models based on a detailed land use study from 1990 aerial photographs. Unfortunately, historic precipitation data from the 25-gage network is not available to fully realize the enhanced accuracy of the WY90 models. Therefore, it was determined that only "acceptable" precipitation data from the 13-gage network would be used and appropriately assigned to each of the twenty-five precipitation gages.

6. A majority of the thirteen precipitation gages used prior to WY90 required adjustments to their records. This adjustment was done by the ISWS through a rigorous procedure. However, adjusted data is not available prior to WY83. Upon review, only three of the thirteen gages were shown to contain data acceptable for use without the adjustments. Based on a double mass curve analysis by the ISWS, both the Midway Airport and University of Chicago precipitation gages were found to be acceptable on the basis of volume of precipitation measured and correlation between the measurements. For additional details, refer to the ISWS report "An Examination of Precipitation Patterns for Water Year 1984." These findings were again verified by the Northeastern Illinois Planning Commission (NIPC) in their December 1987 Memorandum to the State of Illinois, "Mean Annual Storm Runoff".



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7. The Chicago District decided to include the Midway Airport, University of Chicago and O'Hare Airport precipitation gages in the runoff study. These three gages, along with the Park Forest gage, were the only gages of the thirteen gages which required little or no adjustment by the ISWS for use in the diversion accounting. However, the Park Forest gage showed two distinct changes in slope from the double mass curves done by the ISWS. These two slope changes, one in 1959 and the other in 1977, represent catchment deficiencies at Park Forest, and therefore the gage was excluded from the analysis. Although the O'Hare Airport precipitation gage records only went back to 1 June 1962, it was nevertheless included in this study to provide a better representation of isohyetal distribution over a larger portion of the watershed. The O'Hare gage record prior to 1 June 1962 was synthesized using the program PRECIP developed by the Hydrologic Engineering Center of the U. S. Army Corps of Engineers. Both Midway and the University of Chicago precipitation gages were used as index gages for filling in O'Hare precipitation prior to 1 June 1962. See page 3 for a map of the location of rainfall gages.

8. In addition to precipitation data, HSPF also requires meteorological data in the modeling of the rainfall runoff process. Meteorological data used includes temperature, cloud cover, dew point, and wind. The four temperature gages used in HSPF modeling include O'Hare Airport, Midway Airport, University of Chicago, and Park Forest. Cloud cover, dew point and wind data, along with temperature data, are used as inputs to a separate program HSPFPREP (a program that prepares input data for HSPF) that computes solar radiation and potential evapotranspiration, two other variables used in HSPF modeling. O'Hare was used as the source of meteorological data from 1 November 1958 through 1994. Since meteorological data was not collected at O'Hare prior to 1 November 1958, Midway was used as the meteorological station for computing solar radiation and potential evapotranspiration from 1 October 1949 through 31 October 1958.

9. The HSPF models utilized in this study were those for the twenty-five precipitation gages used in diversion accounting for WY90. The twenty-five precipitation areas were based on Thiessen polygons for which the HSPF models were devised. For each of the twenty-five areas, HSPF was run for three distinct land types: impervious, grassland, and forest. Precipitation that was called for in HSPF was from the twenty-five gages. Since only three precipitation gages were ultimately used for this study, precipitation was apportioned to the twenty-five gages based on overlapping the three Thiessen polygons (O'Hare, Midway, University of Chicago) over the twenty-five polygons. In this way, percentages of precipitation from the three long-term gages were assigned to the twenty-five gage areas. HSPF was then run for each of the twenty-five areas for each of the three land types. Impervious runs computed only surface runoff (IMPRO) while grassland and forest areas computed both surface (OLFRO) and subsurface runoff (SUBRO).

10. The HSPF unit runoffs were used as input to the hydraulic sewer routing model, SCALP, which models the sewers contained in the service areas of the three MWRDGC water reclamation plants: West Southwest (Stickney), North Side, and Calumet. Flows

were generated for 137 sewer sub-basins known as Special Contributing Areas (SCAs). Only those SCAs within the Lake Michigan watershed were modeled. Both combined and separately sewered areas were modeled. Sanitary flow estimates are based on population equivalents within each SCA. Sewer inflow and infiltration for each SCA are based on impervious and pervious areas within each SCA. The impervious and pervious areas were determined through a thorough study of land uses within each SCA. Each land use has assumed percentages of impervious and pervious areas. Population equivalents and impervious and pervious areas for each SCA are based on the WY90 models.

11. Sanitary flows are computed by multiplying population equivalents by per capita sanitary estimates. Monthly, daily, and hourly multipliers are used to simulate changes in sanitary flow generation from month to month, day to day, and hour to hour. Sewer inflows are computed by multiplying impervious surface unit runoffs (IMPRO) and grassland surface unit runoffs (OLFRO) by the impervious and pervious areas falling within the polygon of the corresponding precipitation gage(s) for which the unit hydrographs were computed. Some SCAs fall within more than one of the twenty-five precipitation polygons. Sewer infiltration is computed by multiplying the grassland subsurface unit runoffs (SUBRO) by the pervious areas falling within the polygon of the corresponding precipitation gage. SCALP outputs both interceptor flows and overflows and keeps track of the three constituent flows: sanitary, inflow, and infiltration.

12. It was determined that the SCALP models used in this study, as well as those for WY90-92 diversion accounting analyses, did not fully account for the hydraulic connectivity of the impervious areas. As a result the percent impervious areas are somewhat overstated and should be adjusted downward to compensate for those impervious areas that are not hydraulically connected, (e.g. downspouts that discharge on lawns). The correct evaluation of hydraulically connected areas is difficult to ascertain, in that the calibration balances at the treatment plants can be reasonable for a wide range of values. This is the case because even if the simulated flows conveyed to the treatment plants via sewer interceptors are increased slightly, they do not have a significant impact on the treatment plant balances. Simulated to recorded influents to the plants have remained reasonable with respect to correlation coefficients as well as overall flow volumes. However, the frequency and volumes of simulated sewer overflows did increase significantly.

13. The WY89 diversion accounting model results were compared to the WY90 diversion accounting model results. The WY 90 model results over the period of WY90 to WY94 showed an increase of simulated overflows from 193 cfs to 249 cfs. While the WY89 models accounted for hydraulic connectivity of impervious areas, they required adjustment since they were based on approximately 1980 land use conditions. The revised models, while based on 1990 aerial photographs, did not fully account for the hydraulic connectivity of the impervious areas. Therefore, the most accurate representation of overflows falls somewhere between the results of both sets of models.

14. Accurate calibrations of overflows are difficult, if not impossible, to obtain. The process that is utilized to compensate for the difficult calibration is to adjust the simulated overflows downward. The adjustments were accomplished by multiplying the overflows from the WY90 model by 89%. This reduced the overflows from the WY90 model to the average of the overflows from WY89 and WY90 models. This resulted in a reduction of from 20 cfs to 25 cfs in the simulated overflows.

15. The ungaged Calumet watershed is an 84 square mile area that is separately simulated. The area is modeled separately because it consists of completely separate storm and sanitary sewers, and because it is downstream of all stream gages where streamflow separation techniques are used to determine runoff. Prior to WY90 this watershed was modeled with a 10 percent hydraulically connected impervious areas. Model revisions based on 1990 aerial photographs adjusted this value to 40 percent. This resulted in significant increases in simulated runoff from this area. However, as was discussed with respect to the overflow adjustments, the connectivity of the impervious areas was not correctly accounted for in the update to the percent imperviousness. This area in particular has experienced significant development over the last 25 years and as a result the imperviousness has also increased. However, since this area contains separate storm and sanitary sewers the overall connectivity of impervious areas is significantly less than that in combined areas (i.e. separate sewer areas have primarily disconnected roof downspouts). Therefore a sensitivity analysis was conducted for this watershed over the entire period of record, WY51-94. Runs were made for 10% and 40% imperviousness, reflecting the early and the adjusted models, and as additional run was made using 25% imperviousness (average percent imperviousness form the two other models). Total runoff from the ungaged Calumet watershed for the period of record for 10%, 25%, and 40 % imperviousness was 74.9 cfs, 91.2 cfs, and 107.4 cfs, respectively. For this runoff study the average between the two extremes, 25 % imperviousness, was used in the modeling of the ungaged Calumet watershed.

16. Although all simulations are based on WY90 modeling, some differences between estimated runoffs contained in the diversion reports and within this study can be expected due to variances between the WY90 modeling and the modeling used for this runoff study. The most obvious difference is in the precipitation data used. Diversion accounting utilized 13 precipitation gages prior to WY90 and 25 gages beginning in WY90. This runoff study utilizes only 3 of the original 13 gages. The following four modeling modifications were also made for this runoff study to more accurately reflect simulated runoff.

- The first revision involved correcting the double accounting of a portion of the runoff from the 84 square mile ungaged Calumet watershed. Although this does affect the runoff computation of Column 12 in the Accounting Reports, it does not impact the official computed diversion of Column 10.

- The second and third issues, described in the paragraphs above, concern corrections for the overstatement of imperviousness on the sewer overflows and for the runoff from the ungaged watershed.

- The forth modeling modification involves slight area corrections for two SCAs modeled in SCALP. All of these modifications have been incorporated into this study and will be utilized in the WY93 accounting analysis.

Gaged Watersheds

17. The streamflow runoff (the third element of diverted runoff) is not based on a simulation but, instead, on streamflow separation techniques. As noted above, a streamflow separation technique was used, in which sanitary discharges upstream of a streamgage (including both the sanitary portion of the sewage effluent from the treatment plants and the sanitary portion of the combined sewer overflows) are subtracted from a streamgage record to determine the portion of streamflow that is runoff. To maintain a consistency with the simulated runoff analysis, and as described in detail below, the separation technique was applied to gage records adjusted to 1990 land use conditions.

18. The two gages, and the associated sanitary discharges, where stream flow separation techniques were applied are as follows (see page 3 for location of gages):

- North Branch Chicago River at Niles, Illinois
  North Shore Sanitary District at Clavey Road, Deerfield Sanitary District, and sanitary flow component of 3 Combined Sewer Overflows (CSOs)

- Little Calumet River at South Holland, Illinois
  Thorn Creek Sanitary District, Homewood Sanitary District, Consumer Illinois at University Park, Schererville Sanitary District, Dyer Sanitary District, Consumer Illinois at Plum Creek, Consumer Illinois at Willowbrook, and sanitary flow component of 5 CSOs

19. The total gaged watershed runoff is simply the sum of the runoff from the North Branch Chicago River and the Little Calumet River at South Holland. It should be noted that the historic streamflow records were adjusted to account for the increases in precipitation due to increased urbanization over the period of record. This results in higher runoffs since the adjusted streamflows are now higher during the early portions of the period of record. All streamflow records were adjusted to reflect WY90 conditions by multiplying recorded streamflows by annual adjustment factors based on simulations of the 2-year and 50-year frequency events for the years 1950, 1976, and 2000 on the North Branch Chicago River and 1976 and 2000 on the Little Calumet River. The 2-year and 50-year events were selected because they represent an average and an extreme event. Refer to the two memoranda in attachment A-1 for a description of procedures used in simulating the 2-year and 50-year event runoff volumes.

20. The procedure used to adjust the gage records for the changes in imperviousness over time is as follows:

- Based on the total length of the simulated storm, the volumes for the 2-year and 50-year events were translated into average hourly flows, 144 hours on the North Branch Chicago River and 300 hours on the Little Calumet River (see the first and third tables following the memoranda).

- 2-year and 50-year frequency events for each year during the period of record (1951-1994) were interpolated from the base years, 1950, 1976, and 2000 on the North Branch Chicago River and 1976 and 2000 on the Little Calumet River (see the first and third tables following the memoranda).

- The daily flows for each day of the period of record from the North Branch Chicago River, and the South Holland gage of the Little Calumet River were then adjusted based on the interpolated frequency event. The adjustment was accomplished by multiplying each daily flow by a factor computed through a double interpolation consisting of the following steps: (1) given the year of the daily flow, select the corresponding 2-year and 50-year base flows; (2) if the value of the daily flow is less than or equal to the 2-year flow, then the factor is equal to the ratio of the 2-year flow for 1990 divided by the 2-year flow selected in first step; (3) if the value of the daily flow is greater than or equal to the 50-year flow, then the factor is equal to the ratio of the 50-year flow for 1990 divided by the 50-year flow selected in first step; (4) finally, if the value of the daily flow is between the 2-year and 50-year flows, then the factor is equal to an interpolated ratio of the factors as computed in steps two and three (the results are given in the second and fourth tables following the memoranda).

21. To compute the runoff portion of the streamflow for the Chicago and Little Calumet Rivers, all upstream sanitary discharges for 1990 plus overflows (computed and previously accounted for in the simulation analysis) must be subtracted from the recorded streamflows. The sanitary and overflows for each gage are:

- North Branch Chicago @ Niles = 19.5 cfs Sanitary + Overflows

- Little Calumet @ S. Holland = 23.1 cfs Sanitary + Overflows

The procedure used to perform the streamflow separation is to subtract the total of the sanitary flows and overflows from the daily adjusted record. If this difference of the flows is less than zero, the daily streamflow is set to zero. The results and comparison with unadjusted flows are given in the second and fourth tables and shown in the first two figures). The adjusted flow records, including those computed in the next section for the Grand Calumet River, are shown in the last table and last figure in attachment A-1.

22. The process outlined above results in setting all runoff responses for the individual streamgage watersheds to reflect WY90 land use conditions. Although changes in urbanization are accounted for through the use of the aforementioned adjustment factors, the effects of urbanization on actual precipitation amounts and distributions were not considered. It should also be noted that two differences exist in the streamflow

separation techniques employed in this study compared to those used in the certified reports. These differences would also result in differences between the computed runoff in Column 12 of the certified reports and those contained in this runoff study. The first difference involves the adjustment of streamflow records as previously discussed. Streamflow records prior to WY90 were increased while those after WY90 were decreased to reflect WY90 land use conditions. The second difference is that the two TARP (Tunnel and Reservoir Plan) systems were not considered. Therefore, CSOs from eight SCAs upstream of the stream gages were assumed to be completely measured by the respective stream gages instead of possibly flowing into one of the deep tunnels of TARP. Consequently, all sanitary portions of the eight CSOs were subtracted from the adjusted stream flow records in computing runoff.

Grand Calumet River

23. As with the other gaged areas, the runoff from the Grand Calumet River sub-area is determined by using a streamflow separation technique. The separation technique generally consists of obtaining the total flow at the downstream end of a sub-area from a stream gage record and subtracting the estimated sanitary discharges upstream of the gage. The sanitary discharges are not counted as part of the runoff because they are derived either from Lake Michigan water supply pumpage or from groundwater pumpage, in either case a non-runoff source.

24. The following paragraphs detail the procedures used, and associated difficulties, in determining the Grand Calumet River flow for the WY83-WY92 accounting reports. Following this, a revised methodology is described in which simulation models and gage records are used to compute the flows. This new methodology will be used to compute the flows both for the WY93 and subsequent accounting reports and as well as for this long-term runoff analysis. It should be noted that although the runoff from the Grand Calumet River affects the runoff component in the accounting report, it does not affect the diversion accounting. However, the sanitary discharges subtracted from the Grand Calumet gage record are pumpages of Lake Michigan water by the State of Indiana, not the State of Illinois, and are therefore a deduction from the Romeoville record.

WY83-WY92 Methodology

25. For the WY83-WY92 accounting procedures, the Grand Calumet River flows at the State Line were synthesized using a regression equation developed by Kiefer and Associates in 1978. The Grand Calumet River flows into the Cal-Sag Channel just south of the O'Brien Lock and Dam. The flow consists of runoff from a portion of the diverted Lake Michigan watershed and water supply for urban areas in Indiana. For the accounting process, Kiefer and Associates developed a regression equation linking the flow on the Grand Calumet River at Hohman Avenue in Hammond, Indiana, with the flow on Hart Ditch at Munster, and with the water surface elevation of Lake Michigan. The flows the regression equation is based on were from periodic USGS flow measurements of the Grand Calumet River. The regression equation is as follows:

- If Lake Michigan <= 1.0 ft Chicago City Datum
  Grand Calumet = 23 cfs

- If Lake Michigan > 1.0 CCD
  Grand Calumet = 29.9*Lake Michigan$^{(5/3)}$ + 0.13*Hart Ditch$^{(2/3)}$ - 9.6

26. The Lake Michigan water surface elevation for the regression was measured at Calumet Harbor (see the first figure in attachment A-2). Daily average elevation data were not available before 1970. For the period before 1970, daily average data were linearly interpolated from monthly average data. The monthly average was assumed to occur on the 15th of each month for the interpolation. The daily data for 1970 and after were downloaded from the National Oceanic and Atmospheric Administration (NOAA) Great Lakes Water Level Data Base. When the Calumet Harbor data was not available, the data for Milwaukee, Wisconsin and Holland, Michigan were used.

27. The rainfall runoff portion of the regression is based on Hart Ditch flows at Munster. Daily flows for Hart Ditch gage were retrieved from the USGS. Monthly average flows are shown in the second figure in attachment A-2.

28. As noted above, a streamflow separation technique was used, in which estimated sanitary discharges upstream of a streamgage are subtracted from a streamgage record to determine the portion of streamflow that is runoff. The only exception to this is at the Grand Calumet River gage where water supply is subtracted rather than sanitary outflow. The sanitary flow in the Grand Calumet flow is set as the total water supply to Whiting, Hammond and East Chicago, Indiana. These sewer systems were determined to discharge to the west branch of the Grand Calumet. The reported water supply pumpage is subtracted from the total flow to calculate runoff. If the water supply exceeds the total flow, the runoff is set to zero.

29. There are two significant difficulties associated with the Grand Calumet River streamflow record. The initial concern is with the complex hydraulics of this river which contains a shifting flow divide. The river is divided into two reaches, east and west, at the junction with the Indiana Harbor Canal. Backwater from Lake Michigan affects the flow in the Grand Calumet River because the Indiana Harbor Canal directly connects the river with the lake. West of the canal, a high point in the river bed profile causes a flow divide between the Hammond and East Chicago Sanitary Treatment Plants (STPs).

30. The computed Grand Calumet runoff is also in question due to the unavailability of water supply pumpage data. The Lake Michigan runoff in the Grand Calumet flow was calculated by subtracting the water supply pumpage for Whiting, East Chicago, and Hammond, Indiana from the simulated river flow. Historic records were not readily available, and the simulated period of record assumed the fixed WY89 pumpage of 74.7 cfs. This results in an underestimated runoff from the Grand Calumet River during early portions of the period of record, because it is unlikely that water supply was consistently similar. All Grand Calumet runoff flows below zero were set to zero. Using

these regressions and the runoff computation, the runoff was zero for 82.4% of the period of record.

WY93 and Beyond Methodology

31. The methodology for computing the runoff from the Grand Calumet watershed has been updated for this analysis to include both the data from a new streamflow gage at the state line and the analytic results of simulation models of the Grand Calumet River - Indiana Harbor and Canal basin. The runoff is calculated by subtracting the dry weather discharge from the Hammond and East Chicago Sewage Treatment Plants from the flow in the Grand Calumet River at Hohman Avenue.

32. The accuracy of the calculated stream flow runoff is dependent upon the accuracy of the individual streamgages and estimates of upstream sanitary discharges over the 44 year period of record, WY51-WY94. The Grand Calumet River gages where the stream flow separation techniques were applied, as well as the associated STPs, are as follows (see page 3 for location of gage and STPs):

- Grand Calumet River at Hohman Avenue (just east of the Illinois-Indiana State Line)

- East Chicago, Indiana and Hammond, Indiana STPs

33. A stream gage was installed on the Grand Calumet River at Hohman Avenue in WY92 (see the third figure in attachment A-2). A new regression equation was developed using measured flows for WY 92-94. The new equations are:

- If Lake Michigan <= 1.0 CCD
  Grand Calumet = 17.9*Lake Michigan + 0.038*Hart Ditch + 19.6

- If Lake Michigan > 1.0 CCD
  Grand Calumet = 65.4*Lake Michigan + 0.054*Hart Ditch - 29.6

- The minimum flow in the Grand Calumet is set to 12 cfs, the lowest observed flow.

34. Due to the unique hydraulics of the river, the amount of sanitary discharge flowing past the state line is a function of Lake Michigan water levels. To study the hydraulics of the river and canal, an unsteady state hydraulic model was developed. For a period of over nine years, hourly lake levels, STP discharges, major industrial discharges, and Calumet Sag Channel water levels were modeled. The results of the model aided in this examination of Grand Calumet runoff. The hydraulic model output included flow magnitude and direction for points just east and just west of both treatment plant outflows. The proportioning of the sanitary flow was based on trends observed at these cross sections.

35. An examination of the treatment plant records showed a minimum discharge at Hammond of 25 MGD or about 35 cfs. For East Chicago the minimum is 16 MGD or

25 cfs. The minimum discharge was chosen as having the minimum amount of inflow to the sewer system in proportion to the sanitary flow. This minimum discharge was considered to be the Lake Michigan water supply for the communities that are tributary to the Grand Calumet. The hydraulic model output included flow magnitude and direction for points just east and just west of both treatment plant outflows. The proportioning of the sanitary flow was based on trends observed at these cross sections. When the lake level is at or below 1.0 CCD, the proportion of Hammond discharge flowing west is fairly constant. The water supply flow west was set to the same fraction of the total flow as was observed in the model, 45%. Plotting model flow versus lake level for the cross section just east of Hammond treatment plant demonstrates how the lake forced water west at higher lake levels. In fourth and fifth figures in the attachment, the flow west is shown as positive. When the trend of the flow versus lake level crosses zero flow, all Hammond flow was headed west. This point was determined as 1.4 CCD. This method also determined the point where all East Chicago discharge flowed west as 1.8 CCD. Based on the linear nature of the flow versus lake level plots, linear interpolation was used in the equations (see the sixth figure in attachment A-2).

36. The revised Grand Calumet River regression equations are:

- If Lake Michigan <= 1.0 CCD
  Water Supply = 15.6 cfs

- If 1.0 CCD < Lake Michigan <= 1.4 CCD
  $$\text{Water Supply} = 15.6 + \frac{(\text{CCD} - 1.0)}{0.4} * 19.4 \text{ cfs}$$

- If 1.4 CCD < Lake Michigan <= 1.8 CCD
  $$\text{Water Supply} = 35 + \frac{(\text{CCD} - 1.4)}{0.4} * 25 \text{ cfs}$$

- If Lake Michigan > 1.8 CCD
  Water Supply = 60 cfs

37. As with Kiefer's method of determining the Grand Calumet runoff, this method sometimes produces water supply flows greater than the Grand Calumet flows. At these times, the runoff is set to zero. Also in common with Kiefer's equations, this method considers flows from Gary treatment plant and Gary USX steel plant (discharges to the East Grand Calumet) as runoff. As lake levels exceed 1.8 CCD (18% of runoff period), flows begin to cross from the East Grand Calumet to the West Grand Calumet.

38. The new set of regression equations produces significantly more runoff than Kiefer's equations (see the last three figures in attachment A-2). The new equations produce an average runoff of 24.4 cfs while the original equations produce an average runoff of 8.3 cfs, an increase of 194%. The total flow increased from 46.1 cfs to 52.4 cfs, a 14% increase. The major change in the flows was the 62% decrease in flow deducted from the total flow. The set value of 74.7 cfs was the total reported water supply pumpage for

Whiting, Hammond and East Chicago. This number does not account for the 81% of the runoff period when the East Chicago discharges flowed entirely to Lake Michigan or the 80% of the runoff period when less than half the Hammond discharges flowed west. The 74.7 cfs also includes consumptive use so it is an inflated value compared to the actual flow into the river. As a result of the high fixed water supply, the original runoff was set to zero for 83% of the period against 22% of the period for the new runoff.

Baseflow

39. The Chicago District and the Illinois Office of the United States Geological Survey undertook an analysis of the ground water discharge to the canals and watercourses, downstream of any gages, within the diverted watershed. The subject streams include portions of: the Chicago River, North Branch Chicago River, South Fork of the South Branch of the Chicago River, the Chicago Sanitary and Ship Canal, the Calumet River, the Grand Calumet River and the Cal-Sag Channel. The procedure for determining the discharge is given in attachment A-3. The total annual discharge is 4.0 cfs.

Results

40. The annual runoff from the diverted watershed is computed by summing the simulated flows (for the SCAs and the ungaged Calumet watershed), the gaged flows (from the North Branch Chicago, Little Calumet and Grand Calumet Rivers) plus the baseflow. The results of the WY51-WY94 continuous period simulation of the diverted Lake Michigan diversion accounting runoff is shown in the table A-1, on the next page, and the average annual runoff over the period of record is 785.2 cfs.

Table A.1 Lake Michigan Watershed Runoff

Water Year	Streamgage (cfs)	Simulated (cfs)	Baseflow (cfs)	Total (cfs)
1951	282.0	551.5	4.0	837.5
1952	356.5	460.7	4.0	821.2
1953	174.8	316.6	4.0	495.5
1954	187.1	416.8	4.0	607.9
1955	289.0	508.3	4.0	801.3
1956	158.6	290.7	4.0	453.2
1957	204.4	517.8	4.0	726.3
1958	162.5	339.0	4.0	505.5
1959	233.1	412.4	4.0	649.5
1960	305.2	454.6	4.0	763.8
1961	164.4	441.1	4.0	609.5
1962	226.4	390.7	4.0	621.1
1963	73.8	243.0	4.0	320.8
1964	92.7	275.0	4.0	371.7
1965	250.5	572.0	4.0	826.6
1966	273.9	534.6	4.0	812.4
1967	233.5	584.7	4.0	822.1
1968	224.0	449.2	4.0	677.2
1969	298.8	558.7	4.0	861.5
1970	289.2	658.7	4.0	951.8
1971	228.4	435.6	4.0	668.0
1972	345.5	557.5	4.0	907.0
1973	492.0	677.1	4.0	1173.0
1974	472.6	687.7	4.0	1164.3
1975	370.4	658.1	4.0	1032.4
1976	330.8	462.4	4.0	797.2
1977	137.9	387.6	4.0	529.6
1978	293.3	454.4	4.0	751.7
1979	340.6	600.7	4.0	945.3
1980	297.0	409.8	4.0	710.8
1981	346.4	488.7	4.0	839.1
1982	324.7	534.8	4.0	863.5
1983	422.2	875.4	4.0	1301.6
1984	353.0	605.4	4.0	962.4
1985	309.9	571.1	4.0	885.0
1986	376.5	595.4	4.0	975.9
1987	343.9	496.0	4.0	843.9
1988	215.9	351.8	4.0	571.7
1989	241.4	493.9	4.0	739.3
1990	277.4	524.4	4.0	805.8
1991	341.1	524.6	4.0	869.7
1992	235.9	476.0	4.0	715.9
1993	482.2	857.2	4.0	1343.4
1994	239.1	373.2	4.0	616.4
Average	279.5	501.7	4.0	785.2

Attachment A-1
Watershed Urbanization

CENCC-ED-HH

10 5 MAR 1996

MEMORANDUM FOR RECORD

SUBJECT: Analysis of Increases in Flood Volumes due to Urbanization for Portions of the Chicago Metropolitan Area

1. References:

a. North Branch Chicago River, Phase I General Design Memorandum, Appendix D.

b. Little Calumet River, Indiana, Phase I General Design Memorandum, Technical Volume E, July 1982.

2. The impacts on runoff volumes was assessed with adjustments to the loss rate parameters developed for HEC-1 models for various landuse conditions in models for the North Branch Chicago River and the Little Calumet River hydrologic models. Landuse for 1950, 1976 and year 2000 conditions were modeled for the North Branch Chicago River using TC's, R's and percent impervious. Landuse for 1976 and 2000 were modeled for the Little Calumet River using SCS Curve numbers and lag times. The hydrologic modeling consisted of HEC-1 models for the 2 year and 50 year synthetic events. Volume assessments were made at one location in each basin.

North Branch Chicago River, Illinois

3. HEC-1 models from the Phase II analysis were modified to reflect landuse for 1950, 1976 and 2000. The parameters were developed for the Phase I analysis, and are recreated in an attachment. It should be noted that the percent impervious used in the HEC-1 modeling was the equivalent of 30 percent of the percent impervious calculated during the development of the TC's and R's. The impervious factor in the hydrologic model is used to specify that maximum proportion of the subbasin that is direct runoff before loss rates are applied. Thus, the factor must be reduced to account for runoff from portions of the basin that are not directly connected. The reduction in the percent impervious was developed from historic calculations.

4. The runoff volume at Niles, the downstream end of the drainage area for the Main Stem and the West Fork, were compared for both the 2 year and the 50 year, for each of the different landuse conditions. The results of that analysis are presented in table 1. Frequency-volume and frequency-percent difference plots based on the analysis are attached.

SUBJECT: Analysis of Increases in flood Volumes due to Urbanization for Portions of the Chicago Metropolitan Area

Table 1 - North Branch Chicago River - Flood Runoff Volumes at Niles

Frequency Event	Runoff Volume(acre-feet)			Difference (%)	Difference (%)
	1950 Landuse	1976 Landuse	2000 Landuse	1950-1976	1976-2000
2 year	4,651	4,985	5,081	7.2	1.9
50 year	9,333	10,126	10,376	8.5	2.5

Little Calumet River, Indiana

5. Portions of the HEC-1 models for the Little Calumet River were modified to reflect 1976 and 2000 landuse conditions as developed for the Phase I General Design Memorandum. The landuse parameters used consisted of SCS curve numbers and lag time. A listing of those values is contained in the attachment..

6. The runoff volume analysis for the Little Calumet watershed is based on a hydrologic analysis of Hart Ditch runoff routed to the Illinois-Indiana stateline. The runoff volumes at the Illinois-Indiana stateline were compared for the 1976 and year 2000 landuse conditions, for the 2 and 50 year events. The results of the analysis are contained in table 2, below. Frequency-volume and frequency-percent difference plots based on the analysis are attached.

Table 2 - Little Calumet River Flood Runoff Volumes at the Stateline

Frequency Event	Runoff Volume (acre-feet)		Difference (%)
	1976 Landuse	2000 Landuse	1976-2000
2 year	3,229	3,433	6.3
50 year	8,245	8,650	4.9

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SUBJECT: Analysis of Increases in flood Volumes due to Urbanization for
Portions of the Chicago Metropolitan Area

7. The runoff volumes for the Little Calumet basin were based on runoff changes reflective of changes in curve numbers and impervious changes.

8. Point of contact for this memorandum is the undersigned.

Susanne J. Davis

Encls ,

Susanne J. Davis, P.E.
Hydraulic Engineer, Hydraulic Engineering Section
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Attachment 1 - Table 1 - Landuse Parameters for North Branch Chicago River, Phase I GDM

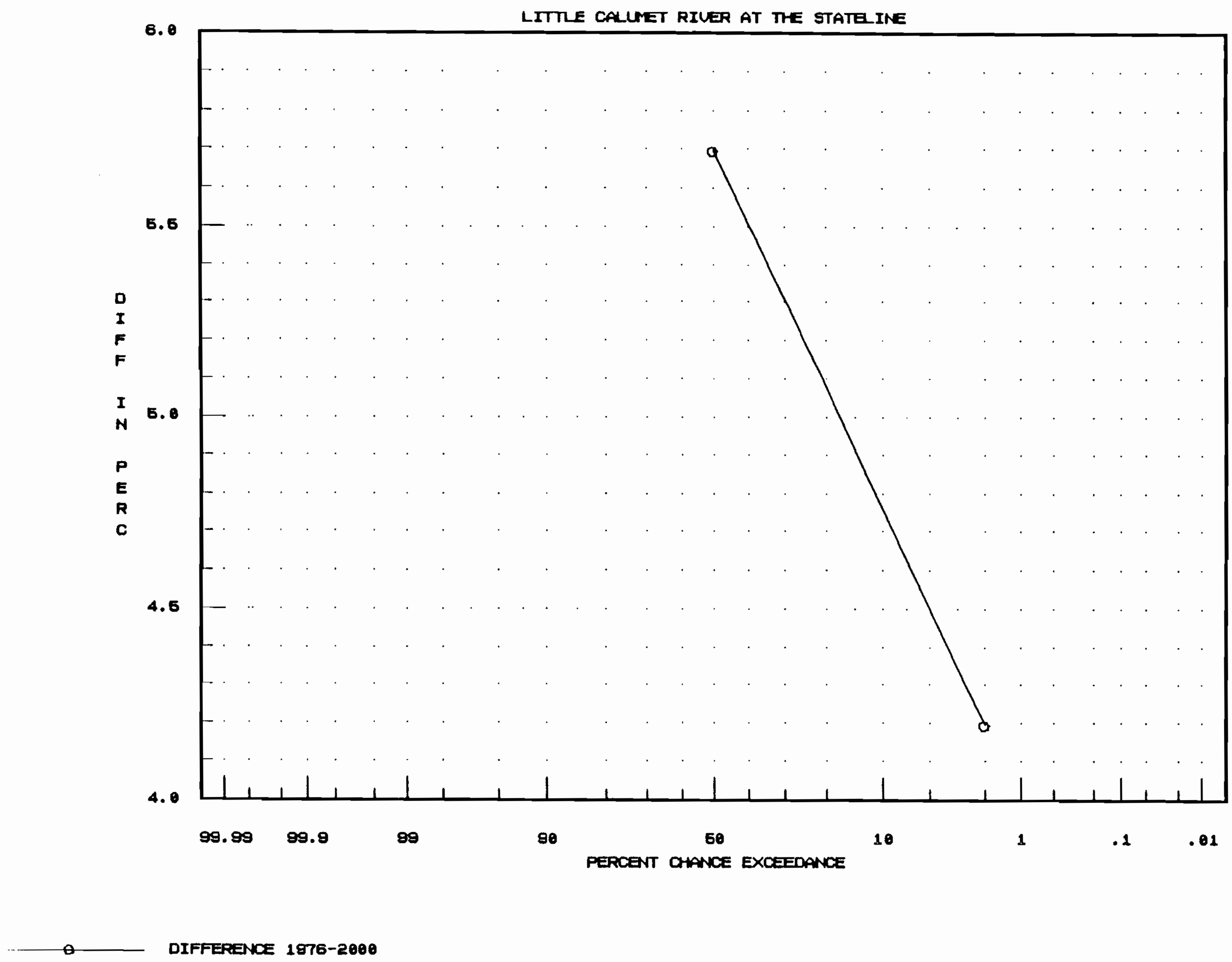
	Hydrologic Parameters								
	1950 Landuse Conditions			1976 Landuse Conditions			2000 Landuse Conditions		
Drainage Area[1]	% Impervious[2]	TC	R	% Impervious[2]	TC	R	% Impervious[2]	TC	R
1 (10)	3	4.00	9.24	8	3.20	7.43	9	3.09	7.18
2 (12)	4.2	3.22	8.09	7.5	2.80	7.06	8	2.73	6.89
3 (14)	5.1	2.55	6.55	9	2.19	5.63	10.5	2.11	5.44
4 (16)	6.9	1.63	5.64	12	1.40	4:85	13	1.37	4.74
5 (18)	6.6	3.69	8.22	9	3.36	7.51	10	3.31	7.39
6 (20	6.6	0.36	3.17	9	0.33	2.88	11	0.31	2.75
7A (221)	10.2	3.39	7.40	11.5	3.27	7.14	12	3.25	7.10
7B (222)	10.2	3.39	7.40	11.5	3.27	7.14	12	3.25	7.10
8 (24)	9.9	2.86	6.65	12.5	2.67	6.20	13	2.66	6.19
9 (30)	3.3	1.00	8.41	5.5	1.00	7.55	6	1.00	7.39
10 (32)	3.6	1.00	5.08	7	1.00	4.34	9	1.00	4.09
11 (34)	6.0	1.00	4.64	7	1.00	4.48	8.5	1.00	4.23
12 (36)	3.6	1.00	5.78	5	1.00	5.46	5.5	1.00	5.30
13 (38)	3.6	1.00	8.27	5	1.00	7.77	6	1.00	7.46
14 (40)	5.7	1.00	5.63	10	1.00	4.92	11.5	1.00	4.79
15 (42)	6.6	1.47	4.40	11.5	1.26	3.78	13	1.22	3.67
16 (44)	5.4	1.94	5.37	9	1.69	4.69	11.5	1.59	4.41
17 (46)	6.0	3.36	7.05	11	2.86	6.02	12.5	2.74	5.77
18 (48)	5.4	0.92	4.20	11	0.76	3.50	11.5	0.75	3.46
19 (50)	6.9	3.74	7.66	13	3.11	6.40	13.5	3.10	6.38
20 (60)	3.3	2.99	7.09	5.5	2.68	6.37	6.5	2.57	6.11
21 (62)	4.5	1.57	8.75	8.5	1.34	7.50	10	1.28	7.16
22 (64)	4.8	2.54	9.54	12	1.99	7.50	13.5	1.93	7.29
23 (66)	4.5	2.25	8.65	11.5	1.76	6.80	13	1.71	6.61
24 (68)	5.7	3.86	8.17	11.5	3.20	6.80	14.5	2.98	6.32
25 (70)	4.8	4.83	16.98	12	3.80	13.40	15	3.57	12.60
26 (72)	4.2	4.12	14.65	10	3.30	1180	15.5	2. 91	10.41
27 (74)	5.1	4.94	17.29	8	4.40	15.40	8.5	4. 33	15.18
28 (76)	5.1	5.04	18.11	11	4. 10	14.80	12	4.00	14.42
29 (78)	5.4	3.00	6.96	13	2.38	5.53	13.5	2.35	5.47
30 (80)	5.1	5.32[3]	10.88[3]	15	3.95	8.12	15.5	3.92	8.04

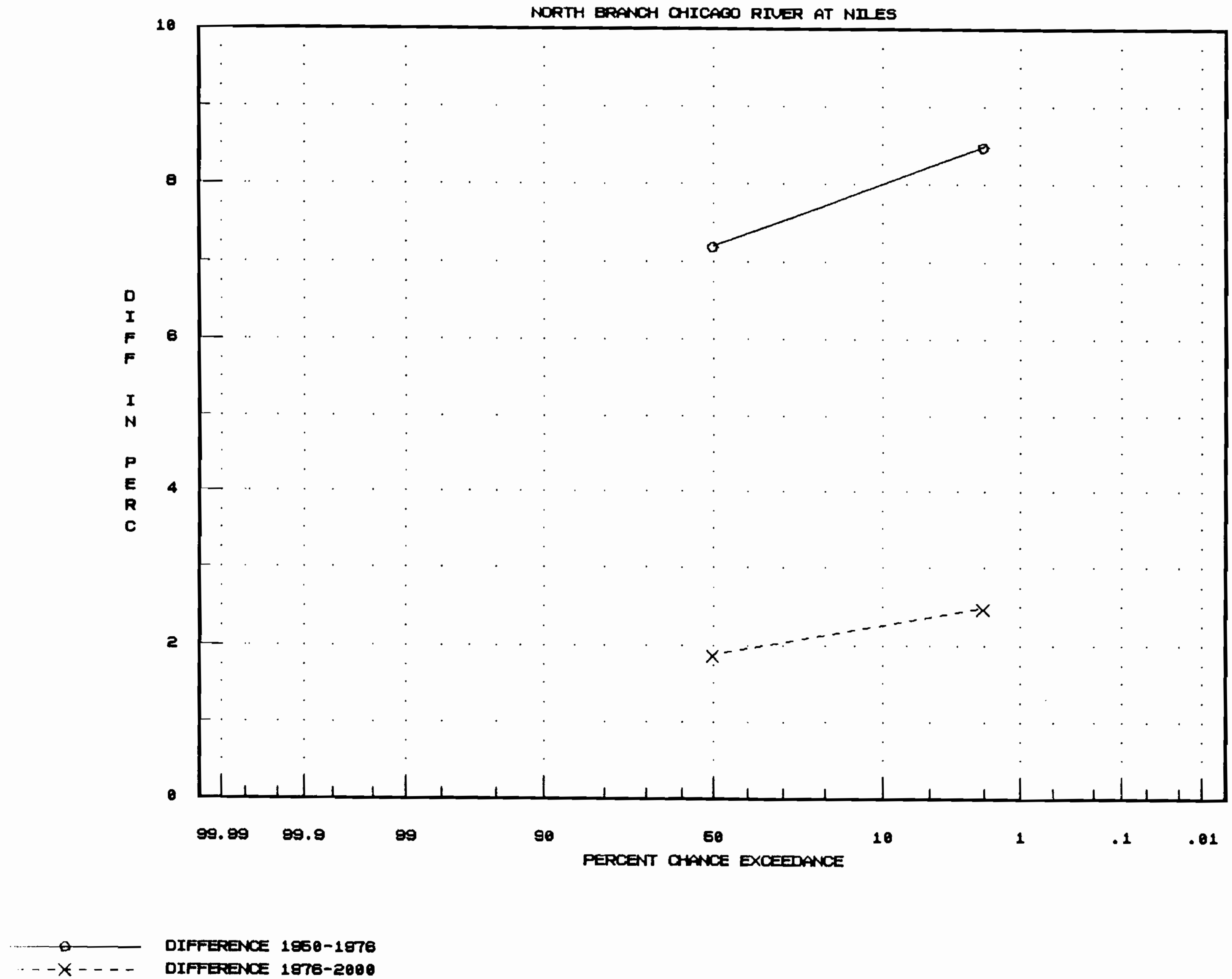
1 The drainage area designation was changed after the Phase I analysis. The numbers in parantheses are the current basin identifiers.
2 The percent impervious values are 30 % of the values contained in the original table in the Phase I report.
3 This subbasin was divided into 4 for the current modeling. New TC's and R's were developed for the 1950 landuse using the regression equations in the the Phase I report. New TC's and R's were not developed for 1976 and 2000 conditions, because the change in TC's and R's were minimal.

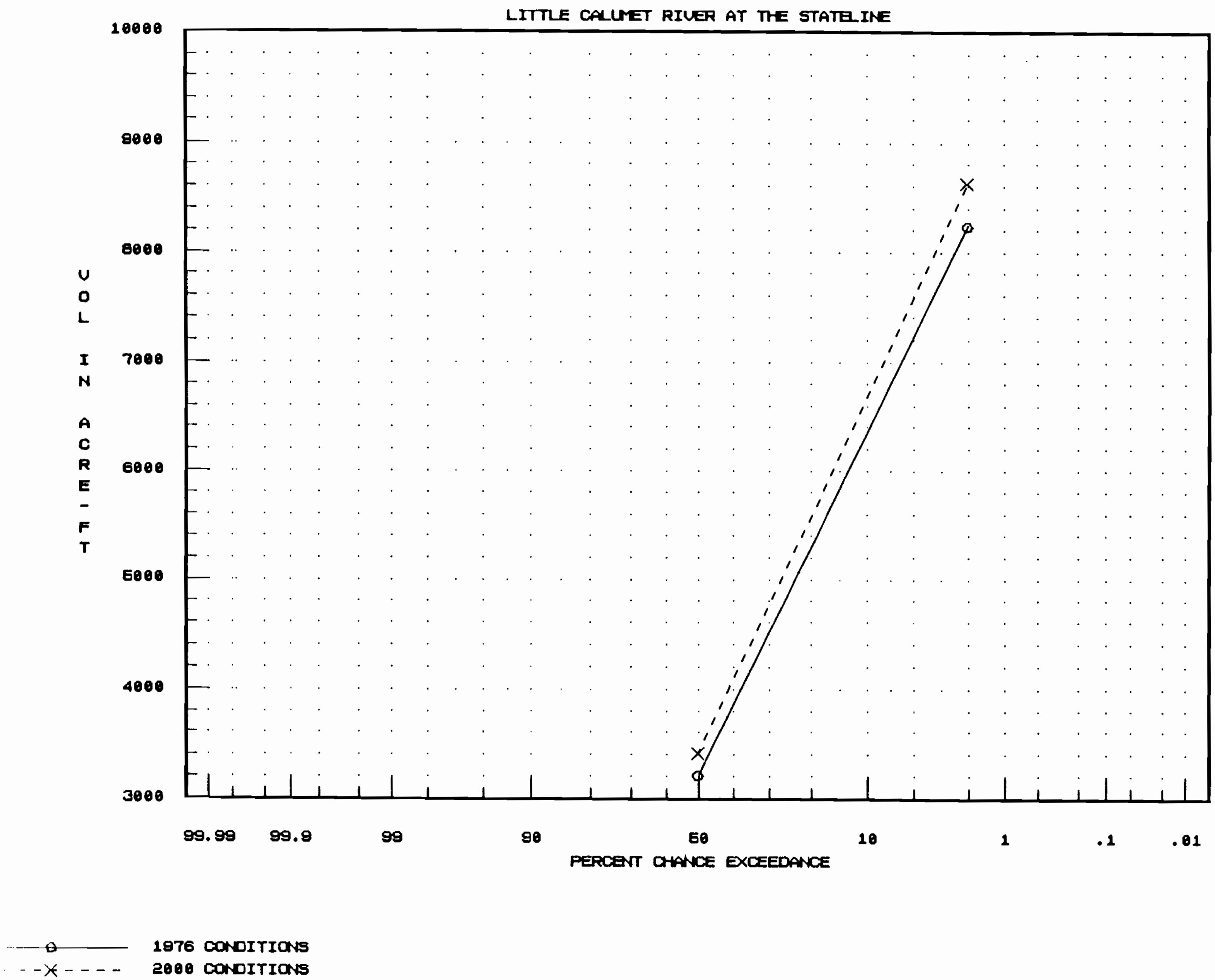
Attachment 1 - Table 2 - Landuse Parameters - Little Calumet River, Phase I GDM

Subarea Number	Drainage Area (sq.miles)	1976 Landuse		2000 Landuse	
		Curve Number	Subarea Lag (hours)	Curve Number	Subarea Lag (hours)
01	36.4	81	26.5	81	26.4
02	2.19	86	4.0	90.5	3.4
03	9.00	82	20.0	84	18.6
04	6.82	88	21.0	88.5	20.6
05	0.44	92	2.1	93	2.0
06[1]	15.82	83	22.0	85	20.7
8-1	1.30	80.9	4.07	83	4.07
8-2	0.29	83	0.87	85	0.87
8-3	1.06	80.9	2.98	83	2.98
8-4	0.74	80.9	3.64	83	3.64
8-5	0.80	80.9	3.40	83	3.40
8-6	0.50	80.9	3.90	83	3.90
8-7	0.38	80.9	1.57	83	1.57
10-1	0.16	80	1.58	80	1.58
10-2	0.49	69	1.70	69	1.70
10-3	1.21	86	2.29	86	2.29
12-3	1.04	82	1.70	82	1.70
12-4	1.10	80	3.14	80	3.14
12-2	0.95	73	2.36	73	2.36
12-1	1.14	73	3.60	73	3.60

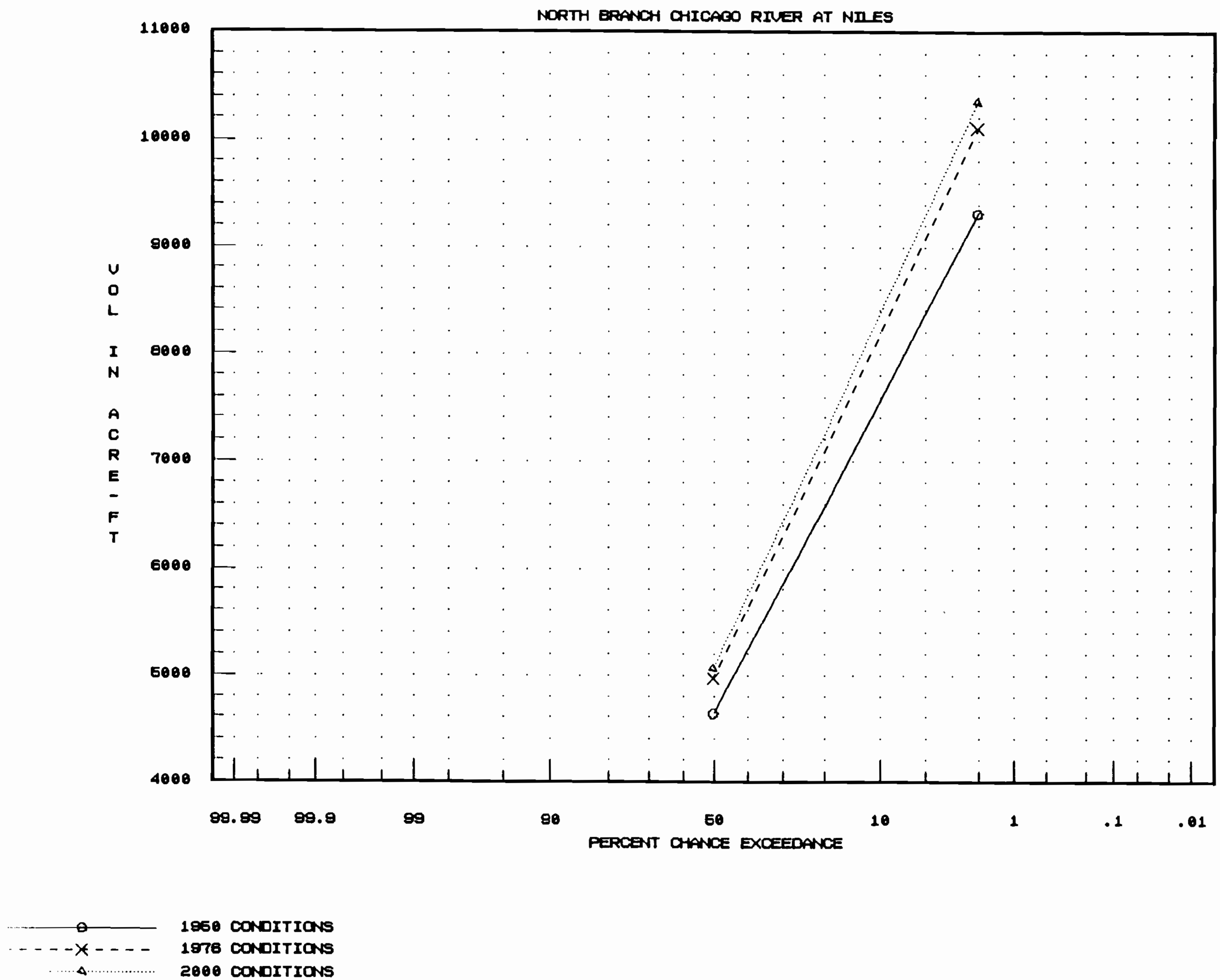
1 Subarea 06-Cady Marsh Ditch-is subdivided in the HEC-1 modeling to reflect details developed after the original hydrologic models were assembled. Adjustments to the individual lag times and curve numbers were made based on the percentage difference between 1976 and 2000 conditions for subarea 06.


LITTLE CALUMET RIVER AT THE STATELINE
6.0
5.5
5.0
4.5
4.0
DIFF IN PERC
99.99
99.9
99
90
50
10
1
.1
.01
PERCENT CHANCE EXCEEDANCE
DIFFERENCE 1976-2000


NORTH BRANCH CHICAGO RIVER AT NILES
DIFF IN PERC
10
8
6
4
2
0
99.99
99.9
99
90
50
10
1
.1
.01
PERCENT CHANCE EXCEEDANCE
DIFFERENCE 1950-1976
DIFFERENCE 1976-2000



LITTLE CALUMET RIVER AT THE STATELINE
10000
9000
8000
7000
6000
5000
4000
3000
VOL IN ACRE-FT
99.99
99.9
99
90
50
10
1
.1
.01
PERCENT CHANCE EXCEEDANCE
1976 CONDITIONS
2000 CONDITIONS


NORTH BRANCH CHICAGO RIVER AT NILES
VOL IN ACRE-FT
11000
10000
9000
8000
7000
6000
5000
4000
99.99
99.9
99
90
50
10
1
.1
.01
PERCENT CHANCE EXCEEDANCE
1950 CONDITIONS
1976 CONDITIONS
2000 CONDITIONS

0 6 MAY 199_

CENCC-ED-HH

MEMORANDUM FOR RECORD

SUBJECT: Analysis of Increases in Flood Volumes due to Urbanization for Portions of the Chicago Metropolitan Area

1. References:
   a. North Branch Chicago River, Phase I General Design Memorandum, Appendix D.
   b. Little Calumet River, Indiana, Phase I General Design Memorandum, Technical Volume E, July 1982.
   c. Memorandum for Record dated 5 March 1996, SAB.

2. Twenty-four hour and full event volumes were developed for the North Branch Chicago River based on a full event, using an extended memory version of HEC-1. Additional hydrograph ordinates were added in order to capture the full event. The event duration is 144 hours (6 days).

Table 1 - North Branch Chicago River - 24-Hour Flood Runoff Volumes at Niles

Frequency Event	Runoff Volume(acre-feet)			Difference (%)	Difference (%)
	1950 Landuse	1976 Landuse	2000 Landuse	1950-1976	1976-2000
2 year	2,560	2,810	2,884	9.8	2.6
50 year	5,071	5,497	5,626	8.4	2.3

Table 2 - North Branch Chicago River - Flood Runoff Volumes at Niles[1]

Frequency Event	Runoff Volume(acre-feet)			Difference (%)	Difference (%)
	1950 Landuse	1976 Landuse	2000 Landuse	1950-1976	1976-2000
2 year	6,448	6,922	7,073	7.4	2.2
50 year	13,586	14,729	15,133	8.4	2.7

1 Based on a six day event (144 hours)

3. Twenty-four hour and full event volumes were developed for the Little Calumet River at the Illinois-Indiana Stateline and at South Holland. Volumes for South Holland are available only for year 2000 landuse. (The same loss rates, year 2000, were used for the Illinois portion of the watershed for both the 1976 and 2000 landuse models.) The Little Calumet River hydrology models were run for 300 hours (at a one hour time step), which captures the full event.

Table 3 - Little Calumet River- 24-Hour Flood Runoff Volumes at Stateline and South Holland

	Stateline			South Holland
Frequency Event	Runoff Volume (acre-feet)		Difference (%)	Runoff Volume (acre-feet)
	1976 Landuse	2000 Landuse	1976-2000	2000 Landuse
2 year	1,139	1,232	8.2	4,272
50 year	3,178	3,321	4.5	5,929

Table 4 - Little Calumet River- Flood Runoff Volumes at Stateline and South Holland

	Stateline			South Holland
Frequency Event	Runoff Volume (acre-feet)		Difference (%)	Runoff Volume (acre-feet)
	1976 Landuse	2000 Landuse	1976-2000	2000 Landuse
2 year	3,229	3,433	6.3	12,012
50 year	8,245	8,650	4.9	23,032

4. Point of contact for this memorandum is the undersigned.

Encls

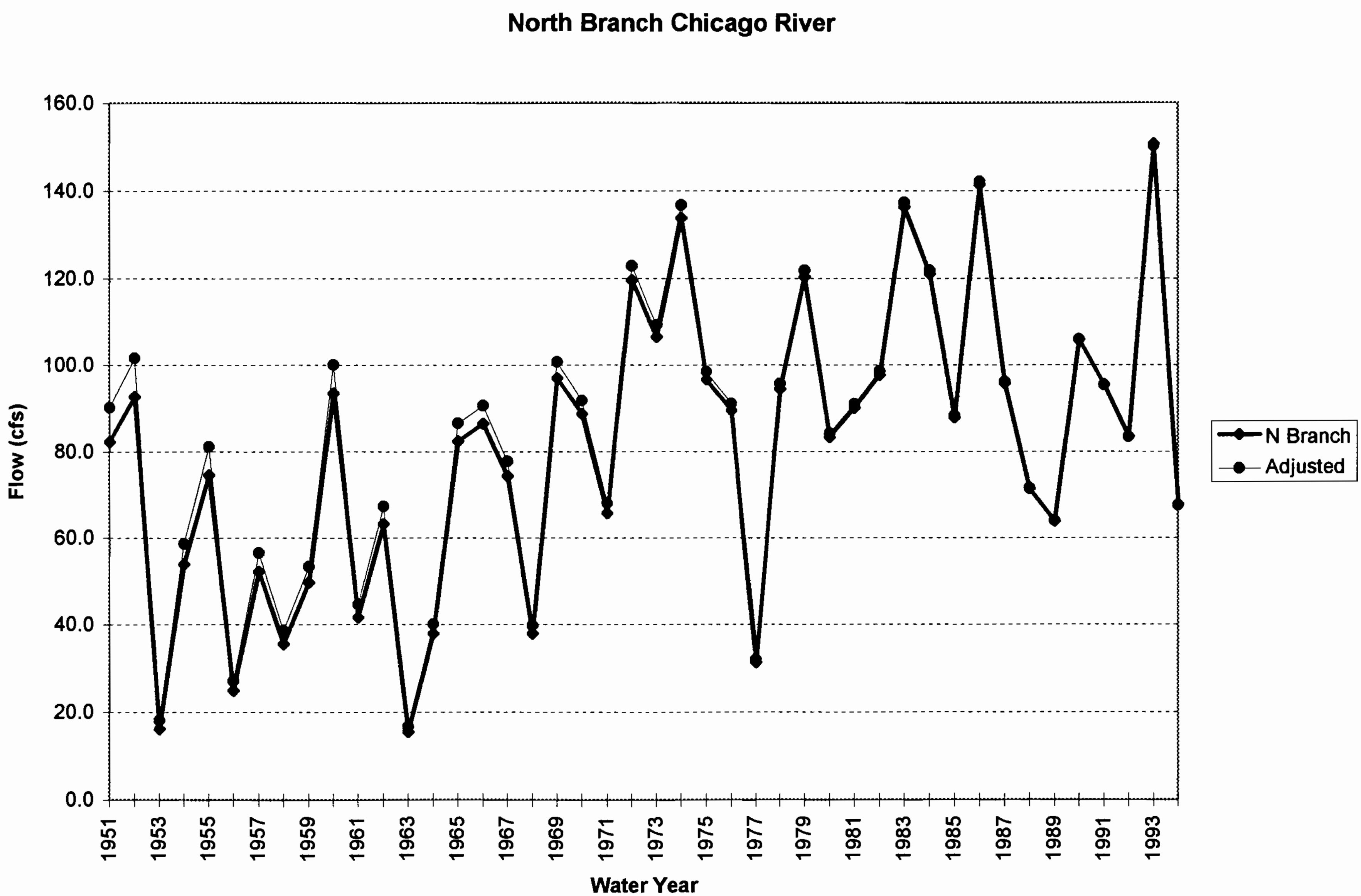
Susanne J. Davis, P.E.
Hydraulic Engineer, Hydraulic Engineering Section
Hydraulic & Environmental Engineering Branch

North Branch Chicago River			
Volumes (acre-feet)			
Landuse	1950	1976	2000
2 year	6448	6922	7073
50 year	13586	14729	15133

North Branch Chicago River			
Volumes (avg cfs/144 hours)			
Landuse	1950	1976	2000
2 year	542	582	594
50 year	1142	1238	1272
Conversion Factor:		0.08	

North Branch Chicago River		
Adjusted Frequency Flows		
Year	2-Yr Flow	50-Yr Flow
1950	542	1142
1951	543	1145
1952	545	1149
1953	546	1153
1954	548	1156
1955	549	1160
1956	551	1164
1957	553	1167
1958	554	1171
1959	556	1175
1960	557	1179
1961	559	1182
1962	560	1186
1963	562	1190
1964	563	1193
1965	565	1197
1966	566	1201
1967	568	1204
1968	569	1208
1969	571	1212
1970	572	1215
1971	574	1219
1972	576	1223
1973	577	1227
1974	579	1230
1975	580	1234
1976	582	1238
1977	582	1239
1978	583	1240
1979	583	1242
1980	584	1243
1981	584	1245
1982	585	1246
1983	585	1248
1984	586	1249
1985	586	1250
1986	587	1252
1987	587	1253
1988	588	1255
1989	589	1256
1990	589	1257
1991	590	1259
1992	590	1260
1993	591	1262
1994	591	1263
1995	592	1265
1996	592	1266
1997	593	1267
1998	593	1269
1999	594	1270
2000	594	1272

North Branch Chicago River						
Total Yearly Flows						
Water Year	NB Gage	Adj Gage	Overflow	Sanitary	NB Total	Adj Total
1951	96.9	105.2	0.3	19.5	82.2	90.2
1952	109.8	118.9	0.3	19.5	92.7	101.6
1953	28.4	30.6	0.2	19.5	16.1	18.0
1954	67.1	72.2	0.2	19.5	53.9	58.7
1955	90.3	97.0	0.3	19.5	74.6	81.0
1956	36.6	39.2	0.2	19.5	24.8	27.0
1957	67.1	71.6	0.4	19.5	52.2	56.4
1958	53.0	56.4	0.3	19.5	35.4	38.6
1959	65.8	69.7	0.3	19.5	49.6	53.3
1960	112.1	118.7	0.4	19.5	93.5	100.0
1961	57.0	60.1	0.3	19.5	41.7	44.5
1962	80.9	85.2	0.3	19.5	63.1	67.3
1963	28.1	29.4	0.2	19.5	15.5	16.5
1964	53.7	56.2	0.2	19.5	37.9	40.0
1965	100.6	105.0	0.5	19.5	82.3	86.5
1966	104.6	108.9	0.4	19.5	86.4	90.6
1967	92.5	96.1	0.4	19.5	74.3	77.7
1968	56.7	58.7	0.3	19.5	37.9	39.7
1969	116.5	120.3	0.4	19.5	97.0	100.7
1970	108.4	111.6	0.5	19.5	88.6	91.8
1971	85.2	87.6	0.3	19.5	65.7	67.9
1972	139.0	142.3	0.5	19.5	119.5	122.8
1973	126.4	129.2	0.5	19.5	106.5	109.2
1974	153.9	156.8	0.6	19.5	133.8	136.7
1975	116.6	118.4	0.5	19.5	96.6	98.4
1976	109.3	110.8	0.3	19.5	89.5	91.0
1977	50.4	51.0	0.2	19.5	31.3	31.9
1978	114.3	115.6	0.3	19.5	94.5	95.8
1979	140.4	141.8	0.5	19.5	120.4	121.8
1980	103.0	104.0	0.4	19.5	83.2	84.2
1981	110.0	110.9	0.4	19.5	90.1	91.0
1982	117.6	118.5	0.4	19.5	97.7	98.6
1983	156.4	157.5	0.7	19.5	136.3	137.3
1984	140.9	141.7	0.5	19.5	121.0	121.8
1985	107.6	108.1	0.4	19.5	87.7	88.2
1986	161.4	162.0	0.4	19.5	141.4	142.1
1987	115.6	115.9	0.4	19.5	95.7	96.1
1988	91.1	91.2	0.2	19.5	71.3	71.5
1989	83.7	83.8	0.3	19.5	63.9	64.0
1990	125.8	125.8	0.4	19.5	105.9	106.0
1991	115.3	115.2	0.4	19.5	95.4	95.4
1992	103.3	103.2	0.4	19.5	83.4	83.3
1993	170.7	170.3	0.7	19.5	150.7	150.3
1994	86.7	86.3	0.3	19.5	67.7	67.4
Average:	98.9	101.3	0.4	19.5	80.7	83.0

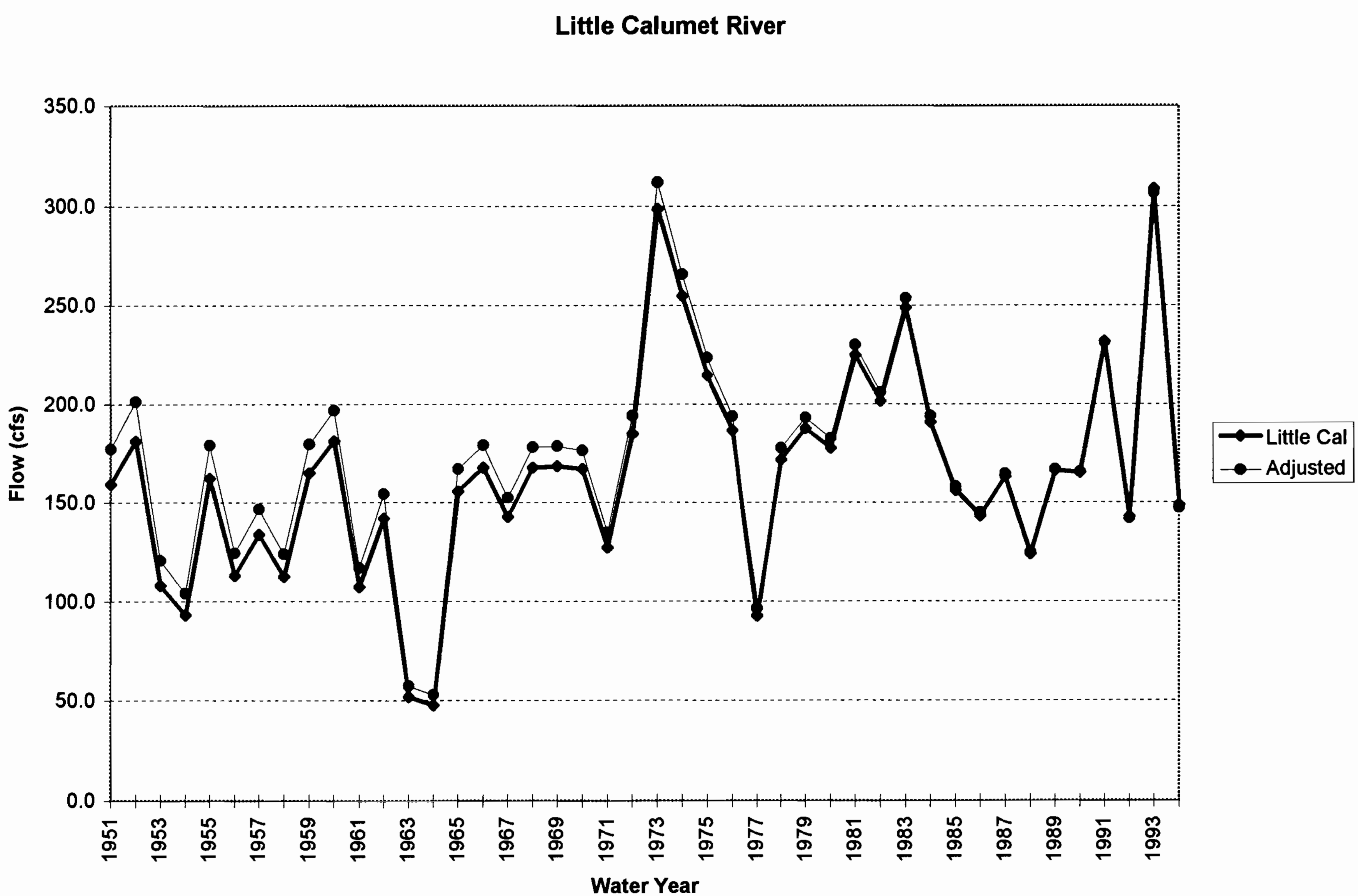

North Branch Chicago River
160.0
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
Flow (cfs)
1951
1953
1955
1957
1959
1961
1963
1965
1967
1969
1971
1973
1975
1977
1979
1981
1983
1985
1987
1989
1991
1993
Water Year
N Branch
Adjusted

Little Calumet River Volumes (acre-feet)		
Landuse	1976	2000
2 year	11298	12012
50 year	21954	23032
2 year ratio:		0.94
50 year ratio:		0.95

Little Calumet River Volumes (avg cfs/300 hrs)		
Landuse	1976	2000
2 year	456	484
50 year	886	929
Conversion Factor:		0.04

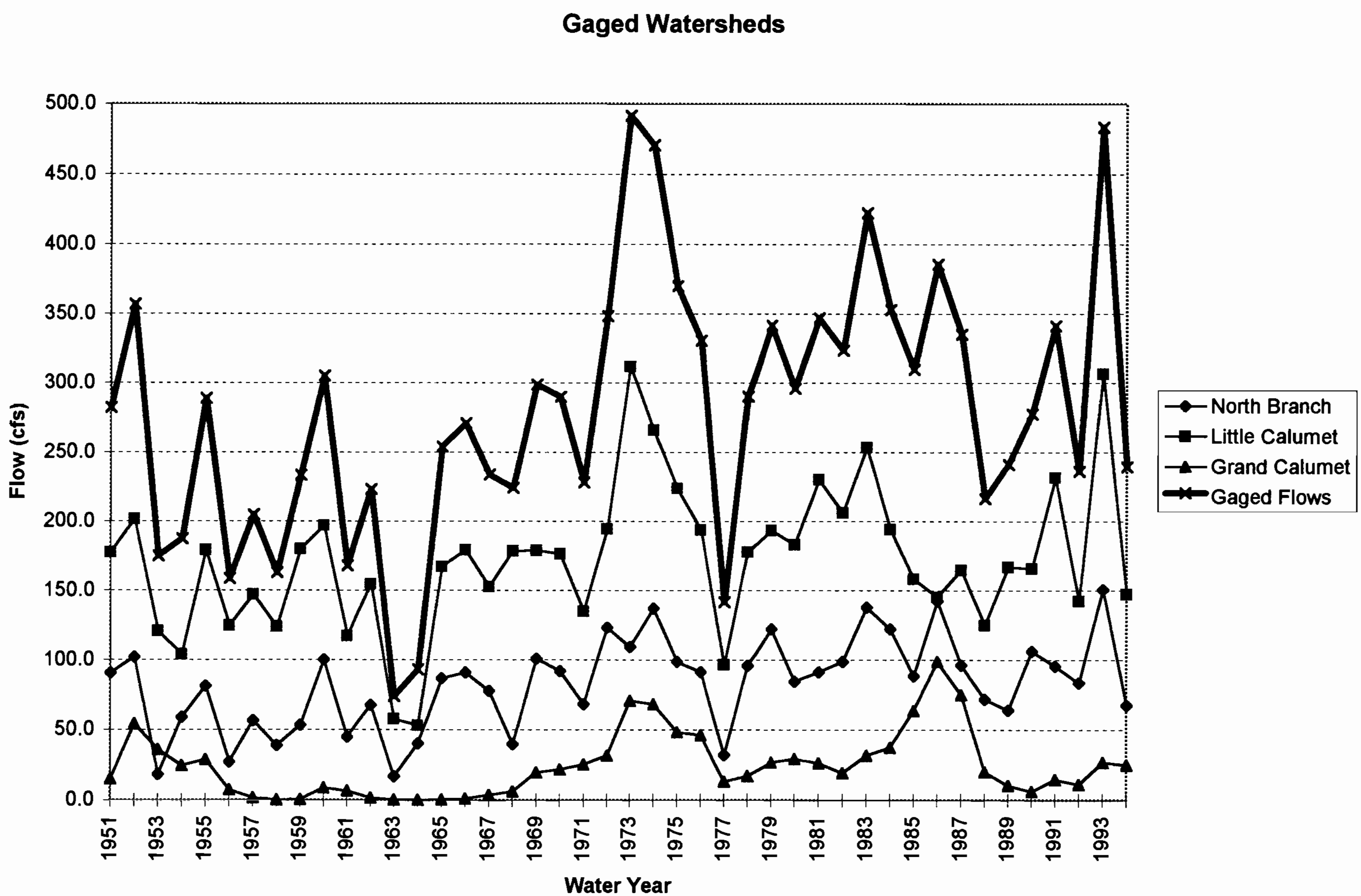
Little Calumet River Adjusted Frequency Flows		
Year	2-Yr Flow	50-Yr Flow
1950	425	839
1951	426	840
1952	427	842
1953	428	844
1954	429	846
1955	431	848
1956	432	849
1957	433	851
1958	434	853
1959	435	855
1960	437	857
1961	438	858
1962	439	860
1963	440	862
1964	441	864
1965	443	866
1966	444	867
1967	445	869
1968	446	871
1969	447	873
1970	449	875
1971	450	877
1972	451	878
1973	452	880
1974	453	882
1975	455	884
1976	456	886
1977	457	887
1978	458	889
1979	459	891
1980	461	893
1981	462	895
1982	463	896
1983	464	898
1984	465	900
1985	467	902
1986	468	904
1987	469	905
1988	470	907
1989	471	909
1990	473	911
1991	474	913
1992	475	914
1993	476	916
1994	477	918
1995	479	920
1996	480	922
1997	481	924
1998	482	925
1999	483	927
2000	484	929

Little Calumet River							
Total Yearly Flows							
Water Year	LC Gage	Adj Gage	SH Over	Mun San	Thom San	LC Total	Adj Total
1951	183.5	201.5	1.7	3.9	19.2	159.2	177.0
1952	205.8	226.0	1.7	3.9	19.2	181.0	201.2
1953	132.6	145.4	1.6	3.9	19.2	107.9	120.6
1954	117.6	128.6	1.7	3.9	19.2	93.0	104.0
1955	187.0	203.9	1.7	3.9	19.2	162.2	179.1
1956	137.1	148.8	1.5	3.9	19.2	112.7	124.4
1957	158.2	171.2	1.7	3.9	19.2	133.8	146.6
1958	137.0	148.5	1.6	3.9	19.2	112.3	123.8
1959	189.8	204.3	1.6	3.9	19.2	165.0	179.6
1960	206.0	221.5	1.7	3.9	19.2	181.1	196.7
1961	132.0	141.7	1.7	3.9	19.2	107.2	116.9
1962	166.7	179.1	1.7	3.9	19.2	141.9	154.3
1963	76.3	81.8	1.5	3.9	19.2	51.8	57.2
1964	72.4	77.4	1.6	3.9	19.2	47.7	52.7
1965	180.6	191.9	1.9	3.9	19.2	155.6	166.9
1966	192.5	203.9	1.8	3.9	19.2	167.7	179.0
1967	167.5	177.3	1.9	3.9	19.2	142.6	152.3
1968	192.5	203.0	1.7	3.9	19.2	167.7	178.1
1969	193.3	203.6	1.8	3.9	19.2	168.4	178.7
1970	191.9	201.3	1.9	3.9	19.2	166.9	176.3
1971	151.9	159.4	1.7	3.9	19.2	127.1	134.6
1972	209.6	218.9	1.8	3.9	19.2	184.7	194.0
1973	323.4	337.0	2.0	3.9	19.2	298.4	311.9
1974	280.0	290.7	2.0	3.9	19.2	254.9	265.6
1975	239.7	248.6	2.0	3.9	19.2	214.6	223.4
1976	211.3	218.4	1.7	3.9	19.2	186.5	193.6
1977	117.2	121.1	1.6	3.9	19.2	92.5	96.4
1978	196.5	202.3	1.7	3.9	19.2	171.7	177.5
1979	212.5	218.1	1.9	3.9	19.2	187.5	193.1
1980	202.4	207.4	1.6	3.9	19.2	177.7	182.7
1981	249.5	254.8	1.7	3.9	19.2	224.6	230.0
1982	226.4	230.7	1.8	3.9	19.2	201.5	205.8
1983	273.9	278.6	2.1	3.9	19.2	248.8	253.4
1984	215.7	218.9	1.8	3.9	19.2	190.8	193.9
1985	180.8	183.0	1.8	3.9	19.2	155.9	158.1
1986	168.1	169.8	1.8	3.9	19.2	143.1	144.9
1987	187.7	189.1	1.7	3.9	19.2	162.9	164.3
1988	148.4	149.3	1.7	3.9	19.2	123.8	124.6
1989	191.0	191.5	1.7	3.9	19.2	166.1	166.7
1990	190.2	190.2	1.8	3.9	19.2	165.3	165.4
1991	256.5	256.1	1.8	3.9	19.2	231.6	231.3
1992	167.4	166.8	1.8	3.9	19.2	142.6	141.9
1993	333.8	331.6	2.1	3.9	19.2	308.6	306.5
1994	173.4	171.9	1.7	3.9	19.2	148.6	147.1
Average:	189.3	196.9	1.8	3.9	19.2	164.4	172.1

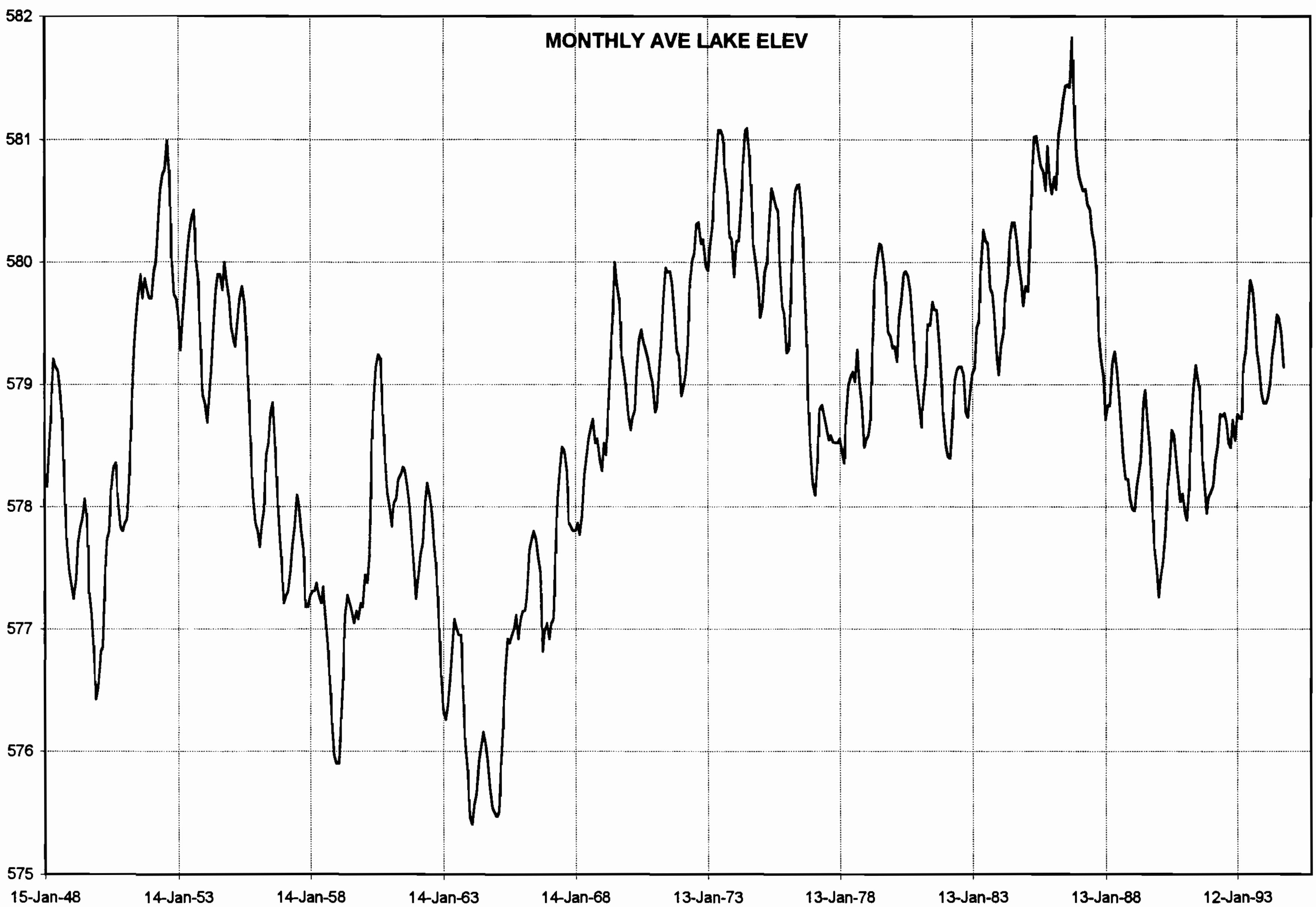


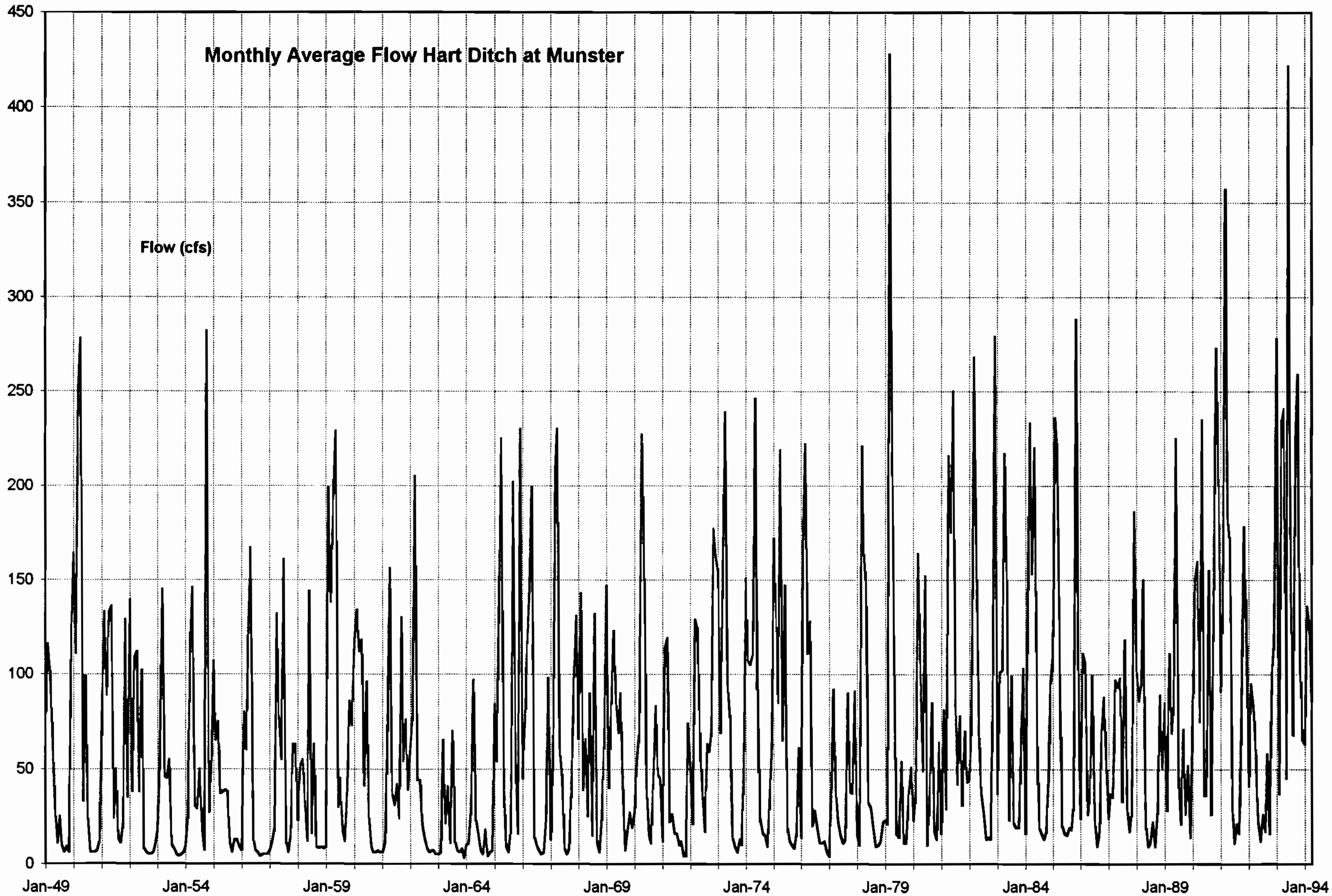
Little Calumet River
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
Flow (cfs)
1951
1953
1955
1957
1959
1961
1963
1965
1967
1969
1971
1973
1975
1977
1979
1981
1983
1985
1987
1989
1991
1993
Water Year
Little Cal
Adjusted

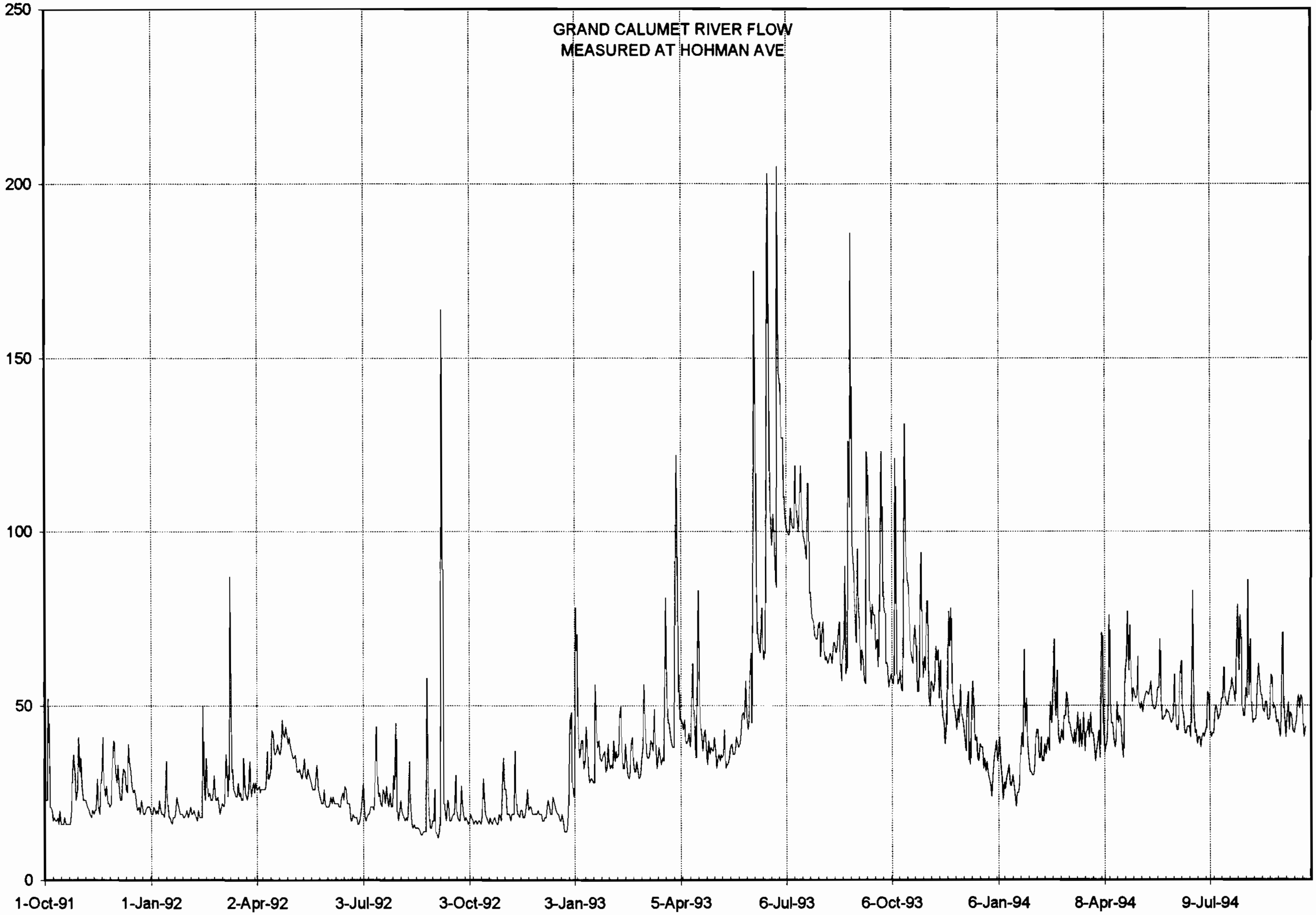
Gaged Watersheds				
Total Yearly Flows				
Date	NB Adj	LC Adj	Grn Cal	Gaged
1951	90.2	177.0	15.0	282.2
1952	101.6	201.2	54.1	356.8
1953	18.0	120.6	35.9	174.6
1954	58.7	104.0	24.4	187.1
1955	81.0	179.1	28.7	288.9
1956	27.0	124.4	7.1	158.5
1957	56.4	146.6	1.4	204.5
1958	38.6	123.8	0.1	162.6
1959	53.3	179.6	0.4	233.3
1960	100.0	196.7	8.6	305.3
1961	44.5	116.9	6.2	167.6
1962	67.3	154.3	1.4	223.0
1963	16.5	57.2	0.0	73.8
1964	40.0	52.7	0.0	92.8
1965	86.5	166.9	0.2	253.6
1966	90.6	179.0	1.1	270.7
1967	77.7	152.3	3.5	233.6
1968	39.7	178.1	6.2	224.1
1969	100.7	178.7	19.4	298.8
1970	91.8	176.3	21.7	289.8
1971	67.9	134.6	25.3	227.8
1972	122.8	194.0	31.6	348.4
1973	109.2	311.9	70.5	491.6
1974	136.7	265.6	68.1	470.4
1975	98.4	223.4	48.2	370.1
1976	91.0	193.6	46.0	330.6
1977	31.9	96.4	13.0	141.2
1978	95.8	177.5	16.9	290.2
1979	121.8	193.1	26.7	341.6
1980	84.2	182.7	29.2	296.0
1981	91.0	230.0	26.1	347.1
1982	98.6	205.8	19.2	323.6
1983	137.3	253.4	31.5	422.2
1984	121.8	193.9	37.4	353.1
1985	88.2	158.1	63.5	309.9
1986	142.1	144.9	98.5	385.4
1987	96.1	164.3	74.6	335.1
1988	71.5	124.6	20.0	216.1
1989	64.0	166.7	10.3	241.0
1990	106.0	165.4	6.1	277.4
1991	95.4	231.3	14.5	341.1
1992	83.3	141.9	10.8	236.1
1993	150.3	306.5	26.4	483.2
1994	67.4	147.1	24.7	239.2
Average:	83.0	172.1	24.4	279.5


Gaged Watersheds
Flow (cfs)
Water Year
North Branch
Little Calumet
Grand Calumet
Gaged Flows
500.0
450.0
400.0
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
1951
1953
1955
1957
1959
1961
1963
1965
1967
1969
1971
1973
1975
1977
1979
1981
1983
1985
1987
1989
1991
1993

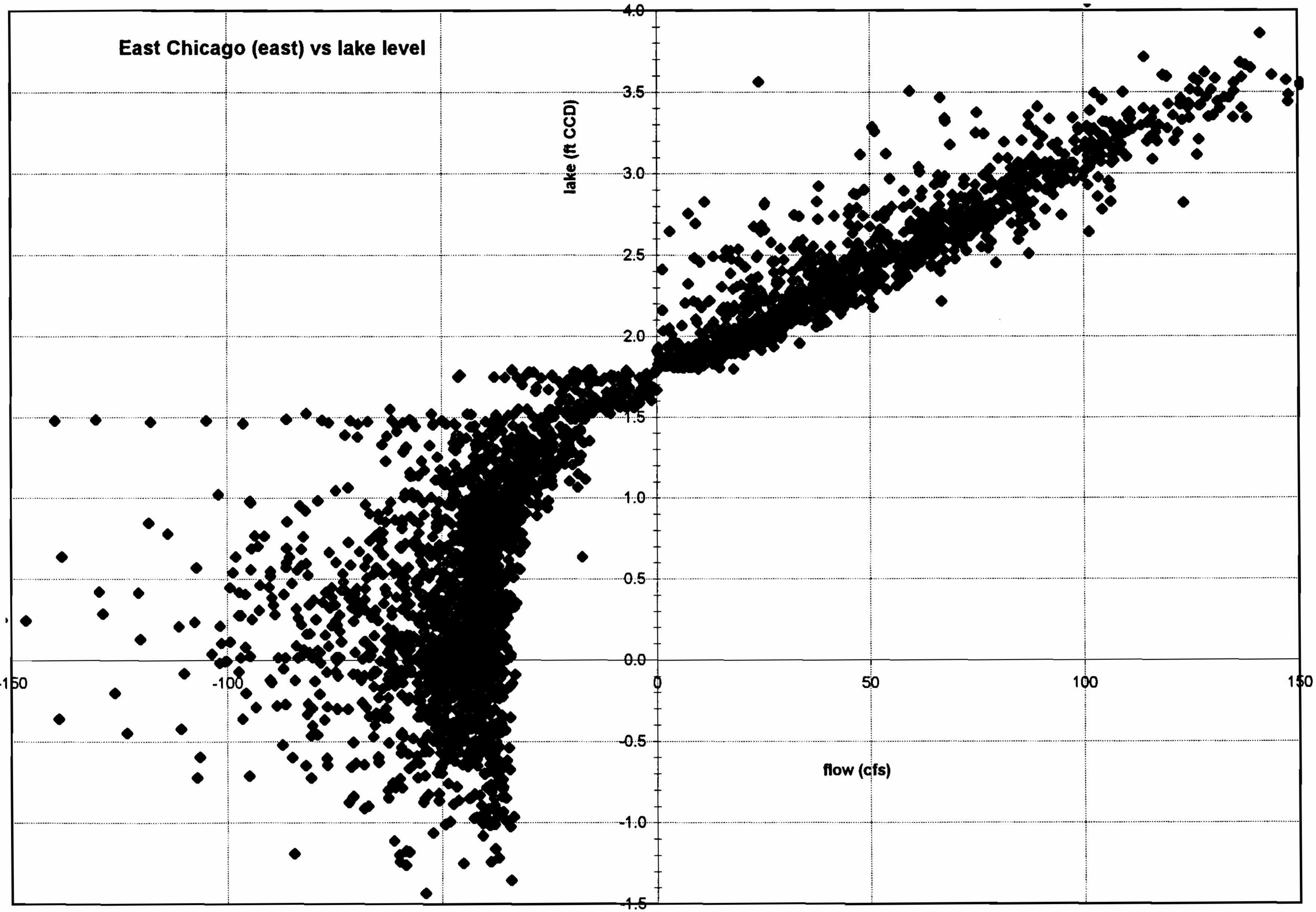
Attachment A-2
Grand Calumet River Flows

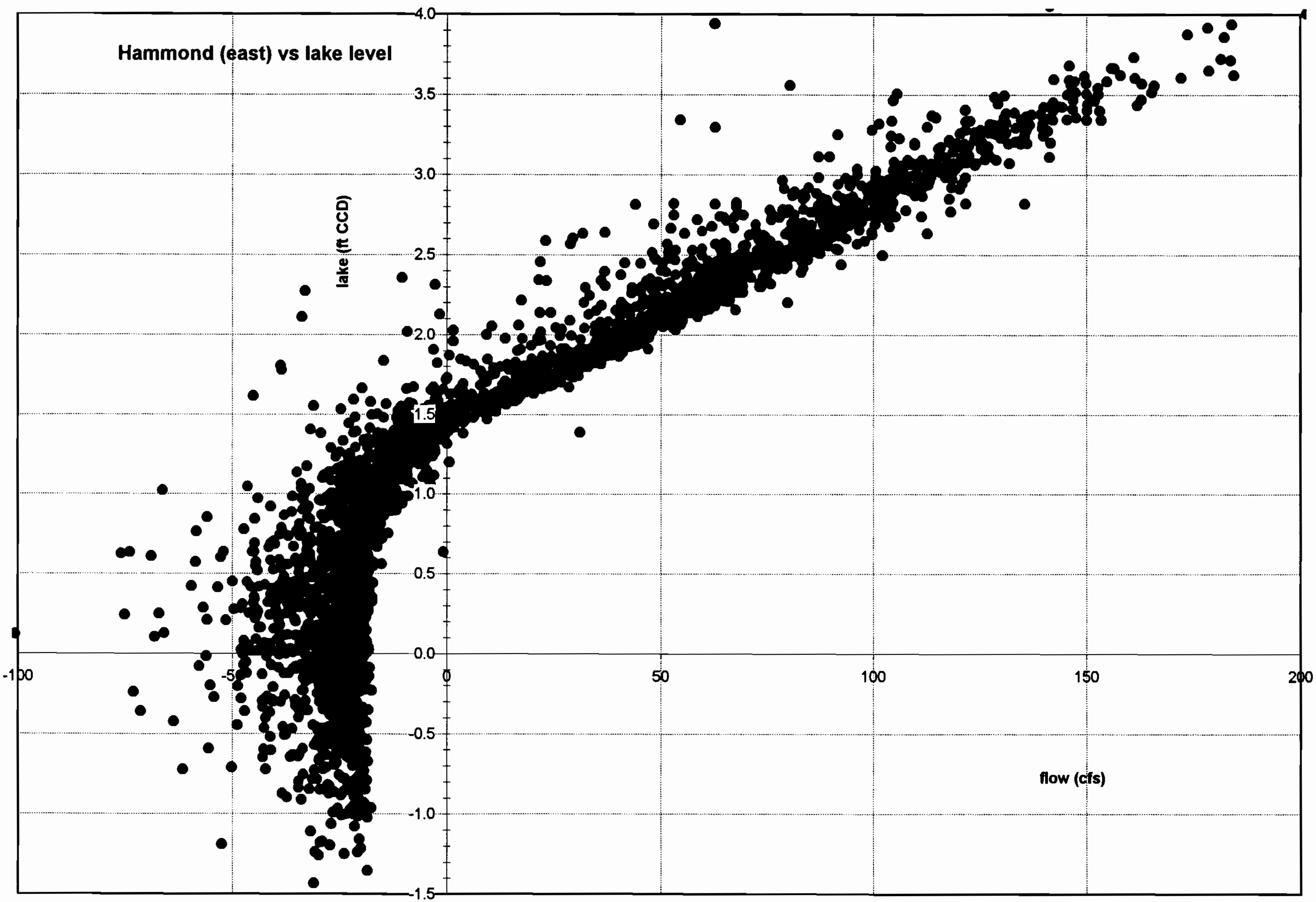

MONTHLY AVE LAKE ELEV
582
581
580
579
578
577
576
575
15-Jan-48
14-Jan-53
14-Jan-58
14-Jan-63
14-Jan-68
13-Jan-73
13-Jan-78
13-Jan-83
13-Jan-88
12-Jan-93


Monthly Average Flow Hart Ditch at Munster
Flow (cfs)
450
400
350
300
250
200
150
100
50
0
Jan-49
Jan-54
Jan-59
Jan-64
Jan-69
Jan-74
Jan-79
Jan-84
Jan-89
Jan-94

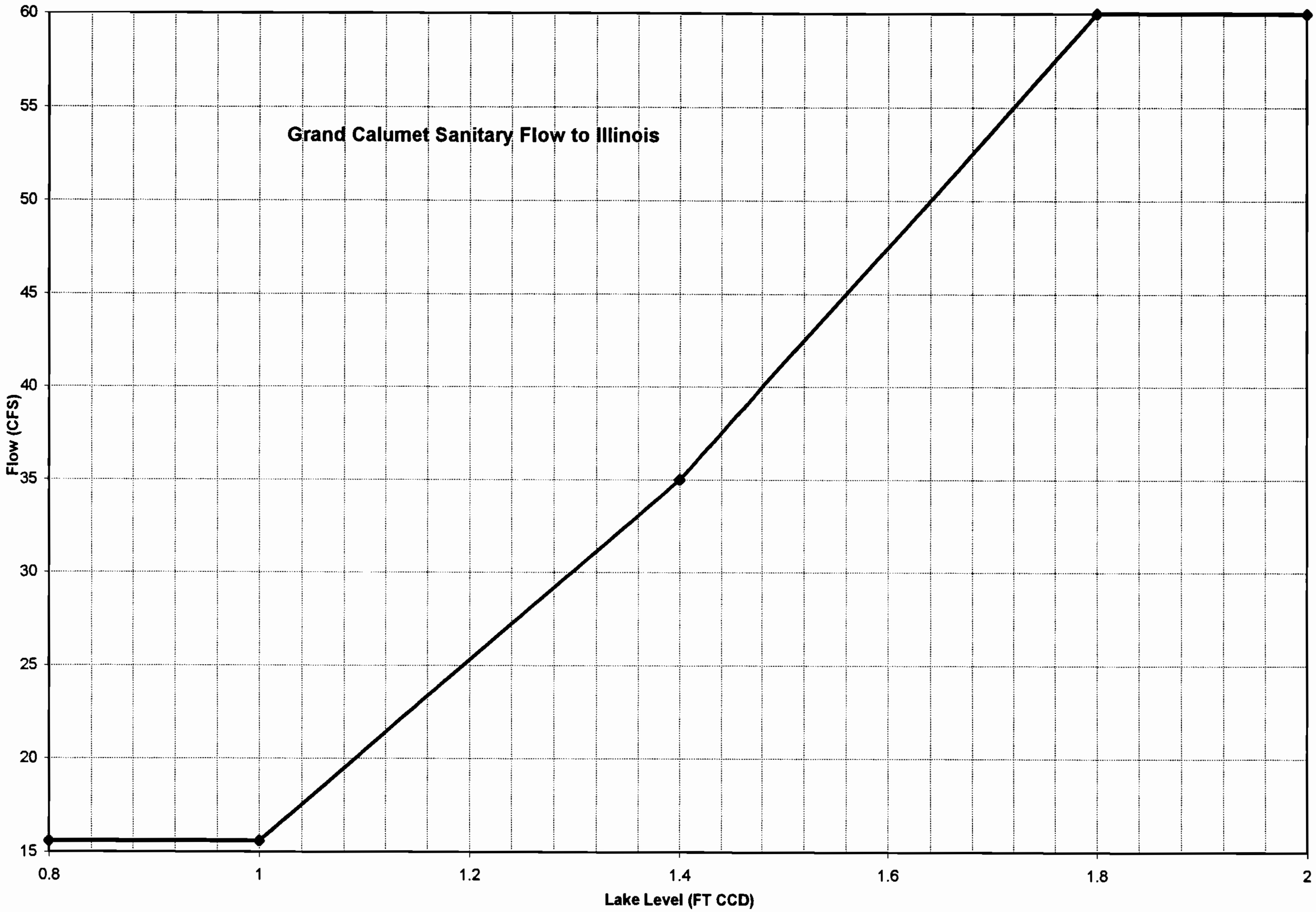


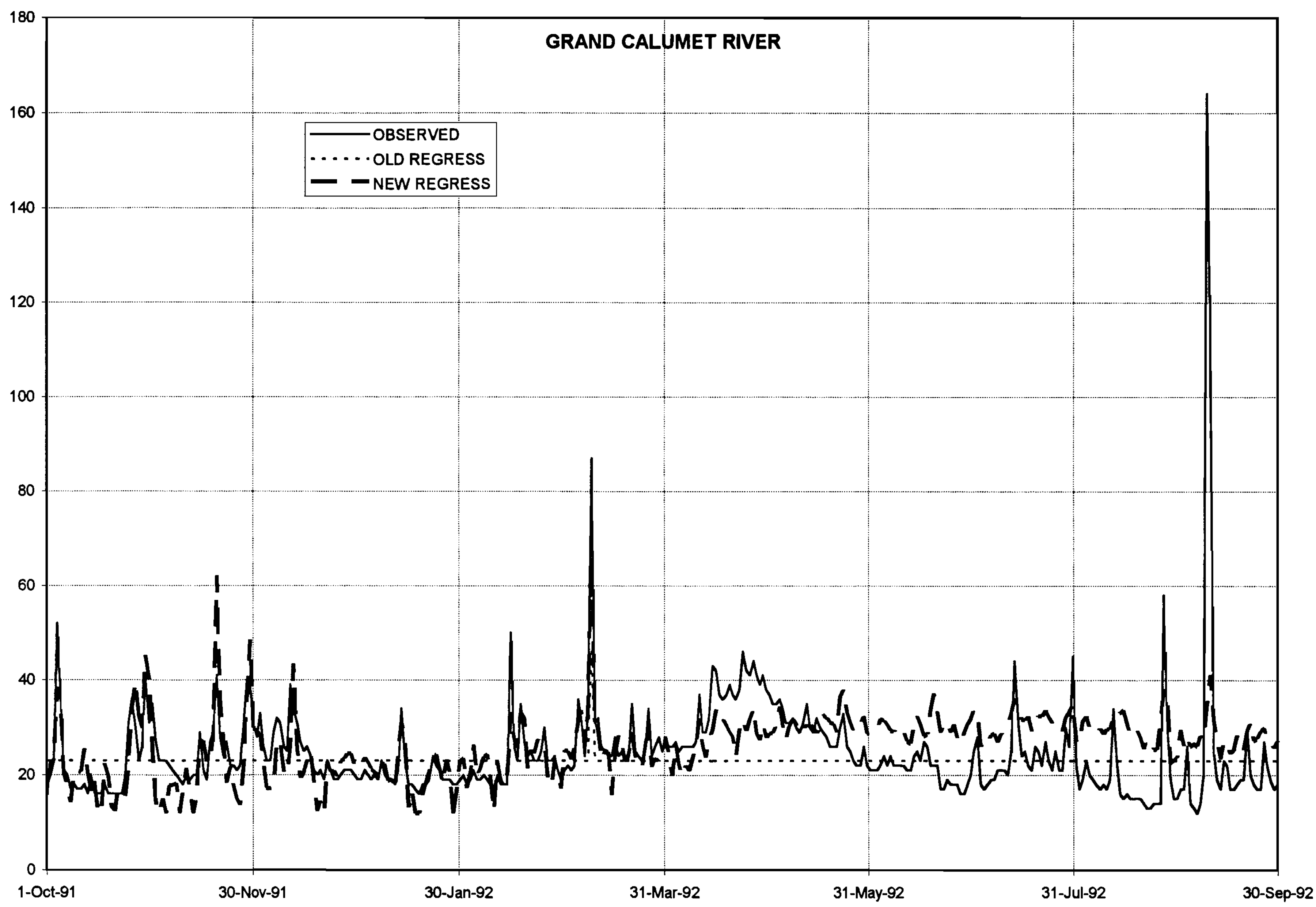
GRAND CALUMET RIVER FLOW
MEASURED AT HOHMAN AVE
250
200
150
100
50
0
1-Oct-91
1-Jan-92
2-Apr-92
3-Jul-92
3-Oct-92
3-Jan-93
5-Apr-93
6-Jul-93
6-Oct-93
6-Jan-94
8-Apr-94
9-Jul-94

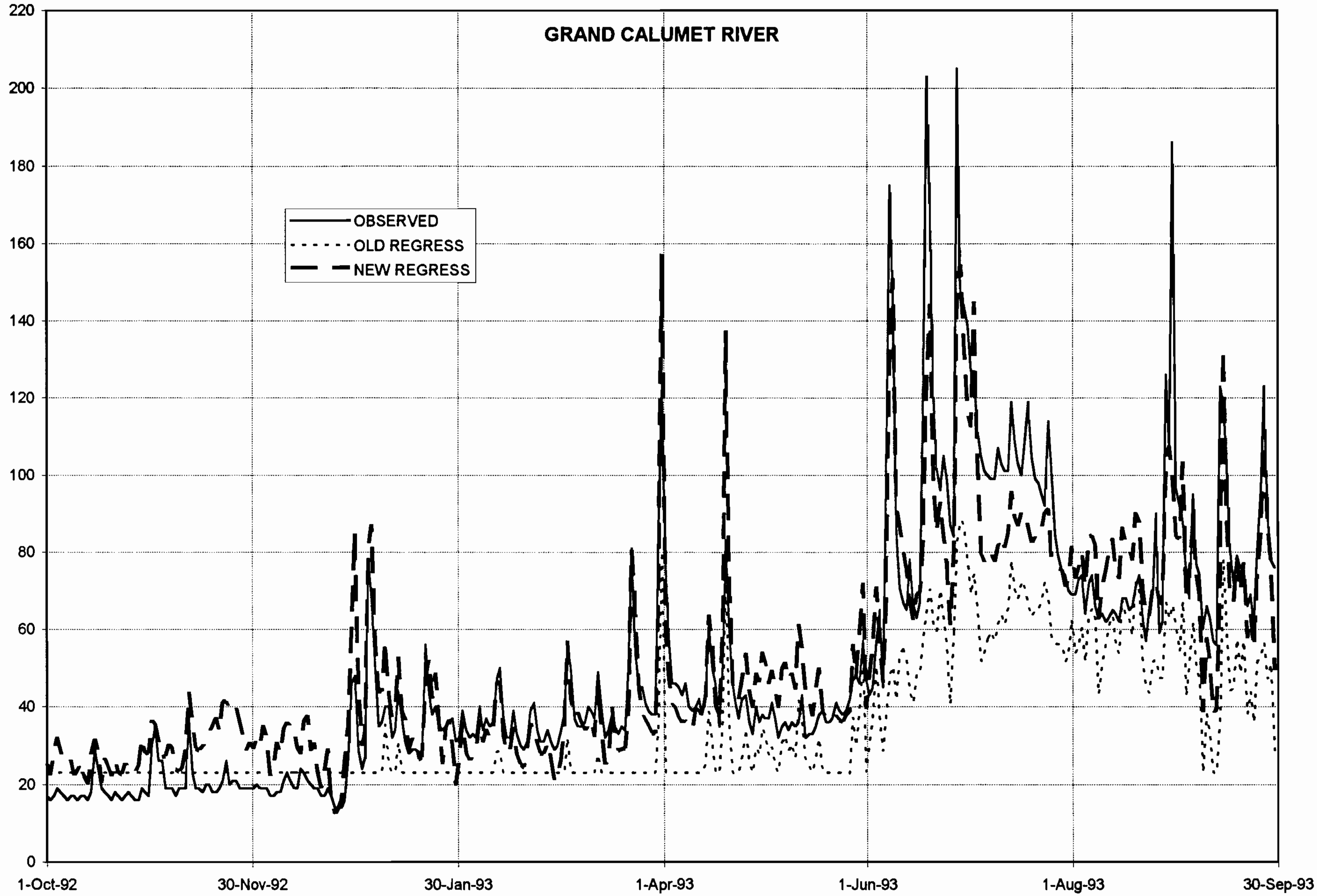

East Chicago (east) vs lake level
lake (ft CCD)
flow (cfs)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-150
-100
0
50
100
150

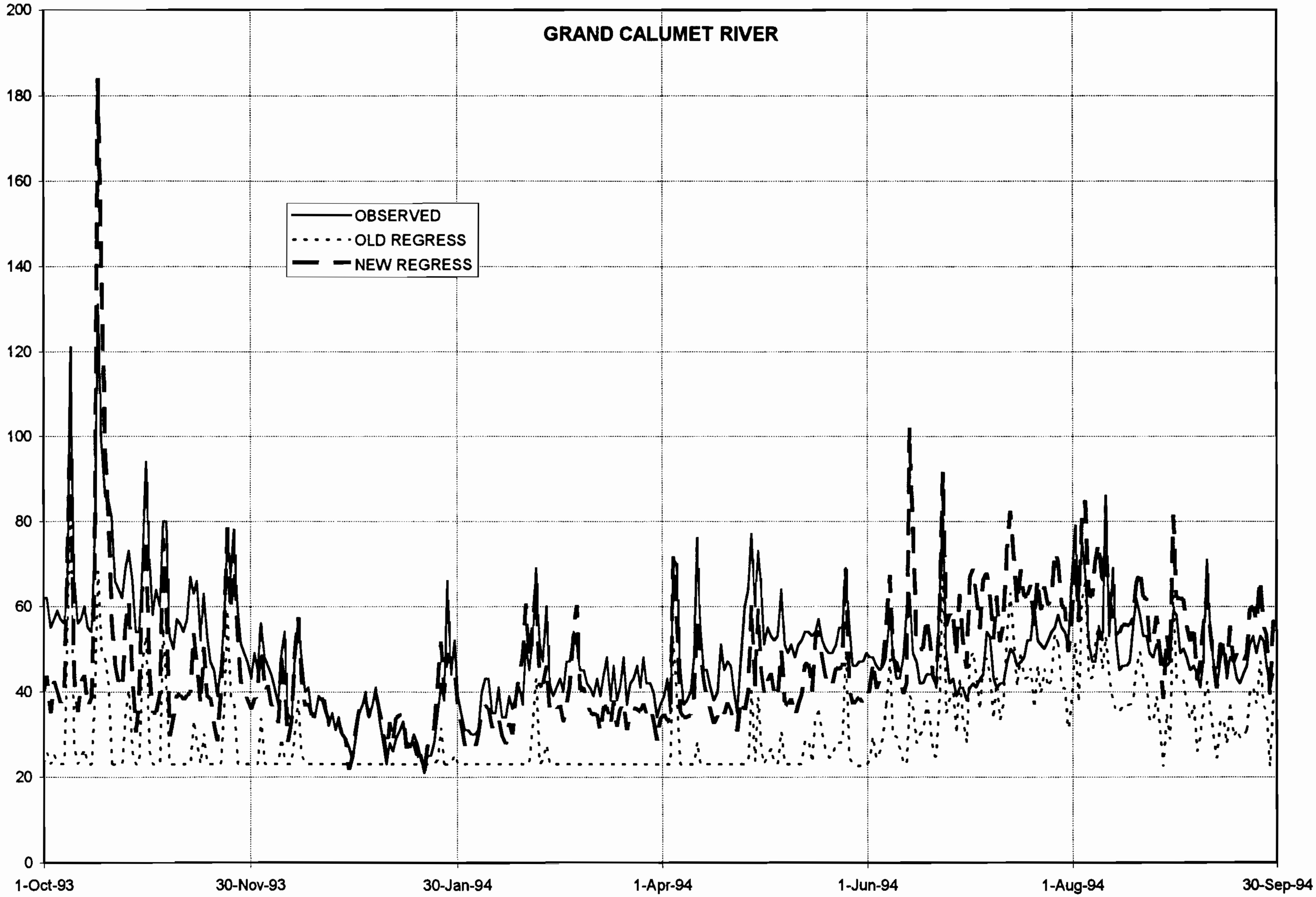


Hammond (east) vs lake level
lake (ft CCD)
flow (cfs)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-100
-50
0
50
100
150
200


Grand Calumet Sanitary Flow to Illinois
Flow (CFS)
60
55
50
45
40
35
30
25
20
15
0.8
1
1.2
1.4
1.6
1.8
2
Lake Level (FT CCD)


GRAND CALUMET RIVER
OBSERVED
OLD REGRESS
NEW REGRESS
180
160
140
120
100
80
60
40
20
0
1-Oct-91
30-Nov-91
30-Jan-92
31-Mar-92
31-May-92
31-Jul-92
30-Sep-92


GRAND CALUMET RIVER
OBSERVED
OLD REGRESS
NEW REGRESS
220
200
180
160
140
120
100
80
60
40
20
0
1-Oct-92
30-Nov-92
30-Jan-93
1-Apr-93
1-Jun-93
1-Aug-93
30-Sep-93



GRAND CALUMET RIVER
OBSERVED
OLD REGRESS
NEW REGRESS
200
180
160
140
120
100
80
60
40
20
0
1-Oct-93
30-Nov-93
30-Jan-94
1-Apr-94
1-Jun-94
1-Aug-94
30-Sep-94

Attachment A-3
Baseflow Analysis

CENCC-ED-GE 22 April 1996

MEMORANDUM THRU: CENCC-ED-G M.B. 4/22/96

FOR: CENCC-ED-H

SUBJECT: Determining the Ground Water Discharge (cubic feet/day) into the Streams of the Chicago Basin.

1. 347,656 cubic feet per day (4.02 cfs) represents the total ground water discharge, SAB, as calculated by the Chicago District and the U.S. Geological Survey (USGS). A spread sheet displaying this result is included as Enclosure 1.

2. The subject streams are as follows: the Chicago River Main Stem (entire), North Branch (from Touhy Ave. to the Main Stem), South Branch (entire), and South Fork of the South Branch (entire); the North Shore Channel (entire); the Chicago Sanitary and Ship Canal (from Western Ave. to the Cal. Sag Channel); the Calumet River (entire); the Grand Calumet River (from the Little Calumet to Hammond, Indiana), the Little Calumet River (from the Cal Sag to Hart Ditch, Indiana); and the Cal Sag Channel (from the Little Calumet to the Chicago Sanitary and Ship Canal).

3. Chicago District personnel generated the results for the Chicago Rivers, the North Shore Channel, and the Chicago Sanitary and Ship Canal. Members from the USGS produced the discharges into the Calumet Area Rivers. The USGS report is included as Enclosure 2.

4. Darcy's Equation, $Q=KAi$, was used to determine the discharge, Q (ft^3/day). "K" is the horizontal hydraulic conductivity (ft/day); "i", which is unitless, represents $\Delta h/\Delta l$; and the area, A (ft^2), is the thickness of the saturated soil thickness multiplied by the length of the reach.

5. With respect to "K", NCC used the same values, with occasional interpolation, as the USGS (see Enclosure 2). The difference between the water table elevation and the stream surface elevation determined "Δh"; and "Δl" was the distance between the stream and where the water table elevation was measured. Boring logs obtained from the Chicago Park District and the University of Illinois at Chicago (see Enclosure 3) along with the stream surface elevations furnished by NCC-ED-HH were used to generate "i" values. The saturated thicknesses were found by using the USGS methodology (Enclosure 2). Regarding the various reaches defined by ED-GE, these were demarcated by the surficial geology as extracted from the "Stack-Unit Map of Northern Illinois" by Berg and Kempton. The geologic profiles have been included as Enclosure 4.

6. Due to the general unavailability of ground water data in the Chicagoland area, engineering judgement was utilized to accomplish the subject task. The following assumptions and extrapolations were applied:

(a) For any particular reach that had no accompanying boring log data, the nearest boring was assumed to be at that reach and at the same distance away from the stream. Obvious examples include:

(i) The River Park borings, located near the confluence of the North Shore Channel and the North Branch, were used for the entire length of the North Shore Channel.

(ii) A boring from the location of the Harold Washington Library was used for the Main Stem and the South Branch and South Fork.

(iii) One boring site at 44th and Laramie was used for the entire Sanitary and Ship Canal

(b) The opposite side of each reach behaved identically to the side on which the boring was located.

(c) Regardless of the distance that a boring was from the stream, the soil characteristics remained constant.

(d) Data supplied by multiple borings from the same site, water depth, soil type, and distance from the stream, were averaged.

(e) Linear quantities were measured from USGS quad maps.

(f) All elevations are National Geodetic Vertical Datum (NGVD) of 1929.

(g) For simplicity, effects from any sheet piling along stream banks were not considered.

7. The NCC-ED-GE POC is the undersigned, 3007.

Robert Vanoer

4 Enclosures

Robert Vanoer
Civil Engineer Intern

GROUND WATER DISCHARGES INTO THE STREAMS OF THE CHICAGO BASIN -- ROBERT VANOER -- 12 APRIL 1996

REACH	REACH DESCRIPTION	GEOLOGY	K (ft/d)	SATURATED THICKNESS (ft.)	LENGTH OF REACH (ft.)	AREA (sq.ft.)	WELL HEAD (ft.)	STREAM HEAD (ft.)	DISTANCE TO STREAM (ft.)	i VALUE	Q (cf/d)
	N. Shore Channel										
	D.S. From Wilmette										
1	0–0.85 miles	Sand	5	18.7	4,488.0	83,925.6	599	578	420	5.00E–02	20,981.4
2	0.85–5 miles	Clay	0.058	18.7	21,912.0	409,754.4	599	578	420	5.00E–02	1,188.3
3	5–7 miles	Sand	5	18.7	10,560.0	197,472.0	599	578	420	5.00E–02	49,368.0
	N. Branch Chicago R.										
	D.S. from Touhy Ave.										
4	0–5 miles	Silt	0.1	9.1	27,984.0	254,654.4	596.76	592	3000	1.59E–03	40.4
5	5.3–7.1 miles	Clay	0.058	14.7	9,504.0	139,708.8	595	590	200	2.50E–02	202.6
6	7.1–9.7 miles	Fill (cinders)	5	6.2	13,728.0	85,113.6	586.45	577.9	842	1.02E–02	4,321.4
7	9.7–13 miles	Clay	0.058	12.8	17,424.0	223,027.2	574.44	577.9	1462	–2.37E–03	(30.6)
8	13–16.4 miles	Loose Silt	0.3	22	17,952.0	394,944.0	584.8	577.9	2400	2.87E–03	340.6
	Main Channel Chicago R.										
	U.S. from Confluence										
9	0–0.4 miles	Clay	0.058	23	2,112.0	48,576.0	586.67	577.9	4000	2.19E–03	6.2
10	0.4–0.7 miles	Fill	0.5	23	1,584.0	36,432.0	586.67	577.9	4500	1.95E–03	35.5
	S. Branch Chicago R.										
	D.S. From Main Channel										
11	0–1.3 miles	Clay	0.058	23	6,864.0	157,872.0	586.67	577.9	1000	8.77E–03	80.3
12	1.3–1.5 miles	Silt	0.1	12.7	1,056.0	13,411.2	586.67	577.9	1000	8.77E–03	11.8
13	1.5–5.2 miles	Clay	0.058	19.65	19,536.0	383,882.4	583.7	577.9	1000	5.80E–03	129.1
	S. Fork, S. Branch of										
	Chicago River										
	U.S. From S. Branch										
14	0–1.5 miles	Clay	0.058	10.9	7,920.0	86,328.0	584	577.9	1000	6.10E–03	30.5
	Chicago Sanitary and										
	Ship Canal										
	D.S. From Western Ave.										
15	0–7.9 miles	Clay	0.058	16.3	41,712.0	679,905.6	580.74	577.75	1345	2.22E–03	87.7
16	7.9–9.2 miles	Silt	0.1	16.3	6,864.0	111,883.2	580.74	577.6	1345	2.33E–03	26.1
17	9.2–9.5 miles	Clay	0.058	16.3	1,584.0	25,819.2	580.74	577.8	1345	2.19E–03	3.3
18	9.5–16 miles	Silt	0.1	16.3	34,320.0	559,416.0	580.74	577.9	1345	2.11E–03	118.1
			COMBINED TOTAL					TOTAL GROUND WATER DISCHARGE			
			LENGTH OF STREAMS		247,104	ft. =		FOR ONE SIDE OF STREAM			76,941
					46.8	miles					
								TOTAL FOR BOTH SIDES			153,881
										TOTAL CFS=	1.78

ENCLOSURE 1

CALUMET	AREA DISCHARGES -- USGS										
				SATURATED	LENGTH		WELL	STREAM	DISTANCE		
	REACH		K	THICKNESS	OF REACH	AREA	HEAD	HEAD	TO STREAM	i	Q
REACH	DESCRIPTION	GEOLOGY	(ft/d)	(ft.)	(ft.)	(sq.ft.)	(ft.)	(ft.)	(ft.)	VALUE	(cf/d)
1N	Calumet River	Fill	0.5	10	6,450	64,500	582.00	580.50	4,000	3.75E-04	12.1
1S	Calumet River	Fill	0.5	10	4,000	40,000	581.50	580.50	3,800	2.63E-04	5.3
2W	Calumet River	Sand	5	10	31,200	312,000	583.50	581.20	625	3.68E-03	5,740.8
2E	Calumet River	Sand	5	10	22,000	220,000	583.50	581.20	625	3.68E-03	4,048.0
3E-W	Squaw Creek	Sand	28	16	4,200	67,200	582.90	582.10	300	2.67E-03	5,017.6
3N-S	Squaw Creek	Sand	13	15	2,300	34,500	581.24	581.30	250	-2.40E-04	(107.6)
4W	Calumet River	Sand	5	8	7,100	56,800	582.75	580.90	680	2.72E-03	772.6
4E	Calumet River	Sand	5	8	7,200	57,600	582.75	580.90	680	2.72E-03	783.5
5W	Calumet River	Sand	3.4	8	4,500	36,000	582.49	580.00	350	7.11E-03	870.8
5E	Calumet River	Sand	3.4	8	2,950	23,600	581.20	580.00	15	8.00E-02	6,419.2
6	Lake Calumet	Fill	0.53	8	42,700	341,600	582.50	579.90	200	1.30E-02	2,353.6
7W	Pullman Creek	Fill	0.53	8	5,000	40,000	581.83	579.90	150	1.29E-02	272.8
7E	Pullman Creek	Fill	0.53	8	5,000	40,000	581.83	579.90	150	1.29E-02	272.8
8	Lake Calumet	Fill	0.53	8	18,400	147,200	580.93	579.90	1,500	6.87E-04	53.6
9W	Little Calumet River	Sand	0.8	6	12,000	72,000	580.63	578.40	250	8.92E-03	513.8
9E	Little Calumet River	Sand	1	7	12,000	84,000	581.00	578.40	3,550	7.32E-04	61.5
10S	Little Calumet River	Fines	0.058	21	12,500	262,500	579.04	578.30	1,300	5.69E-04	8.7
10N	Little Calumet River	Fines	0.058	21	11,000	231,000	579.04	578.30	1,300	5.69E-04	7.6
11S	Little Calumet River	Sand	11	8	9,000	72,000	580.00	578.30	500	3.40E-03	2,692.8
11N	Little Calumet River	Sand	11	8	10,300	82,400	580.00	578.30	500	3.40E-03	3,081.8
12N	Cal-Sag Channel	Sand	11	6	18,500	111,000	579.73	578.60	750	1.51E-03	1,839.6
12S	Cal-Sag Channel	Sand	11	6	18,500	111,000	579.73	578.60	750	1.51E-03	1,839.6
13N	Grand Calumet River	Sand	10	10	14,400	144,000	580.15	578.5	150	1.10E-02	15,840.0
13S	Grand Calumet River	Sand	10	10	14,400	144,000	580.15	578.5	150	1.10E-02	15,840.0
14N	Grand Calumet River	Sand	10	15	15,400	231,000	580.86	579.70	50	2.32E-02	53,592.0
14S	Grand Calumet River	Sand	10	15	15,400	231,000	580.86	579.70	50	2.32E-02	53,592.0
15N	Little Calumet River	Fines	0.058	20	6,300	126,000	588.00	587.00	300	3.33E-03	24.4
15S	Little Calumet River	Fines	0.058	20	6,300	126,000	588.00	587.00	300	3.33E-03	24.4
16N	Little Calumet River	Sand	5	5	16,000	80,000	586.00	585.50	300	1.67E-03	666.7
16S	Little Calumet River	Sand	5	5	16,000	80,000	586.00	585.50	300	1.67E-03	666.7
17N	Little Calumet River	Fines	0.058	25	10,500	262,500	595.52	583.90	2,750	4.23E-03	64.3
17S	Little Calumet River	Fines	0.058	25	10,500	262,500	595.52	583.90	2,750	4.23E-03	64.3
18N	Little Calumet River	Sand	5	15	20,600	309,000	597.80	580.00	5,900	3.02E-03	4,661.2
18S	Little Calumet River	Sand	5	15	20,600	309,000	597.80	580.00	5,900	3.02E-03	4,661.2
19N	Little Calumet River	Fines	0.058	25	17,500	437,500	580.00	579.60	500	8.00E-04	20.3
19S	Little Calumet River	Fines	0.058	25	17,500	437,500	580.00	579.60	500	8.00E-04	20.3
20E	Little Calumet River	Sand	5	5	6,300	31,500	580.00	578.30	500	3.40E-03	535.5
20E	Little Calumet River	Sand	5	5	6,300	31,500	580.00	578.30	500	3.40E-03	535.5
21N	Cal-Sag Channel	Sand	11	6	32,208	193,248	579.73	578.60	750	1.51E-03	3,202.8
21S	Cal-Sag Channel	Sand	11	6	32,208	193,248	579.73	578.60	750	1.51E-03	3,202.8
								TOTAL GROUND WATER DISCHARGE			193,775
										TOTAL CFS=	2.24
		GRAND TOTAL GROUND WATER DISCHARGE=				347,656	CFD		GRAND TOTAL CFS=		4.02

United States Department of the Interior

GEOLOGICAL SURVEY

1420 Sycamore Road
DeKalb, IL 60115
April 7, 1996

Mr. Kevin Richards
U.S. Army Corp of Engineers
Suite 600
111 N. Canal St.
Chicago, IL 60606-7206

Dear Mr. Richards:

As per our discussions, I have calculated the amount of ground-water discharge to surface water on the Calumet River system based on data collected during June 22-24, 1992. The information on which these calculations are based can be obtained from U.S. Geological Survey Water-Resources Investigations Report 95-4253 "Geohydrology, Water Levels and Directions of Flow, and Occurrence of Light-Nonaqueous-Phase Liquids on Ground Water in Northwestern Indiana and the Lake Calumet area of Northeastern Illinois" by Kay and others. You should be receiving a copy of this report in the next few days.

As we agreed, the calculation of discharge from ground water to surface water is based on the solution of the Darcy Equation $Q = K A (dh/dl)$. Q is the calculated discharge, in cubic feet per day. K is the horizontal hydraulic conductivity of the deposits surrounding the stream, in feet per day. A is the cross-sectional area of flow to the stream, which is equal to the length of stream reach (L), in feet, multiplied by the thickness of the zone of ground-water flow to the stream (b), in feet. dh is the change in water level between surface water (SWL) and ground water (GWL), in feet. dl is the distance over which the water-level change was measured, in feet. Conservative assumptions were made, where needed, to provide a realistic calculation for the maximum amount of ground-water discharge to surface water in this area. For example, it is assumed (probably unrealistically) that the presence of sheet piles along the rivers does not affect the amount of flow between ground water and surface water. Stream reaches were divided into areas where flow direction is constant and the surrounding geology is uniform. Data were not available for a number of tributaries to the Little Calumet River, including Thorn Creek and the Union Drainage Canal, and tha amount of ground-water discharge to these bodies was not calculated. In addition, ground-water discharge to Wolf Lake was not calculated because of the complex interactions between surface water and ground water in this area.

Horizontal hydraulic conductivity was assumed to be 5.8×10^{-2} ft/d if the surrounding deposits were composed of fine-grained material (silt and clay). This is the median horizontal hydraulic conductivity value for the shallow fine-grained material in this area (Kay and others, 1996, p. 33). Horizontal hydraulic conductivity of fill deposits was assumed to be 0.5 ft/d based on median values calculated at a site near Lake Calumet. Horizontal hydraulic conductivity of the sand deposits was the value determined from slug tests in the nearest well, if one was available (Kay and others, 1996, table 1, and other reports). Otherwise, a value of 5.0 ft/d was assumed.

ENCLOSURE 2

Stream lengths were measured from U.S. Geological Survey quadrangle maps for the area. The thickness of the zone of flow to the stream was assumed to be equal to the saturated thickness of the surrounding sand deposit if the stream was surrounded by sand. If the stream was surrounded by fine-grained material, the thickness of the zone of flow was assumed to be equal to the saturated thickness of the weathered zone in these deposits. The weathered zone is assumed to be 30 ft thick based on reports from studies in this area.

The difference in water level was measured by subtracting the water level in the well nearest the stream reach being analyzed (Kay and others, 1996, Appendix 1) by a representative measured or interpolated value of surface-water altitude (Kay and others, 1996, fig. 15). If well data were not available, it was estimated from ground-water contours (Kay and others, 1996, plate 1). The distance between the point of ground-water measurement and the surface-water measuring point was estimated from site-specific or quadrangle maps.

Variations in geology, data coverage, length of reach, and other factors made it possible to calculate the ground-water discharge from each bank along reaches 1, 2, 4, 5, 9, 10, and 11. Because the remaining reaches did not have any identified differences in the variables required for solution of the Darcy equation, equal discharge at each bank of the river was assumed.

The Calumet River System was divided into the following reaches:

1. Calumet River from Calumet Harbor to Turning Basin 1. This area is characterized by saturated fill deposits surrounding the river. During the period of measurement flow along this reach was toward Lake Michigan. Much of this area contains sheet piling. The nearest wells are at Calumet Park.

North Side	South Side
L = 6,450 ft	L = 4,000 ft
b = 10 ft	b = 10 ft
SWL = 580.5 ft	SWL = 580.5 ft
GWL = 582.0 ft	GWL = 581.5 ft
dl = 4,000 ft	dl = 3,800
K = 0.5 ft/d	K = 0.5 ft/d
Q = 16 ft^3/d	Q = 5 ft^3/d

2. Calumet River from Turning Basin 1 to Squaw Creek. Squaw Creek is the creek that connects Wolf Lake to the Calumet River. This area is characterized by sand deposits surrounding the river. During the period of measurement, flow along this reach was toward Lake Michigan. The calculation assumes that the sheet piling in this area has no effect on the rate of flow between ground water and surface water. Wells on the east side of the river indicate flow from the river to ground water, but none are in the immediate vicinity of the river. Ground-water levels are available on the west side of the river at the Wisconsin Steel site. It was assumed that dh/dl on the east side of the river equaled that on the west side of the river.

West Side	East Side
L = 31,200 ft	L = 22,000 ft
b = 10 ft	b = 10 ft
SWL = 581.2 ft	SWL = 581.2 ft
GWL = 583.5 ft	GWL = 583.5 ft (assumed)
dl = 625 ft	dl = 625 (assumed)
K = 5 ft/d	K = 5 ft/d
Q = 5,741 ft^3/d	Q = 4,048 ft^3/d

3. Squaw Creek. Squaw Creek flows from Wolf Lake to the Calumet River and was the location of a surface-water divide on the Calumet River between flow toward Lake Michigan and flow toward Lake Calumet during June 22-24, 1992. This area is characterized by sand deposits surrounding the river. Well data is available near Wolf Lake and near the bend in Squaw Creek. The data indicate that ground water flows to surface water along the east-west reach of the creek but surface water flows to ground water along the north-south reach of the creek (negative flow).

E-W Reach
(north side)
L = 4,200 ft
b = 16 ft
SWL = 582.1 ft
GWL = 582.9 ft
dl = 300 ft
K = 28 ft/d
Q = 5,018 ft^3/d

(south side assumed to equal north)
Q = 5,018 ft^3/d

N-S Reach
(west side)
L = 2,300 ft
b = 15 ft
SWL = 581.3 ft (lowest possible)
GWL = 581.24 ft
dl = 250 (estimated)
K = 13 ft/d
Q = -107 ft^3/d

(east side assumed to equal west)
Q = -107 ft^3/d

4. Calumet River from Squaw Creek to Turning Basin 5. Flow is from Squaw Creek to Lake Calumet. This area is characterized by sand deposits surrounding the river. Ground-water levels are available near the turning basin on the west side of the river. No nearby wells are present on the east side of the river. SWL, GWL, and dl values for the east side of the river are assumed to equal those for the west side. Sheet pile is present along some of this reach on the east bank of the river.

West Side
L = 7,100 ft
b = 8 ft
SWL = 580.9 ft
GWL = 582.75 ft
dl = 680 ft
K = 5 ft/d
Q = 772 ft^3/d

East Side
L = 7,200 ft
b = 8 ft
SWL = 580.9 ft (assumed)
GWL = 582.75 ft (assumed)
dl = 680(assumed)
K = 5 ft/d
Q = 784 ft^3/d

5. Calumet River from Turning Basin 5 to O'Brien Lock and Dam. Flow in this reach is not documented but is probably toward the lock. The lock is typically closed, restricting the flow of water north of the lock to the south of the lock. This reach is surrounded by sand deposits. Sheet pile is present along some of this reach on the west bank of the river.

West Side
L = 4,500 ft
b = 8 ft
SWL = 580.0 ft (estimate)
GWL = 582.49 ft
dl = 350 ft
K = 3.4 ft/d
Q = 871 ft^3/d

East Side
L = 2,950 ft
b = 8 ft
SWL = 580.0 ft (estimate)
GWL = 581.20 ft
dl = 15
K = 3.4 ft/d
Q = 6,419 ft^3/d

6. Lake Calumet from Turning Basin 5 to Pullman Creek inlet. Pullman Creek is the creek that runs along the western edge of the lake. This reach includes the piers in Lake Calumet. Flow in this reach is not well defined, but was into the lake during the period of measurement. A constant lake level of 579.9 ft was assumed. This reach is surrounded by fill deposits. Sheet pile is present along some of this reach near the Turning Basin.

L = 42,700 ft
b = 8 ft
SWL = 579.9 ft
GWL = 582.5 ft
dl = 200 ft
K = 0.53 ft/d
Q = 2,354 ft^3/d

7. Pullman Creek. Flow on this reach was not defined during the June 1992 measurement period, but it is reported to be typically to Lake Calumet. A lake level of 579.9 ft was assumed. This reach is surrounded by fill deposits. Data are only available for the west side of the reach. It is assumed that flow from the east side and flow from the west side are equal.

West Side
L = 5,000 ft
b = 8 ft
SWL = 579.9 ft
GWL = 581.83 ft
dl = 150 ft
K = 0.53 ft/d
Q = 273 ft^3/d

(East Side)
Q = 273 ft^3/d

8. Lake Calumet from Pullman Creek inlet to Turning Basin 5. A lake level of 579.9 ft was assumed. This reach is surrounded by fill deposits. Sheet piling is present along much of this reach.

L = 18,400 ft
b = 8 ft
SWL = 579.9 ft
GWL = 580.93 ft
dl = 1500 ft
K = 0.53 ft/d
Q = 54 ft^3/d

9. Little Calumet River from the O'Brien Lock and Dam to about 2,000 ft west of I-94. Flow in this reach was toward the Calumet Sag Channel. This reach is surrounded by sand deposits.

West/North Bank
L = 12,000 ft
b = 6 ft
SWL = 578.4 ft
GWL = 580.63 ft
dl = 250 ft
K = 0.8 ft/d
Q = 514 ft^3/d

East/South Bank
L = 12,000
b = 7 ft
SWL = 578.4 ft
GWL = 581.0 ft (estimate)
dl = 3,550 ft
K = 1.0 ft/d
Q = 62 ft^3/d

10. Little Calumet River from about 2,000 ft west of I-94 to the bend in the river near ACME Steel site/Indiana Ave. Flow on this reach was toward the Calumet Sag Channel. This reach is surrounded by fine-grained deposits. No data are available for the north bank so all of the variables for the north bank, except L, are assumed to be equal to those for the south bank.

South Bank
L = 12,500 ft
b = 21 ft
SWL = 578.3 ft
GWL = 579.04 ft
dl = 1,300 ft
K = 0.058 ft/d
Q = 9 ft^3/d

North Bank
L = 11,000
b = 21 ft
SWL = 578.3 ft
GWL = 579.04 ft
dl = 1,300 ft
K = 0.058 ft/d
Q = 8 ft^3/d

11. Little Calumet River from the bend in the river near ACME Steel site/Indiana Ave. to the Cal Sag Channel. Flow on this reach was toward the Calumet Sag Channel. This reach is surrounded by sand deposits. No nearby data are available for this reach so most of the variables are estimated.

South Bank
L = 9,000 ft
b = 8 ft (estimated)
SWL = 578.3 ft
GWL = 580.0 ft (estimated)
dl = 500 ft (estimated)
K = 11 ft/d (estimated)
Q = 2,693 ft^3/d

North Bank
L = 10,300 ft
b = 8 ft (estimated)
SWL = 578.3 ft
GWL = 580.0 ft (estimated)
dl = 500 ft (estimated)
K = 11 ft/d (estimated)
Q = 3,082 ft^3/d

ED-H: PARTIAL FAX?

12. Cal Sag Channel to Crawford Ave. Flow on this reach was assumed to be to the west. This reach is surrounded by sand deposits. Flow from the south bank, where no data are available, is assumed to equal flow from the north bank. Sheet piling may be present along much of this reach.

North Bank
L = 18,500 ft
b = 6 ft
SWL = 578.6 ft
GWL = 579.73 ft
dl = 750 ft
K = 11 ft/d (estimated)
Q = 1,839 ft^3/d

(South Bank)
Q = 1,839 ft^3/d

13. Grand Calumet River from the State Line to the Little Calumet River. Flow in this reach was to the west. This reach is surrounded by sand deposits. Discharge from the south bank, where no data is available, is assumed to equal discharge from the north bank.

North Bank
L = 14,400 ft
b = 10 ft (estimated)
SWL = 578.5 ft
GWL = 580.15 ft
dl = 150 ft
K = 10 ft/d
Q = 15,840 ft^3/d

(South Bank)
Q = 15,840 ft^3/d

14. Grand Calumet River from the State Line to the Hammond Wastewater Treatment Plant. Flow in this reach was to the west. East of the Hammond Plant, surface-water flow is toward the east. This reach is surrounded by sand deposits. Discharge from the south bank, where non data is available, is assumed to equal discharge for the north bank.

North Bank
L = 15,400 ft
b = 15 ft (estimated)
SWL = 579.7 ft
GWL = 580.86 ft
dl = 50 ft
K = 10 ft/d (estimated)
Q = 53,592 ft^3/d

(South Bank)
Q = 53,592 ft^3/d

15. Little Calumet River from Hart Ditch to about 1,000 ft east of Calumet Ave. Flow in this reach was to the west. East of Hart Ditch, surface-water flow is toward the east. This reach is surrounded by fine-grained deposits. The variables for this area are estimated. Equal flow from the south and north is assumed.

North Bank
L = 6,300 ft
b = 20 ft (estimated)
SWL = 587 ft (estimated)
GWL = 588 ft (estimated)
dl = 300 ft (estimated)
K = 0.058 ft/d (estimated)
Q = 24 ft^3/d

(South Bank)
Q = 24 ft^3/d

16. Little Calumet River from about 1,000 ft east of Calumet Ave to about Burnham Ave.. Flow in this reach was to the west. This reach is surrounded by sand deposits. The variables for this area are estimated. Equal flow from the south and north is assumed.

North Bank
L = 16,000 ft
b = 5 ft (estimated)
SWL = 585.5 ft (estimated)
GWL = 586.0 ft (estimated)
dl = 300 ft (estimated)
K = 5 ft/d (estimated)
Q = 667 ft^3/d

(South Bank)
Q = 667 ft^3/d

17. Little Calumet River from about Burnham Ave. to about 1,000 ft west of Torrence. Flow in this reach was to the west. This reach is surrounded by silt and clay deposits. Equal flow from the south and north is assumed.

South Bank
L = 10,500 ft
b = 25 ft
SWL = 583.9 ft
GWL = 595.52 ft
dl = 2,750 ft
K = 0.058 ft/d
Q = 64 ft^3/d

(South Bank)
Q = 64 ft^3/d

18. Little Calumet River from about 1,000 ft west of Torrence Ave to about 1,000 ft west of South Park Ave.. Flow in this reach was to the west. This reach is surrounded by sand deposits. No data are available to the south of the river for this reach. Equal flow from the south and north is assumed.

North Bank
L = 20,600 ft
b = 15 ft (estimated)
SWL = 580.0 ft
GWL = 597.80 ft
dl = 5,900 ft
K = 5 ft/d (estimated)
Q = 4,661 ft^3/d

(South Bank)
Q = 4,661 ft^3/d

19. Little Calumet River from about 1,000 ft west of South Park Ave. to the bend in the river west of I-57. Flow in this reach was to the west. This reach is surrounded by silt and clay deposits. Equal flow from the south and north is assumed.

North Bank
L = 17,500 ft
b = 25 ft (estimated)
SWL = 579.6 ft
GWL = 580.0 ft (estimated)
dl = 500 ft (estimated)
K = 0.058 ft/d
Q = 20 ft^3/d

(South Bank)
Q = 20 ft^3/d

20. Little Calumet River from the bend in the river west of I-57 to the Cal Sag Channel. Flow in this reach was to the west. This reach is surrounded by sand deposits. Equal flow from the east and west is assumed.

East Bank
L = 6,300 ft
b = 5 ft (estimated)
SWL = 578.3 ft
GWL = 580.0 ft (estimated)
dl = 500 ft (estimated)
K = 5.0 ft/d (estimated)
Q = 536 ft^3/d

(West Bank)
Q = 536 ft^3/d

Summary of calculated ground-water inputs are as follows:

Reaches 1, 2 and 3 14,721 ft^3/d
Calumet River north of Squaw Creek toward Lake Michigan (assumes half of inflow from Squaw Creek, reach 3, to Lake Michigan and half to Lake Calumet)

Reaches 3, 4, 5, 6, 7 and 8 16,711 ft^3/d
Calumet River and Lake Calumet south and west of Squaw Creek to the O'Brien Lock and Dam (assumes half of inflow from Squaw Creek, reach 3, to Lake Michigan and half to Lake Calumet)

Reaches 13 and 14 138,864 ft^3/d
Grand Calumet River west of the Hammond Treatment Plant. Flow is to the Cal Sag Channel.

Reaches 9, 10, and 11 6,368 ft^3/d
Little Calumet River from the Grand Calumet River to the Cal Sag Channel. Flow is to the Cal Sag Channel.

Reaches 15, 16, 17, 18, 19 and 20 11,944 ft^3/d
Little Calumet River from Hart Ditch to the Cal Sag Channel. Flow is to the Cal Sag Channel.

Reach 12 3,678 ft^3/d
Cal Sag Channel

The daily mean discharge of the Grand Calumet River during June 23 and 24, 1992 was measured to be 1,555,200 ft^3/d at a station near the State Line in Indiana. The daily mean discharge of the Little Calumet River during June 23 and 24, 1992 was measured to be 864,000 ft^3/d at a station near the State Line in Indiana.

Feel free to call me at 815-76-9207 if you have any further questions or comments regarding this analysis.

Robert T. Kay

Robert T. Kay
Hydrologist

cc. Oberg
Martin

PARK DISTRICT BORINGS [0' CCD = 579.48' NGVD]

1/4

SUMMARY

SAUGANASH PK

JAN : FEB 1976

BORING	FT. B. S. WATER	K VALUE	@ WATER MATERIAL	DISTANCE TO STREAM N. BRANCH	WATER NGVD EL
1	18	5.8×10^{2} ft/d	TOUGH SILTY CLAY	3,000'	583.28
2	8'		LOOSE SILTY CLAY	"	599.38
3	6'		VERY TOUGH SILTY CLAY	"	601.28
4	6.5		TOUGH - HARD " "	"	600.78
5	6.5		" " " "	"	600.78
6	25		TOUGH SILTY CLAY	"	582.03
7	12'		HARD - TOUGH SILTY CLAY	"	595.58
8	8'		FIRM CLAYEY SILT	"	599.48
9	8		" " "	"	599.48
10	8		TOUGH SILTY CLAY	"	599.38

$\bar{m} = \frac{106}{10} = 10.6$

$\bar{m} = \frac{5967.6}{10}$ 596.76

~~$\frac{4796.24}{8} = 599.5$~~

GOMPERS PK

BORING	WATER DEPTH BELOW SURFACE	K VALUE	MATERIAL @ WATER TABLE	DISTANCE TO STREAM N. BRANCH	WATER EL. NGVD 595
NORTH OF FOSTER (3/23/73) 1	5'	5.8×10^{-2} ft/day	GRAY SILT, BROWN CLAY	200'	595.00
2	DRY			—	
3	"			—	
4	"			—	
5	"			—	
SOUTH OF FOSTER (6/27/34) 1	DRY 15' BORE			1000'	
2	DRY 15' BORE				

RIVER PL

BORING	WATER DEPTH BELOW SURF.	K VALUE	MATERIAL @ WATER TABLE	DISTANCE TO STREAM N. B. CHANNEL	WATER EL NGVD
6/18/70 1	1.5	5.0 ft/day	FINE SILTY SAND	450'	598.5
2	2.5	0.5 ft/day	BLACK ORGANIC CLAY [illegible]	300'	599.5

ENCLOSURE 3

PARK DISTRICT BORING SUMMARY (CONT)

HORNER PK

BORING	WATER DEPTH BELOW SURFACE	MATERIAL @ WATER TABLE	K VALUE	DISTANCE TO STREAM	WATER TABLE EL. NGVD
(7-28-71) Bleving					
1	24'	TOUGH, SILTY CLAY	5.8×10^{-2} ft/d	225'	566.0
2 California & Montrose	DRY				
(4-3-72)					
1	7.0	FILL, CINDERS, SOME RUBBISH	5.0 ft/d	805	587.58
2	9.5	" " "	"	795	587.48
3	10.0	" " "	"	857.5	584.98
4	8.6	" " "	"	910'	585.78
5	6.5	BROWN & GRAY CLAY, TOUGH, OCCASIONAL SAND	5.8×10^{-2}	920'	587.98

$\bar{m}$ w/out #5 = $\frac{35.1}{4}$ = 8.8 $\qquad$ $\bar{m}$ = 857.5 $\qquad$ $\bar{m}$ = 586.76

w/out #5 $\bar{m}$ = 842 $\qquad$ $\bar{m}$ = 586.45

6/4/70

WELLES PK.

BORING	WATER DEPTH BELOW SURFACE	MATERIAL @ WATER TABLE	K VALUE	DISTANCE TO STREAM	WATER EL. NGVD
1	5.7	FINE SAND LITTLE SILT, LOOSE	5.0 ft/d	2,300'	589.3
2	5.0	" "	"	"	590.0
3	5.0	" "	"	"	590.0
4	5.0	" "	"	"	590.0

7/17/74

CALIFORNIA PK

BORING	WATER DEPTH BELOW SURF	MATERIAL @ WATER TABLE	K VALUE	DISTANCE TO STREAM	WATER EL NGVD
1	19	CLAY FILL W/ LAYERS OF DEBRIS	0.5 ft/d	620'	572.4
2	18	ORGANIC CLAY (FILL)	0.5 ft/d	570'	573.4
				$\bar{m}$ = 595	$\bar{m}$ = 572.9

REVERE PK

6/2/77

			K VALUE	DISTANCE	NGVD
1	10'	CINDER & DEBRIS FILL LOOSE SAND, CINDERS CLAY	5.0 ft/d	600'	581.0

PARK DISTRICT
BORING SUMMARY

7/1/69

BRANDS PK

BORING	WATER DEPTH BELOW SURF.	MATERIAL @ WATER TABLE	K VALUE	DISTANCE TO STREAM	WATER EL. NGVD
1	25	DENSE SILTY SAND	5 ft/d	(1,425)	(566.63
2	17	STIFF to SOFT SILTY CLAY	5.8×10^{-2} ft/d	1,550	574.71
3	15	" " "	"	1,400	576.26
4	28	DENSE SANDY SILT	"	1,300	563.69
(12-1-71) 6	15	STIFF TO SOFT SILTY CLAY	"	1,600	577.58
7	23	" " "	"	1,500	568.3
8	23	FIRM SAND & GRAVEL W/ CLAY BINDER	5 ft/day	(1,455)	(568.85
9	14	STIFF SILTY CLAY	5.8×10^{-2} ft/d	1,545	577.63
10	14	" " "	"	1,355	577.43
11	11.5	" " "	"	1,445	579.89

$\bar{m} = \frac{185.5}{10} = 18.5$ or $\frac{137.5}{8} = 17.2$ $\bar{m} = \frac{11695}{8} = 1462$ $\bar{m} = \frac{4595.49}{8}$ $= 574.44$

HAMLIN PK

BORING	WATER DEPTH BELOW SURF.	MATERIAL @ WATER TABLE	K VALUE	DISTANCE TO STREAM	WATER EL NGVD
1	DRY TO 20'		—		
2	DRY TO 20'		—		

DURSO

BORING	WATER DEPTH BELOW SURF	MATERIAL @ WATER TABLE	K VALUE	DISTANCE TO STREAM	WATER EL NGVD
1	DRY TO 15'		—		
2	DRY TO 15'		—		

4-7-86

SEWARD PK

BORING	DEPTH	MATERIAL	K	DISTANCE	NGVD
1	8	LOOSE BROWN SILT	0.3 ft/day	2,350	584.8
2	10	STIFF SILTY CLAY W/ SAND	.055 ft/day	2,300	582.5
3	8	LOOSE BROWN SANDY SILT	0.3 ft/day	2,450	584.9
4	8	" " "	"	2,400	584.9

$\bar{m} = 2,4$[illegible]

PARK DISTRICT
BORING LOG
SUMMARY

LE CLAIRE - HEARST (#305)

BORING	WATER DEPTH BELOW SURFACE	MATERIAL @ WATER TABLE	K VALUE	DISTANCE TO STREAM	WATER EL. NGVD
1	DRY				
2	8.0	VERY TOUGH TO TOUGH SILTY CLAY	0.058 ft/day	1,100	586.18
3	DRY			1,100	
4	14.0	TOUGH SILTY CLAY	"	1,170	580.18
5	DRY				
6	25	TOUGH GRAY SILTY CLAY	"	1,050	569.98
7	13	" " "	"	1,115	581.68
8	8.5	" " "	"	1,200	585.68

$\bar{m} = \frac{68.5}{5} = 13.7$

$\bar{m} = \frac{6735}{5} = \boxed{1347}$

$\bar{m} = \frac{2903.7}{5} = \boxed{580.74}$

$30 - 13.7 = 16.3$

S. Branch Beyond Roosevelt ? S. Fork

$(13.7 + \cancel{10})/2 = 11.85$

$\boxed{(13.7 + 7)/2 = 10.35 \quad 30 - 10.35 = 19.65}$

$(\cancel{580.74} + 520.4)/2 = \cancel{580.57}$

$(580.74 + 586.67)/2 = 583.7$

~~for~~ S. Fork: $(11.85 + 10)/2 = 10.9$

$(530.57 + 530.4)/2 = 580.5$

ASSUME
SAY CAHOKIA = ~~0.02~~ 0.1 ft/d

L-8-6-42

530.33

FROM DR. REDDY

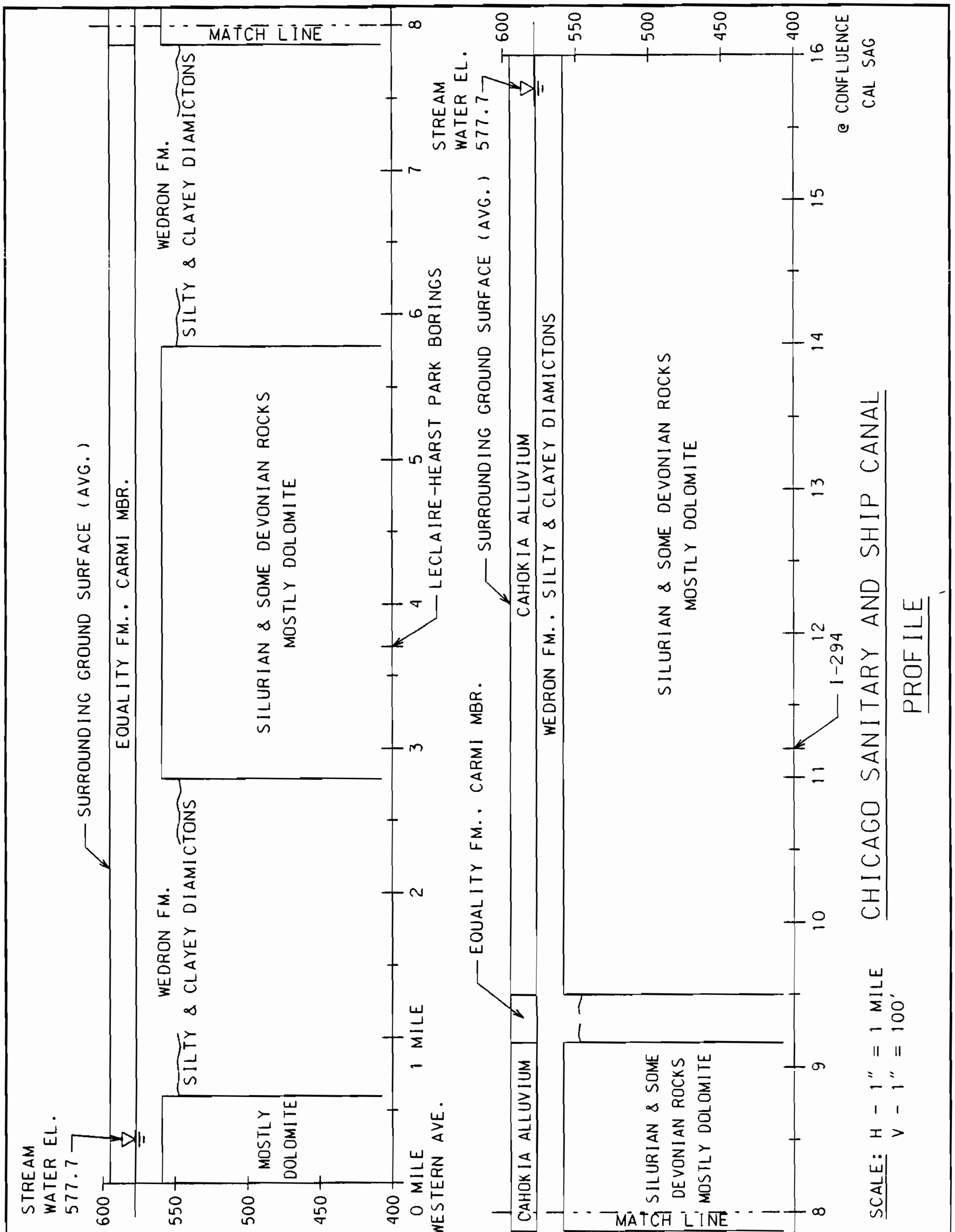
$\boxed{\text{N. Branch Lib.}}$

$\boxed{7\ \cancel{8.0}}$

water level 19.0 ~~8.5~~ A.D.

$\boxed{\text{3L. 14.2 ...}}$

+ 577...
13.3
573.33
593.67 − 7 = 586.67



(CONTRACTOR)

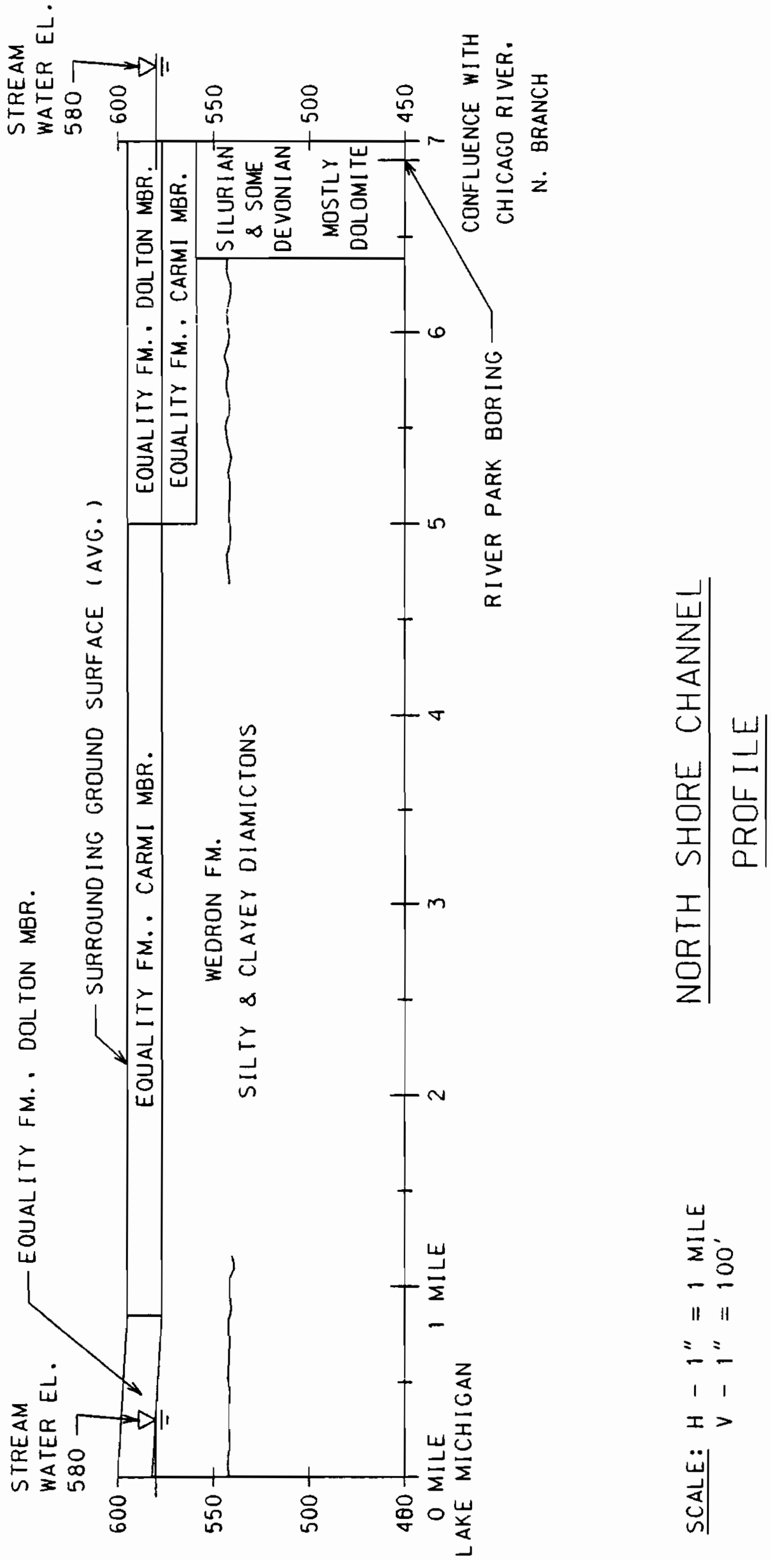
(TOWN, STATE)

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO, ILLINOIS

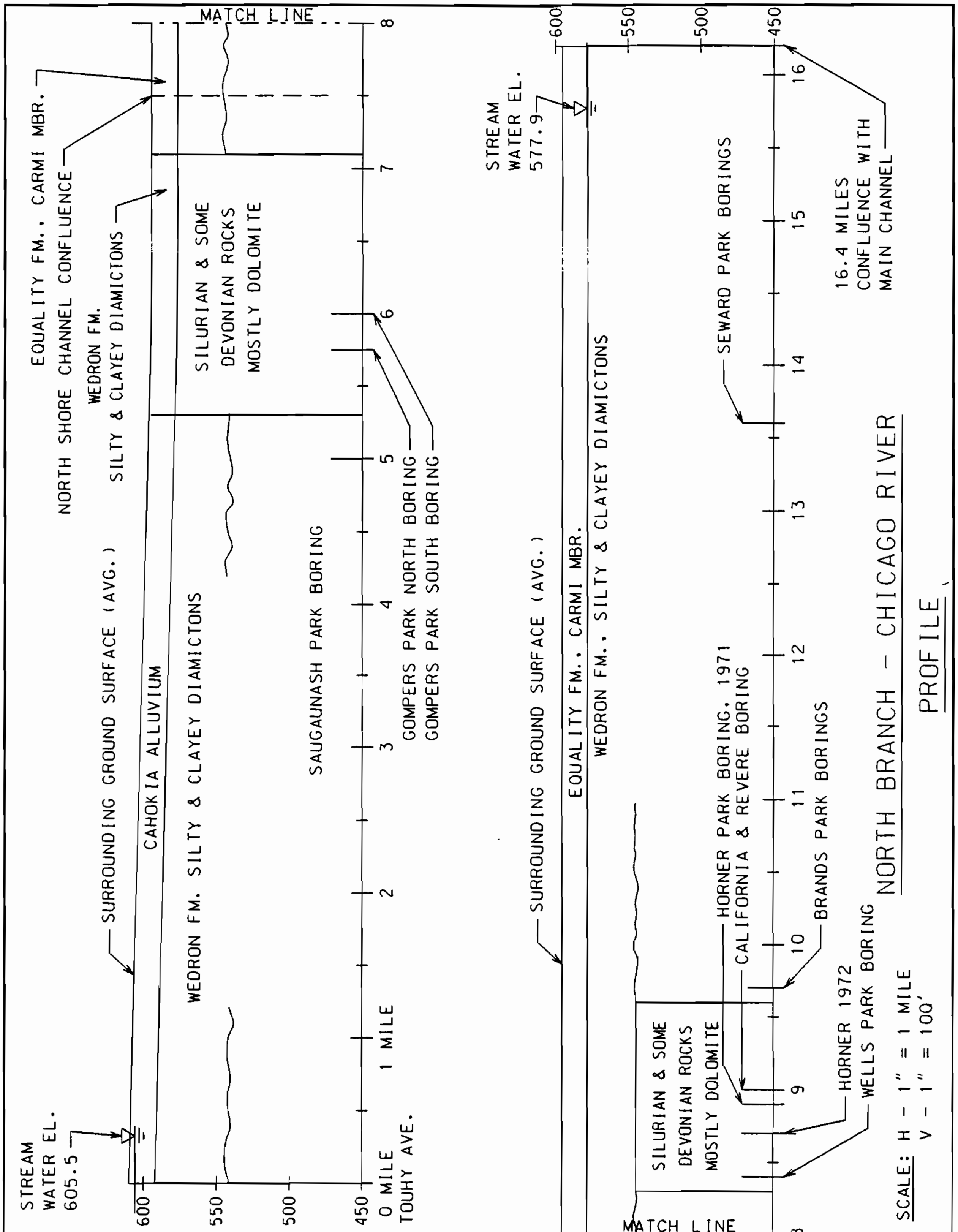
GEOLOGIC CONDITIONS
SURROUNDING THE STREAMS
OF THE CHICAGO BASIN

Scale	Date	Drawing
	8 APRIL 1996	

ENCLOSURE 4



(CONTRACTOR) (TOWN, STATE)	GEOLOGIC CONDITIONS SURROUNDING THE STREAMS OF THE CHICAGO BASIN
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS CHICAGO, ILLINOIS	Scale · Date 8 APRIL 1996 · Drawing



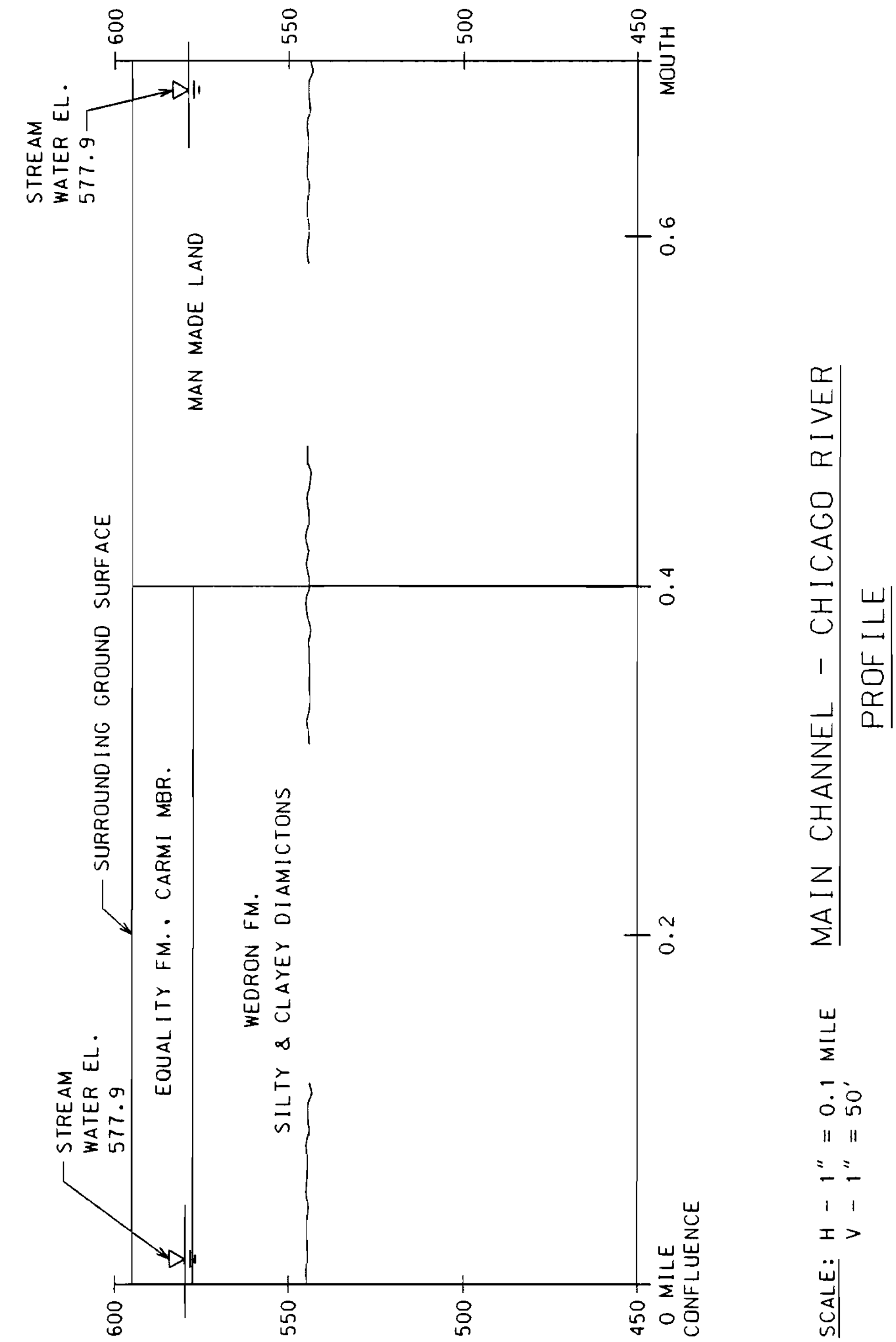
(CONTRACTOR)

(TOWN, STATE)

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO, ILLINOIS

GEOLOGIC CONDITIONS SURROUNDING THE STREAMS OF THE CHICAGO BASIN

Scale	Date	Drawing
	3 APRIL 1996	



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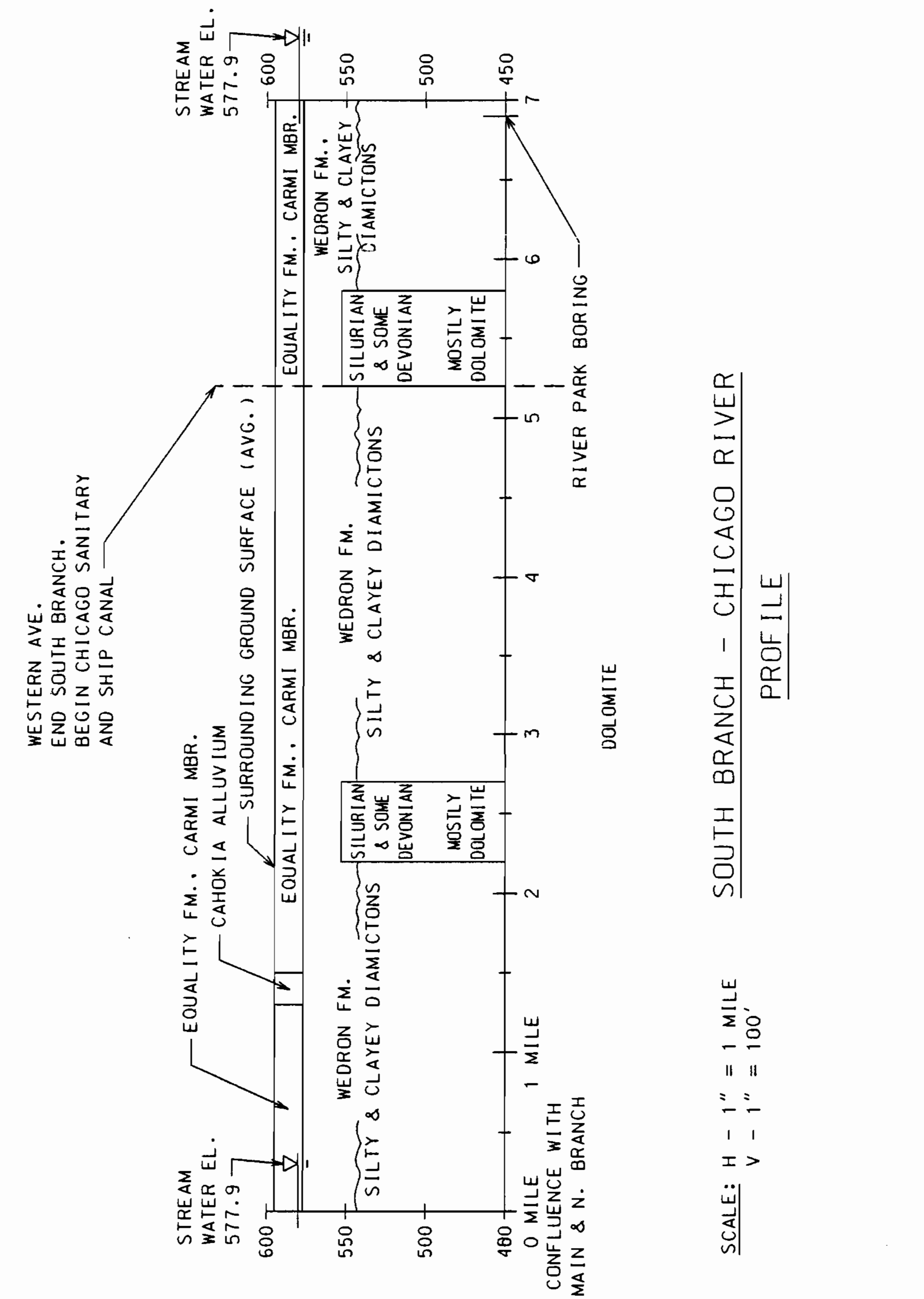
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO, ILLINOIS

GEOLOGIC CONDITIONS
SURROUNDING THE STREAMS
OF THE CHICAGO BASIN

Scale	Date	Drawing
	3 APRIL 1996	



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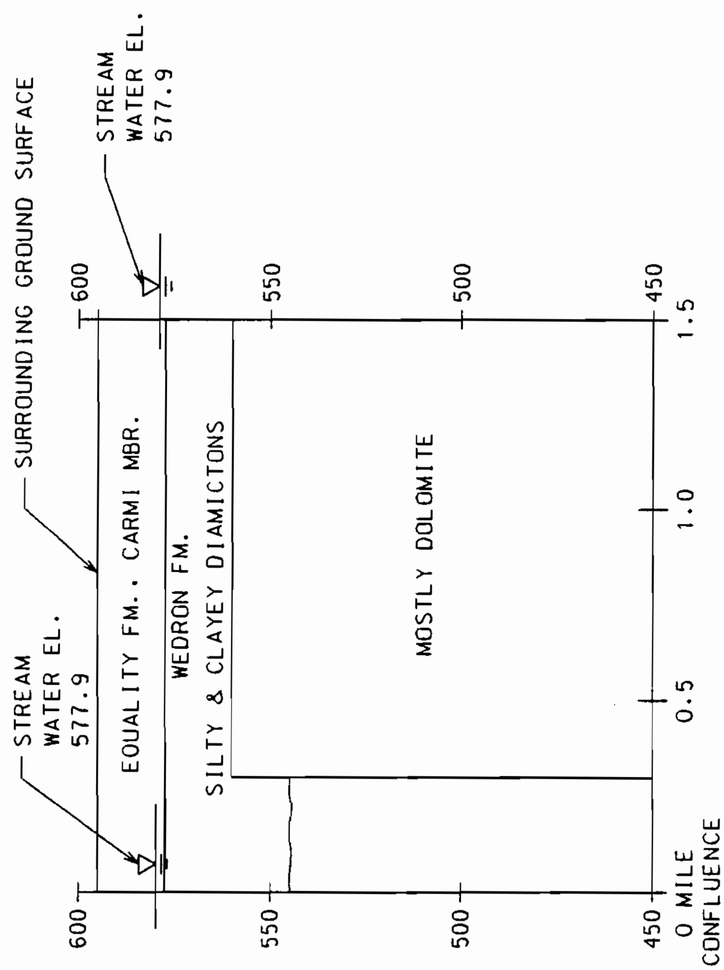
(TOWN, STATE)

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO, ILLINOIS

GEOLOGIC CONDITIONS
SURROUNDING THE STREAMS
OF THE CHICAGO BASIN

Scale	Date	Drawing
	8 APRIL 1996	



SOUTH FORK - SOUTH BRANCH - CHICAGO RIVER

PROFILE

SCALE: H - 1" = 1/2 MILE
V - 1" = 50'

(CONTRACTOR) (TOWN, STATE)	GEOLOGIC CONDITIONS SURROUNDING THE STREAMS OF THE CHICAGO BASIN		
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS CHICAGO, ILLINOIS	Scale	Date 8 APRIL 1996	Drawing

Appendix B

Runoff Sensitivity Analyses

Introduction

1. To gain a better understanding of the long-term runoff values a series of comparisons and analyses have been undertaken. These evaluations include:

- A comparison of the Chicago District's period of record analysis with the analysis of long term average runoff values conducted by the Northeastern Illinois Planning Commission (NIPC).
- A comparison of the period of record runoff values with those computed for the diversion accounting reports
- A series of trend analyses of the rainfall and period of record runoff values.
- A sensitivity analysis of the rainfall gages utilized in the analysis.
- A sensitivity analysis of the effects of imperviousness.
- A comparison with the historic record at Lockport.
- A mass balance of rainfall and runoff components.

Comparison of Chicago District and NIPC Results

2. The runoff study by NIPC resulted in an annual runoff of 636 cfs while the Chicago District's study resulted in an annual average of 785 cfs (including 4.0 cfs for baseflow). A comparison of methodologies used provides a rationale the difference in results. Four primary differences in methodology will be discussed:

- Period of record
  NIPC: WY49-79
  Chicago District: WY51-94
- Model parameters
- Determination of runoff from streamflow areas
- Precipitation data employed in the models

3. The period of record it self explains some of the difference of 149 cfs between the two studies. NIPC used a 31-year period of record (WY49-79) while the District used a 44-year period of record (WY51-94). The District's study shows an average annual runoff of 741.6 cfs over the 29 year period of record portion common to both studies (WY51-79) while the average annual runoff for the 15 year period beyond the NIPC study (WY80-94) was 869.6 cfs. Weighted Lake Michigan watershed precipitation from the three gages used in the District's study was 34.5 inches over the common period (WY51-79) and 37.8 inches over the period beyond the NIPC study (WY80-94).

4. The second factor contributing to the differences in results between the District and NIPC is the use of different model parameters. NIPC used models that were in existence for the WY83 accounting. Some of the model parameters were questioned by the Second Technical Committee and subsequently revised by Christopher Burke Engineering, Ltd. under contract to the District. The model revisions incorporated in the WY84 accounting resulted in increasing the runoff component of flow in the sewers. Those revisions, also used in the runoff study, resulted in a large improvement in simulated to recorded ratios at the MWRDGC WRPs when compared to model results from WY83. Refer to the 1989 Annual Report containing the WY84 and WY85 Accounting Reports for additional details.

5. The third major difference between the District's and NIPC's studies is the method of determining runoff at stream gage sites outside the MWRDGC WRP service areas. The method employed by NIPC was to fully simulate those areas. The District employed streamflow separation techniques. Streamflow separation is superior to modeling in these areas since it helps to account for the complex hydraulics of the rivers in the southeast portion of the diverted watershed. Additionally, streamgages help to capture changes in the isohyetal precipitation distribution since they actually measure flows resulting from localized storms (NIPC used Midway Airport as the sole precipitation gage in their modeling and variances in storm distributions went unnoticed). Finally, both NIPC's modeling of the streamflow areas and the District's adjustment of streamflow records for WY90 conditions account for changes in runoff due to urbanization. However, NIPC's models reflect urbanization conditions from the 1970's up to 1980, while the Corps model reflects 1990 conditions. Additional information on the District's adjustments for urbanization is provided in the Runoff Analysis Appendix (A).

6. The last major difference between the two studies is in the precipitation data used in the modeling. NIPC used only the Midway Airport gage, while the District also used data from the O'Hare Airport and University of Chicago gages. At first glance it would appear that the NIPC study would result in slightly higher runoffs due to precipitation, since the Midway gage measured an annual precipitation of 35.3 inches over the period WY51-79, while the O'Hare and University of Chicago precipitation gages measured 32.9 and 34.8 inches respectively. However, the Midway gage tends to measure low during extreme events which may tend to negate some of the runoff. It is typical for more extreme events to produce a larger proportion of runoff, as compared to infiltration and evapotranspiration losses. Additional information on the effects of the precipitation

is provided in the section of this appendix on the sensitivity runoff as a function of the rainfall gages used in the modeling.

Comparison of Long Term Runoff and Accounting Report Results

7. The comparison of the period of record flows with those previously computed for the diversion accounting reports is given in attachment B-1. Table B-1.1 and Figure B-1.1 in the attachment provide a series of period of record flows and accounting report flows for WY83-94. The initial statistics for the period show that the period of record flows have an average of 885.9 cfs, with a standard deviation of 238.6 cfs. The corresponding accounting report flows for this period have an average of 868.2 cfs, with a standard deviation of 240.3 cfs. A two-tailed paired t-test was completed for WY83-94, comparing the period of record versus accounting report flows. The results of the t-test showed that there was a 69% chance that the flows were from the same population and the correlation coefficient (80%) for the period showed that the flows do tend to move together. Finally, a regression analysis of the period of record flows (independent variable) versus the accounting report runoffs (dependent variable) is presented in Figure B-1.2 (Period of Record versus Accounting Runoff) given in the attachment. The low coefficient of determination (0.38) suggests that there could be significant errors if the regression analysis were used with period of record flows to predict certified flows.

8. To more accurately evaluate the significance of the comparisons, Table B.1 on the following page, details the changes in the methodology used to compute the certified runoff for each year. From the table, it can be pointed out that major changes occurred to the certified modeling for WY84, when the hydraulic parameters were updated, and for WY90 when the model was recalibrated for the revised gage network. The breaks in the comparison can also be seen in the second chart presented in the attachment (Water Year versus Runoff). The conclusion reached from a review of these breaks (notably the second break) is that the WY90 diversion accounting model should be used to compute the long-term runoff. This model more accurately reflects the current land use conditions and the recalibration performed for the new gage network.

9. An additional factor that should be considered is the quality of the calibration of the runoff models. Table B.2 provides the results of a comparison of the ratios of simulated versus record flows at each of the water reclamation plants (WRP), for each of the years in which runoff flows were certified. From this table it can be seen that the calibrations before WY90 were generally low at all of the WRPs. After the recalibration, the flows were high at the West Southwest WRP, low at the North Side WRP and satisfactory at the Calumet WRP. However, the flows at the West Southwest WRP are greater than the sum of the flows at the North Side and Calumet WRPs. This implies that the recalibrations are a little high in WY90 and later, compared to somewhat low in WY89 and before.

Table B.1 Changes Effecting Runoff Simulations

Water Year	Modification
1983	NIPC, under contract to the State of Illinois, constructed the first diversion accounting simulation models. The basis of the models was a previously constructed continuous simulation hydrologic model prepared by NIPC for a US Environmental Projection Agency 208 study.
1984	Updated the hydraulic model parameters questioned by the second technical committee. The parameters for SCALP were updated, and this increased the flows through the sewers and therefore the simulated runoff.
1985	Prepared concurrently with the WY84 report, no additional changes.
1986	The Mainstream System TARP was incorporated into the model. TARP has no effect on the runoff.
1987	The Calumet System TARP was incorporated into the model. TARP has no effect on the runoff.
1988	Prepared with the WY86-89 reports, no additional changes.
1989	The sanitary flow estimates in SCALP for the Calumet WRP service area were improved. This improved the simulated to record flow ratio at the plant, but did not have a significant impact on the runoff. Improvements were also made to groundwater seepage estimates and unrestricted sewer connections for Calumet System TARP models. Again these improvements did not have a significant impact on the runoff.
1990	The 25-gage precipitation network was implemented. Changes to runoff due to the better precipitation distribution, the increased number of gages, and the improved accuracy of the measurements is not easily quantifiable. However, the HSPF models were recalibrated to reflect the changes in the rainfall network. Additional minor changes were made to HSPF, including the removal of a 1.2 multiplication factor for solar radiation, and the modification of the frozen ground methodology. The removal of the solar radiation factor increased the rate of snow melt and the change in the procedures for frozen ground affected the timing, but neither revision had any significant effect on runoff volumes. A detailed land use study resulted in the modification of pervious and impervious areas within the SCAs and gaged watersheds. This resulted in slight increases in simulated runoff at the West Southwest and Calumet WRPs and slight drop at the North Side WRP. Runoff from the gaged Calumet watershed increased significantly due to the increase in impervious area.
1991	The HSPF was reformatted to use the DSS rather than TSS database system. A verification of the model was carried for WY90 that showed that this transformation did no impact the results.
1992	The Grand Calumet River streamgage at Hohman Avenue was implemented. For this year the change had no effect on runoff (the regression analysis, previously used, gave a value of zero runoff, and the gage records showed 0.4 cfs of runoff).

Table B.1 (cont') Changes Effecting Runoff Simulations

1993	Calumet river portion of the water supply pumpage from Indiana that reaches the CSSC was revised to better account for the unique hydraulics of this river. The double accounting of a portion of the runoff from the ungaged Calumet watershed was adjusted.
1994	Prepared with the WY93-94 reports, no additional changes.

Table B.2 Ratios of Simulated versus Record Flows

Water Year	North Side	[2]West Southwest	[3]Main TARP	Calumet WRP	[3]Calumet TARP	[4]Lemont WRP
[1]1983						
1984	0.97	0.99		0.89		1.02
1985	1.00	1.03		0.96		1.16
1986	0.95	1.08		0.84		1.05
1987	0.95	0.99		0.86		0.86
1988	0.97	0.93		0.80		0.82
1989	0.97	1.03		0.99		0.78
1990	0.94	1.07	1.05	1.00	.73	0.86
1991	0.92	1.02	0.92	1.00	.62	0.73
1992	0.93	1.08	1.10	1.05	.76	0.76
1993	0.95	1.07	1.06	1.06	.61	0.88
1994	0.97	1.04	1.23	1.02	.75	0.82

Notes: 1. Ratios at the WRPs are not available for WY83. However, in a comparative evaluation using the WY84-85 District models, the NIPC simulations of WY83 predicted a total ratio that is 10% to 13% too low (240 cfs).
2. The magnitude of the West Southwest WRP flows exceeds the sum of the flows from the North Side and Calumet WRPs.
3. Prior to 1990 the Main and Calumet TARP flows were included with West Southwest and Calumet WRP flows.
4. The Lemont WRP flows are insignificant (2-3 cfs).

10. To further clarify the differences generated by the recalibration in WY90, Table 3 provides the difference in flows between simulated and recorded values (a "+" value implies that the simulated is greater than the recorded). Summing the average differences for WY84-89 gives a total yearly flow of -0.4 cfs, which means that the simulation only very slightly under estimated the recorded values. Similarly, summing the average differences for WY90-94 gives a total yearly flow of +27.9 cfs, which means that simulation slightly overestimated the recorded values (1.3% of the total measured flows of 2,033.1 cfs).

Table B.3 Difference between Simulated and Record Flows (cfs)

Water Year	North Side	[2]West Southwest	[3]Main TARP	Calumet WRP	[3]Calumet TARP	[4]Lemont WRP
[1]1983						
1984	-12.6	-11.6		-39.4		0
1985	0	+33.5		-13.5		+0.3
1986	-22.2	+86.7		-59.9		+0.1
1987	-23.4	-10.5		-49.5		-0.3
1988	-14.2	-92.4		-71.9		-0.4
1989	-14.8	+35.4		-2.3		-0.5
Avg: 84-89	-29.8	+16.5		[5]-2.3		-0.1
1990	-26.1	+70.1	+4.9	+2.4	-10.4	-0.3
1991	-34.9	+27.4	-7.9	-0.4	-17.2	-0.7
1992	-28.4	+89.2	+8.8	+16.7	-8.8	-0.6
1993	-26.0	+87.2	+7.7	+25.5	-25.8	-0.3
1994	-12.2	+42.0	+20.3	+6.7	-7.2	-0.4
Avg: 90-94	-25.5	+63.2	+6.8	+10.2	-13.9	-0.5

Notes: 1-4. Notes 1-4 are identical to those from Table B.2

5. The Sanitary flows for the Calumet WRP were recalibrated in WY89. This value is more representative than an average for WY84-89.

Trend Analyses

11. The next phase of the sensitivity analysis of the period of record flows consists of a review of a series of trend analyses that compare increases over time in station rainfalls, modeled rainfalls, and modeled runoffs. Attachment B-2 lists the rainfall for each of the Midway Airport, O'Hare Airport and University of Chicago stations. The attachment also provides the basic statistics (minimums, maximums, averages and standard deviations) for each of the stations, as well as an analysis of a 5-year running average for each station. However, what are most useful from the attachment are the results of the linear regression done for each station. In the analyses the year (independent variable) was regressed with the station precipitation (dependent variable). Although the coefficients of determination for the three regressions are low, the results all suggest that the precipitation is increasing over time (positive first-order coefficients), but at a decreasing rate (negative second-order coefficients).

12. Attachment B-3 provides annual rainfall for each water year used for the gaged areas, the simulated areas and the total areas. In a manner similar to attachment B-2, this attachment also provides the basic statistics (minimums, maximums, averages and standard deviations) for each of the watersheds, as well as an analysis of a 5-year running average for each type of watershed. Likewise, what are most useful from the attachment are the results of the linear regression done for each category of watershed. In the analyses the year (independent variable) was regressed with watershed precipitation (dependent variable). Again the coefficients of determination for the three regressions are low, but the results all suggest that the annual precipitation is increasing over time

(positive first-order coefficients), but at a decreasing rate (negative second-order coefficients).

13. Listings of the watershed runoff for the gaged areas, the simulated areas and the total areas are provided in attachment B-4. Table B-4.1 lists the runoff values in cubic feet per second, and for comparison to the rainfall totals. The runoff values have been converted in Table B-4.2 to inches (based on a simulated area of 361 square miles, a gaged area of 312 square miles and a total area of 673 square miles). Overlapping areas were assigned as gaged areas. The 4 cfs of baseflow has been evenly divided between the simulated and gaged runoff values. The basic statistics and an analysis of the 5-year running averages are also included. As with the rainfall, the most important information provided in the attachment are the results of the three regression analyses of year versus runoff. These regression showed that runoff is consistent with rainfall in that it is increasing over time (positive first-order coefficients), but at a decreasing rate (negative second-order coefficients).

14. Attachment B-5 provides annual rainfall, the annual runoff and the annual losses for each water year used for the gaged areas, the simulated areas and the total areas. In a manner similar to previous attachments, this attachment also provides the basic statistics (minimums, maximums, averages and standard deviations) for each type of watershed. Likewise, what are most useful from the attachment are the results of the linear regression done for each category of watershed. In the first set of analyses the rainfall (independent variable) was regressed with the runoff (dependent variable). The coefficients of determination for the three regressions are low, but the results all suggest that the runoff is increasing with respect to rainfall over time (positive first-order coefficients), but at a decreasing rate (negative second-order coefficients). In the second set of analyses the rainfall (independent variable) was regressed with the losses (dependent variable). Again the coefficients of determination for the three regressions are low, but the results all suggest that the losses are increasing with respect to rainfall over time (generally positive first-order coefficients), and at decreasing rate (generally negative second-order coefficients). What these two sets of analyses imply is that the trend in increased urbanization impacts (less runoff for the corresponding rainfalls, as well as greater losses) is declining over time.

15. The significant point made in an evaluation of the rainfall records is that for each station, or each category of watershed, there was a small, but consistent increase in the average annual rainfall per year. Again, this small, but consistent increase also occurs in the watershed runoff records. The initial conclusion from this is that the rainfall is increasing over time. This statement is also consistent with studies performed by the Illinois State Water Survey. Whether or not the long-term average runoff should take this factor into account is dependent on if the long-term weather patterns are cyclic, and if the period analyzed here is a representative portion of a complete cycle. A second conclusion is that the models are consistent, in that station rainfall, watershed rainfall, and watershed runoff all increase slightly, on average, from year to year. The final conclusion, from the rainfall-runoff analyses) is that the suggested trend in increased urbanization impacts is declining over time.

Rainfall Gage Sensitivity Analysis

16. A sensitivity analysis was conducted to determine the impacts on runoff when there were changes in the precipitation gages used for hydrologic modeling. The sensitivity analysis was conducted for the period covering WY90-94. The precipitation sensitivity analysis includes the analysis of two additional sets of precipitation gages. In this sensitivity analysis the methods employed for the period of record runoff study were used as the basis for comparison.

17. Since the accuracy of the period of record runoff estimate is dependent on the accuracy of the precipitation gage data it is important to quantify the impact on runoff for changes in the precipitation data used in the modeling. In addition to the 3 precipitation gages (O'Hare, Midway, and University of Chicago) used for the period of record study, WY51-94, the modeling was completed using only the Midway gage (as was done in the NIPC study) and then completed using 20 of the 25 gages currently used in Lake Michigan diversion accounting. Only 20 gages of 25 are in the runoff simulation area, the other 5 are in the Des Plaines basin and not required for the period of record analysis.

18. The results of the analysis are provided in attachment B-6. The average runoff computed over the 5-year period, WY90-94, was 866.2 for the 3-gage period of record study. Using only the Midway gage resulted in increasing the average annual runoff for the 5-year period to 887.4 cfs for a 2.4 percent increase. Using 20 of the 25 gages currently employed in the accounting of Lake Michigan diversion resulted in increasing the average annual runoff for the 5-year period to 916.0 cfs for a 5.6 percent increase. The annual runoff for each gage network simulation is also provided in the attachment.

Imperviousness Sensitivity Analysis

19. A sensitivity analysis was conducted to determine the impacts on runoff when there were changes in the impervious areas of the 137 special contributing areas (SCAs) that have been hydraulically modeled. The sensitivity analysis was conducted for the period covering WY90-94. The imperviousness sensitivity analysis included an analysis of both increasing and decreasing the impervious areas by 10 percent for each of the SCAs. In this sensitivity analysis, as was the case for the precipitation sensitivity analysis, the methods employed for the period of record runoff study were used as the basis for comparison.

20. The period of record runoff analysis used the same impervious and pervious areas of the SCAs as modeled for diversion accounting from WY90-92. The amount of impervious and pervious areas modeled for each SCA are derived from 1990 aerial photographs. Each SCA is further subdivided into various land types categories based on the aerials. Each land type classification has associated with it assumed pervious and

impervious percentages. From this information the total impervious and pervious percentages of each SCA have been determined for use in diversion accounting. Consequently, there are potentials for variations in these values due to the subjective land use classifications and the assumed perviousness of each classification. Therefore, a sensitivity analysis that quantifies the impact of modifying the perviousness of each SCA was deemed important. It was decided that the largest expected error in the impervious area for any one SCA was 10 percent. Therefore, sensitivity analyses were run for both a 10 percent increase and a 10 percent decrease in impervious area for each SCA. For example, if one particular SCA has a total area of 3 square miles, 1 of which is impervious paved areas and 2 of which is pervious grass, increasing the imperviousness by 10 percent results in adding 0.1 square mile to the impervious area (1.1 square mile) and subtracting 0.1 square mile from the pervious area (1.9 square mile).

21. The results of this sensitivity analysis are provided in attachment B-7. The average runoff computed for the 5-year period, WY90-94, using the impervious and pervious breakdowns applied in the period of record study was 866.2 cfs. Increasing the impervious areas by 10 percent resulted in increasing the average annual runoff for the 5-year period to 885.9.0 cfs for a 2.3 percent increase. Decreasing the impervious areas by 10 percent resulted in decreasing the average annual runoff for the 5-year period to 846.5 cfs for a 2.3 percent decrease. The annual runoff for each level of imperviousness is also provided in the attachment.

Comparison of Period of Record Runoff with Lockport Record

22. As a method of evaluating the period of record runoff analysis, the results can be compared to the long-term record at Lockport. Further, a more precise comparison can be made if the runoff is compared to AVM flows estimated from the record at Lockport. Additionally, other information that can be readily derived from the analysis, such as the long-term diversion of Lake Michigan water, can also be presented. The following paragraphs outline the procedures used in the comparisons and other analyses.

23. The procedures used in this analysis consist of the following steps:

- Document the latest USGS AVM flows.
- Correct the Lockport flows so that all turbine flows are based on the turbine rating curves.
- Develop regression equations that can be used to calculate estimated AVM flows based solely on total Lockport flows.
- Use stepwise regression to relate diverted flows to estimated AVM and period of record runoff.

- Estimate diverted flows for WY51-95 using estimated AVM flows and period of record runoff.

- Plot Lockport versus estimated AVM, the ratio of period of record runoff to Lockport flow, the ratio of period of record runoff to the estimated AVM flows, the long term diverted flows and the long term deviations.

24. The average annual USGS AVM flows for WY87-95 are presented in the Table B.4. Note that the flows for WY93-95 are provisional. It should also be noted that values include gage records only, i.e. missing days have not been filled in using the regression equations. Plots of the daily and annual flows are provided in attachment B-8.

Table B.4 Average Annual USGS AVM Flows

Water Year	Annual USGS AVM Flow (cfs)
1987	4,023
1988	3,628
1989	3,487
1990	3,601
1991	3,685
1992	3,720
1993	4,118
1994	3,086
1995	3,268
Average:	3,625

25. For the period November 17, 1992 through July 31, 1994 the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) reported the flows through the turbines at Lockport using measurements from AVMs installed in the turbines. For all other periods during WY51-95, the flows were reported using the turbine power rating curves (see the plots of the ratios and differences of turbines plus leakages plus lockages (TLL) flows versus the AVM flows at Romeoville in attachment B-9). To build a consistent period of record analysis, the turbine flows were adjusted to rating table values using two regression equations: the first relates the turbine rating values to the Romeoville AVM values; and the second relates the turbine AVM values to the Romeoville AVM values. Eliminating the Romeoville AVM values from the two equations gives an equation for estimating the turbine ratings values from the turbine AVM values (see Table B-9.1 in attachment B-9).

26. With all turbine values given in terms of the power rating curves, total Lockport flows can be generated for each day in WY84-95. Using the USGS regression equations for filling in flows for missing days, total Romeoville AVM records can also be computed for the same period of time. Table B-10.1 in attachment B-10 shows the total MWRDGC and AVM flows. The attachment also provides plots of Lockport flows

versus USGS flows (not counting missing days), and Lockport flows versus AVM flows (including regression results for missing days).

27. The USGS regression equations are very accurate at translating Lockport flows into AVM flows. However, the USGS equations require a breakdown of the Lockport record into turbine, sluice gate and controlling works flows. Unfortunately, prior to 1984 the only Lockport flows that were available were for the total flow. Therefore, the next step of the analysis is to develop a series of regression equations that can be used to estimate Romeoville AVM flows based solely on total Lockport flows. The procedure is to first prepare a frequency analysis of Lockport flows versus TLL, sluice gate and controlling works flow patterns (see Table B-11.1 in attachment B-11). Once the frequency analysis is complete, a regression equation can be developed for each range of similar flows (see Table B-11.2 in the attachment, as well as the plots of each range). It should be noted that the regression equations in the mid-range (in particular the equation for 4,000 cfs to 5,000 cfs) are not very accurate, because of the variability of the flows (i.e. turbine only, sluice and turbine, and controlling works). However, while it is true that some of the regression equations are questionable, the overall analysis should produce a reasonable estimation of the overall AVM flow. This is because the majority of the flows are small values, where the regression equations produce good results. Additionally, the annual average errors in predicting runoff are relatively small (see percent errors in Table B-11.2 of the attachment).

28. Preliminary diversion accounting results for WY93-95 are displayed, using the standard format, in Table B-12.1 in attachment B-12. As noted in the table, the accounting results are based on the provisional USGS AVM flows, average values of the total deductions (for WY86-92) and the WY 92 value of the by-passed flows. The WY92 by-passed flow was used because it more closely matches the increased allocation of Lake Michigan water to DuPage and Lake Counties (i.e. counties in which the treatment plants discharge to the Des Plaines River). It is noted that both the total deductions and by-passed flows are probably under estimated. However, the aggregate of the two (diversion accounting = AVM - deductions + by-passed flows) should give a reasonable value. Preliminary values of Illinois water supply pumpage and direct diversions (lockages, navigation makeup and discretionary flow) are available for WY93-94, and the values for WY95 are based on averages of WY86-94. Leakage estimates have been prepared as part of the Error Analysis (see appendix D). The watershed runoff for WY93-94 can be computed using a regression analysis with the period of record flows, and the watershed runoff for WY 95 can be computed using a regression analysis with O'Hare and Midway rainfall values (again see appendix D). Using these computations, the imbalances for WY93-94 can also be computed and are shown in Table B-12.1. Finally, Table B-12.2 in the attachment gives the results of a regression of the flows accountable to the State of Illinois versus the estimated AVM flows and the period of record runoff.

29. The last step to consider is the comparison of period of record runoff values versus the Lockport flows and period of record runoff values versus the estimated AVM flows. Table B-13.1 in attachment B-13 provides a listing of the annual Lockport flows and the

annual AVM flows estimated using the procedure detailed in the paragraph above (Figure B-13.1 in the attachment shows this relationship). The ratios of period of record runoff to Lockport and to the estimated AVM flows are also shown in the table, and plotted in Figures B-13.2 and B-13.3 in the attachment. Finally, in completing the analysis, the long term diverted flows (estimated using the procedures in the above paragraph) and the long-term deviations (cumulative sum of the diverted flows minus 3,200 cfs) are also shown in Table B-13.1 and graphed in Figures B-13.4 and B-13.5 of the attachment.

Mass Balance of Rainfall and Runoff Components

30. To provide insight into the various paths that rainfall can take, a mass balance of rainfall and runoff components was done over the period WY90-94. As can be seen in Table B-14.1 of attachment B-14 the average rain during the five year period was 37.38 inches. This is a weighted average of the 3 precipitation gages used for the runoff study. Using this rainfall the average flow into the 673 square miles of Lake Michigan watershed can be expressed in terms as an average flow is 1852.3 cfs. The two primary elements of the mass balance that are not part of the runoff are subsurface infiltration that percolates all the way down to the deep aquifer and the total evapotranspiration. The total evapotranspiration includes evaporation from the soil or impervious surfaces and transpiration through plants. Over the 5-year period the total flow lost to the deep aquifer was 55.6 cfs or 3.0 percent of the total rainfall while the total evapotranspiration was 926.5 cfs or 50.0 percent of the total rainfall. In terms of sources and sinks the following comparison can be made:

Component	Source	Sink
Rainfall	1852.3 cfs	
Total Runoff		870.2 cfs
Deep Aquifer		55.6 cfs
Evapotranspiration		926.5 cfs

31. The total runoff during the 5-year period was 870.2 cfs or 47.0 percent of the total rainfall. The five components of the runoff shown in Table B-14.2 in the attachment are stream gage runoff, sewer runoff, sewer overflows, runoff from the ungaged Calumet watershed, and baseflow downstream of the diversion stream gages. Stream gage runoff is determined from streamflow separation techniques where runoff is computed by subtracting upstream sanitary discharges from the recorded stream gage records. Sewer runoff is the portion of the inflow and infiltration (I&I) that is conveyed to treatment plants via sewer interceptors, while sewer overflows is that portion of I&I that overflows from the sewers to adjoining rivers or is intercepted by the TARP tunnels. The ungaged Calumet watershed is an 84 square mile area that is modeled separately since it is contains separate storm and sanitary sewer systems and since it is downstream of all diversion accounting stream gages. The baseflow is the component of the subsurface flow that enters the canals and rivers downstream of the gages. In terms of sources and sinks the following comparison can be made:

Component	**Source**	**Sink**
Total Runoff	870.2 cfs	
Gaged Runoff		315.1 cfs
Sewer Runoff		238.2 cfs
Overflow		215.8 cfs
Ungaged Runoff		103.4 cfs
Baseflow		4.0 cfs

32. To further clarify the total flow reaching and exiting from the sewers, Table B-14.3 in the attachment provides a further breakdown (note: the overflow values shown for WY 91-94 do not include overflows upstream of gages). The first half of the table shows the inflows to the sewers, either from overland flow (inflow) or from subsurface flow (infiltration). The second half of the table shows the outflows from the sewers, either to the water reclamation plants or to TARP or the canals and rivers. In terms of sources and sinks the following comparison can be made:

Component	**Source**	**Sink**
Inflows	338.5 cfs	
Infiltration	115.6 cfs	
Reclamation Plants		238.2 cfs
TARP or Canal		215.8 cfs

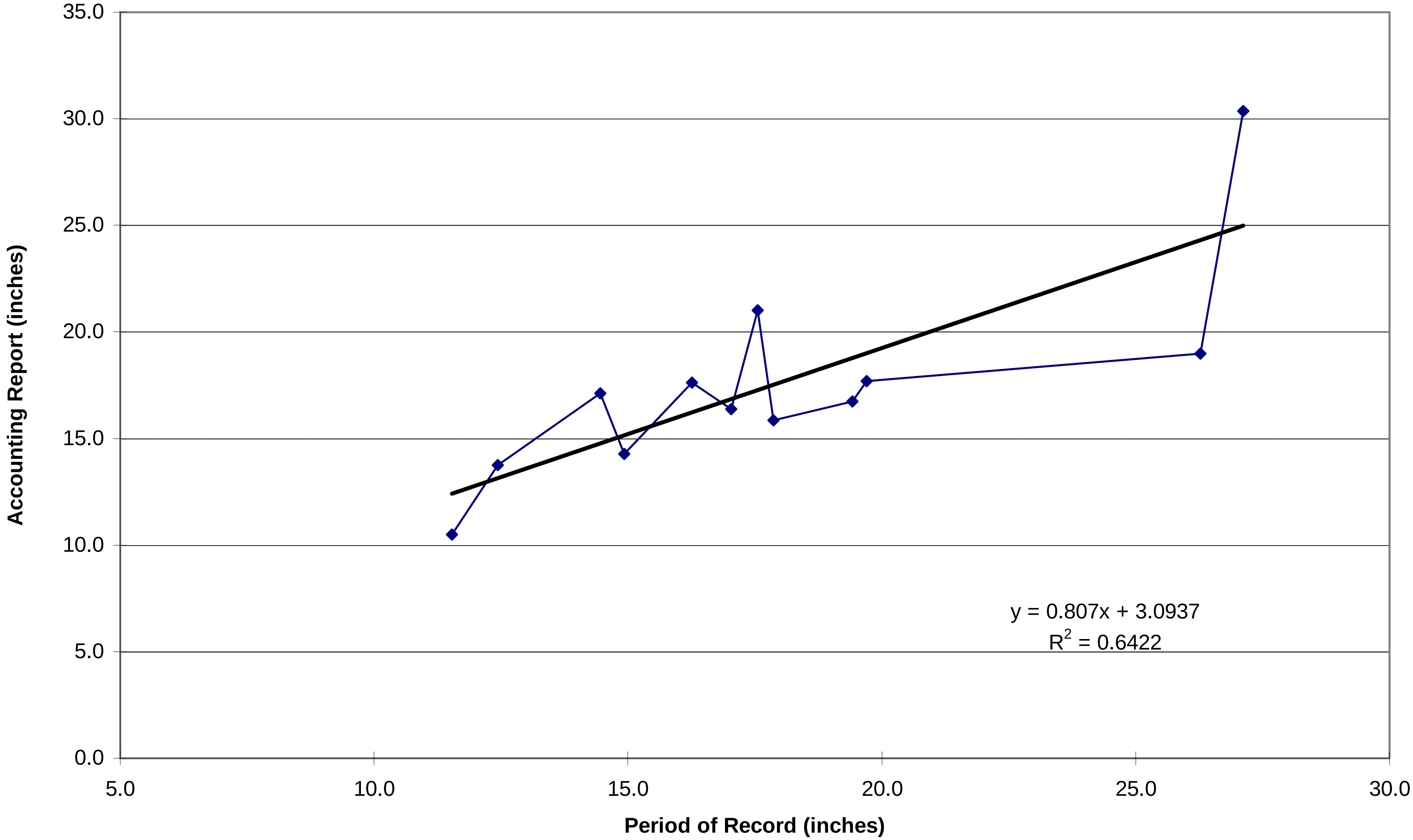
Attachment B-1

Comparison of Period of Record
Versus
Accounting Report Runoff

Table B-1.1 Comparison of Period of Record versus Accounting Report				
Water Year	Period of Record Runoff (cfs)	Accounting Report Runoff (cfs)	Period of Record Runoff (inches)	Accounting Report Runoff (inches)
1983	1301.6	940.8	26.3	19.0
1984	962.4	829.0	19.4	16.7
1985	885.0	785.5	17.9	15.9
1986	975.9	876.5	19.7	17.7
1987	843.9	811.7	17.0	16.4
1988	571.7	519.6	11.5	10.5
1989	739.3	706.8	14.9	14.3
1990	805.8	872.9	16.3	17.6
1991	869.7	1041.4	17.6	21.0
1992	715.9	848.4	14.5	17.1
1993	1343.4	1504.7	27.1	30.4
1994	616.4	681.1	12.4	13.7
Minimum	571.7	519.6	11.5	10.5
Maximum	1343.4	1504.7	27.1	30.4
Average	885.9	868.2	17.9	17.5
Stan Deviation	238.6	240.3	4.8	4.8
Statistical Comparison				
- T-test		0.69		
- Correlation Coefficient		0.80		

Figure B-1.1 Water Year versus Runoff

Figure B-1.2 Period of Record versus Accounting Report



Attachment B-2

Station Rainfall

Table B-2.1 Station Rainfall

Water	Midway	O'Hare	Unv Chic					
Year	(inches)	(inches)	(inches)					
1951	39.54	34.83	38.61		5-Year Running Average			
1952	32.41	29.40	30.61					
1953	29.15	25.21	30.05		Water	Midway	O'Hare	Unv Chic
1954	36.39	32.29	36.15		Year	(inches)	(inches)	(inches)
1955	39.26	31.20	38.75		1955	35.35	30.59	34.83
1956	26.71	27.08	26.63		1956	32.78	29.04	32.44
1957	39.34	40.24	36.60		1957	34.17	31.20	33.64
1958	29.90	28.81	30.01		1958	34.32	31.92	33.63
1959	35.08	27.74	34.04		1959	34.06	31.01	33.21
1960	31.60	34.41	30.61		1960	32.53	31.66	31.58
1961	39.92	32.56	35.67		1961	35.17	32.75	33.39
1962	26.49	26.04	26.68		1962	32.60	29.91	31.40
1963	28.20	24.09	26.56		1963	32.26	28.97	30.71
1964	26.21	27.95	26.50		1964	30.48	29.01	29.20
1965	38.80	37.29	40.79		1965	31.92	29.59	31.24
1966	33.47	29.58	32.04		1966	30.63	28.99	30.51
1967	40.38	35.56	36.53		1967	33.41	30.89	32.48
1968	31.35	32.71	33.51		1968	34.04	32.62	33.87
1969	38.47	33.08	38.02		1969	36.49	33.64	36.18
1970	43.15	45.20	42.60		1970	37.36	35.23	36.54
1971	32.06	27.19	32.98		1971	37.08	34.75	36.73
1972	37.39	44.02	40.24		1972	36.48	36.44	37.47
1973	40.30	38.89	38.43		1973	38.27	37.68	38.45
1974	40.38	36.95	42.21		1974	38.66	38.45	39.29
1975	41.47	38.19	45.49		1975	38.32	37.05	39.87
1976	36.33	31.34	33.21		1976	39.17	37.88	39.92
1977	38.58	29.87	38.40		1977	39.41	35.05	39.55
1978	32.55	34.99	33.32		1978	37.86	34.27	38.53
1979	38.36	37.96	33.42		1979	37.46	34.47	36.77
1980	30.85	38.91	36.28		1980	35.33	34.61	34.93
1981	38.47	40.50	37.93		1981	35.76	36.45	35.87
1982	36.98	32.60	33.59		1982	35.44	36.99	34.91
1983	51.00	53.47	52.27		1983	39.13	40.69	38.70
1984	42.67	38.56	35.75		1984	39.99	40.81	39.16
1985	39.80	34.09	36.56		1985	41.78	39.84	39.22
1986	44.55	40.17	38.47		1986	43.00	39.78	39.33
1987	35.60	39.47	33.65		1987	42.72	41.15	39.34
1988	27.84	22.73	28.09		1988	38.09	35.00	34.50
1989	37.40	39.24	37.97		1989	37.04	35.14	34.95
1990	39.60	35.59	37.99		1990	37.00	35.44	35.23
1991	37.68	34.00	36.46		1991	35.62	34.21	34.83
1992	35.93	33.09	34.55		1992	35.69	32.93	35.01
1993	51.30	49.88	47.46		1993	40.38	38.36	38.89
1994	26.07	27.45	30.48		1994	38.12	36.00	37.39
Minimum	26.07	22.73	26.50		Minimum	30.48	28.97	29.20
Maximum	51.30	53.47	52.27		Maximum	43.00	41.15	39.92
Average	36.34	34.42	35.59		Average	36.39	34.51	35.59
Stan Dev	5.95	6.55	5.49		Stan Dev	3.09	3.57	3.02

Figure B-2.1 Midway Airport Rainfall

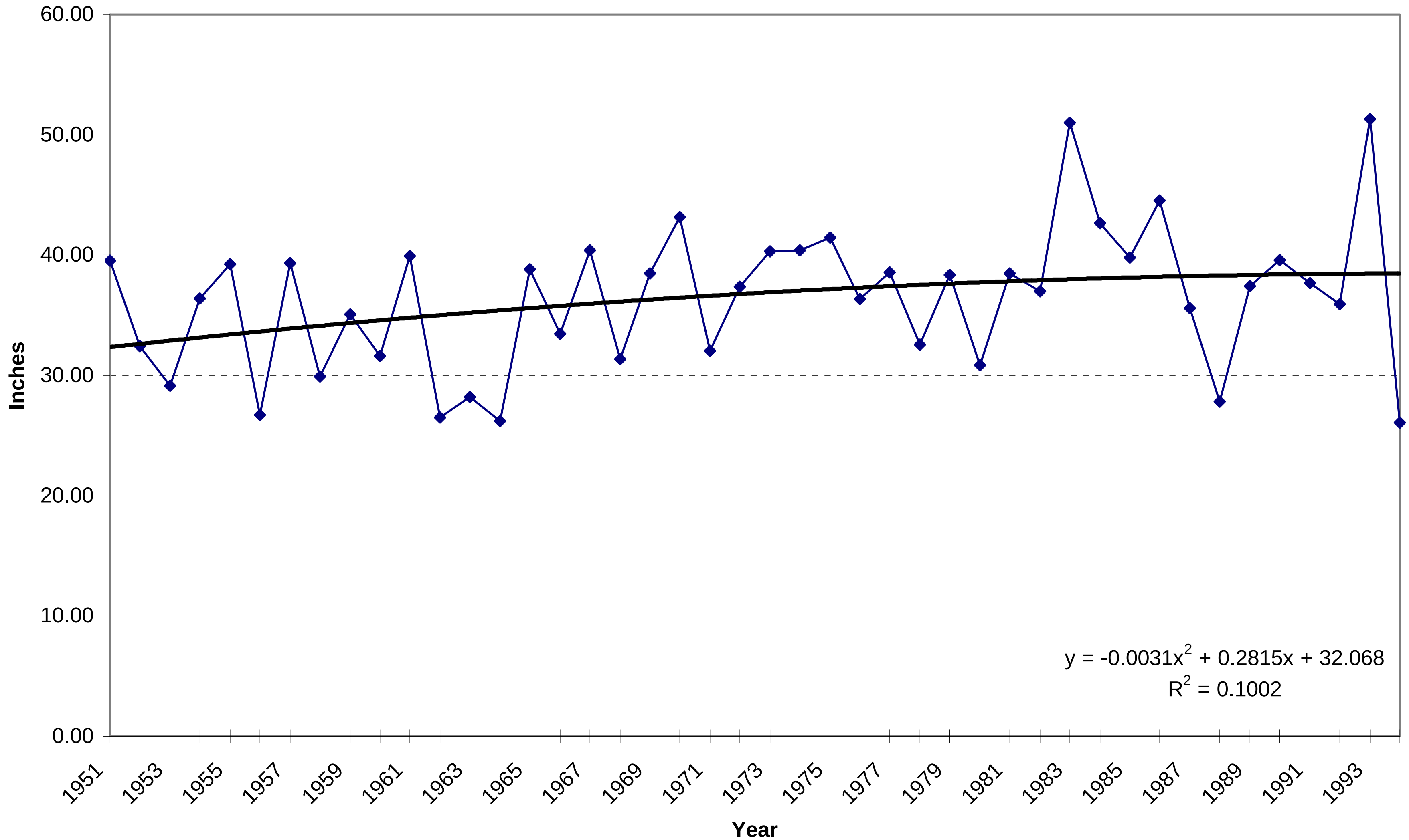


Figure B-2.2 O'Hare Airport Rainfall

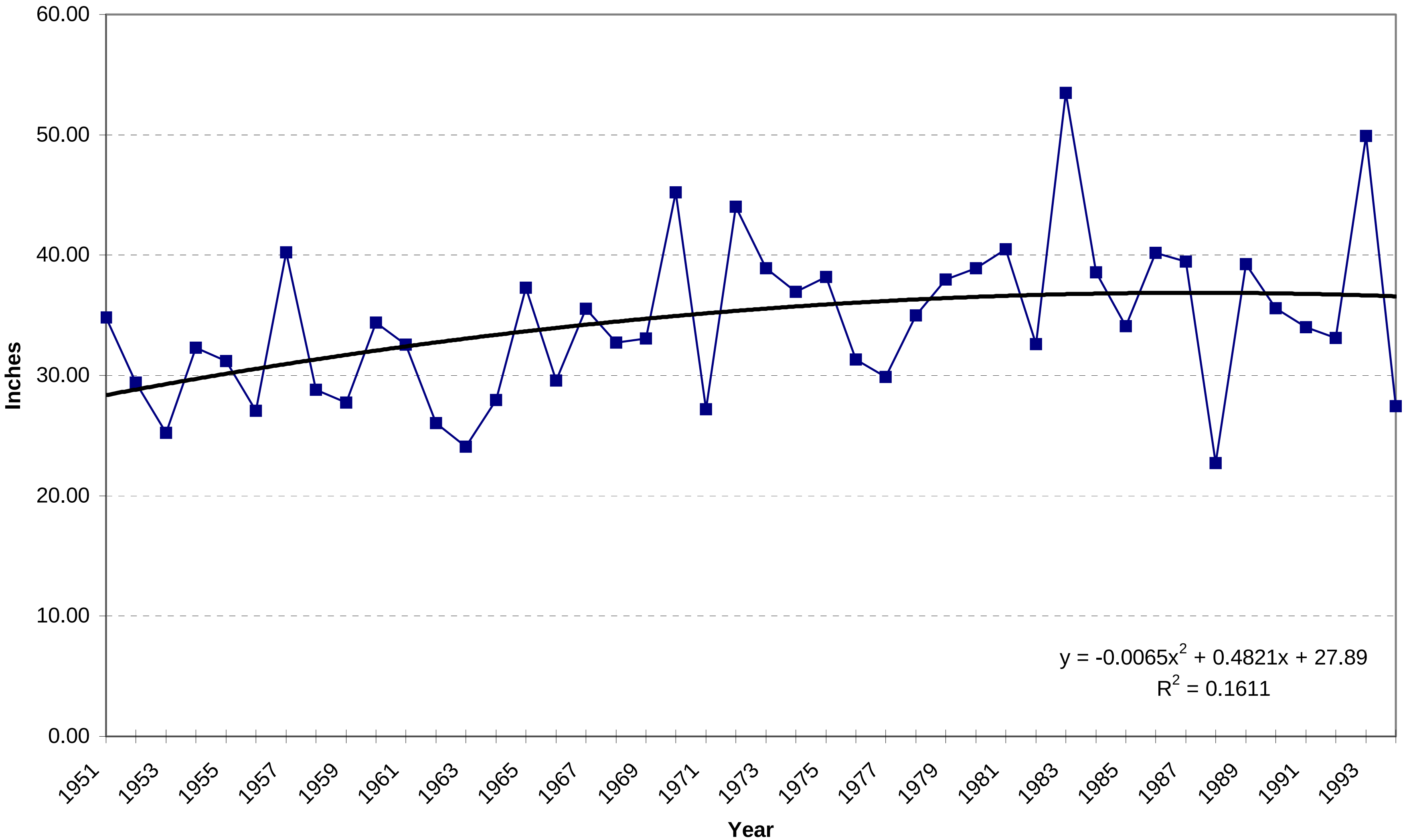


Figure B-2.3 University of Chicago Rainfall

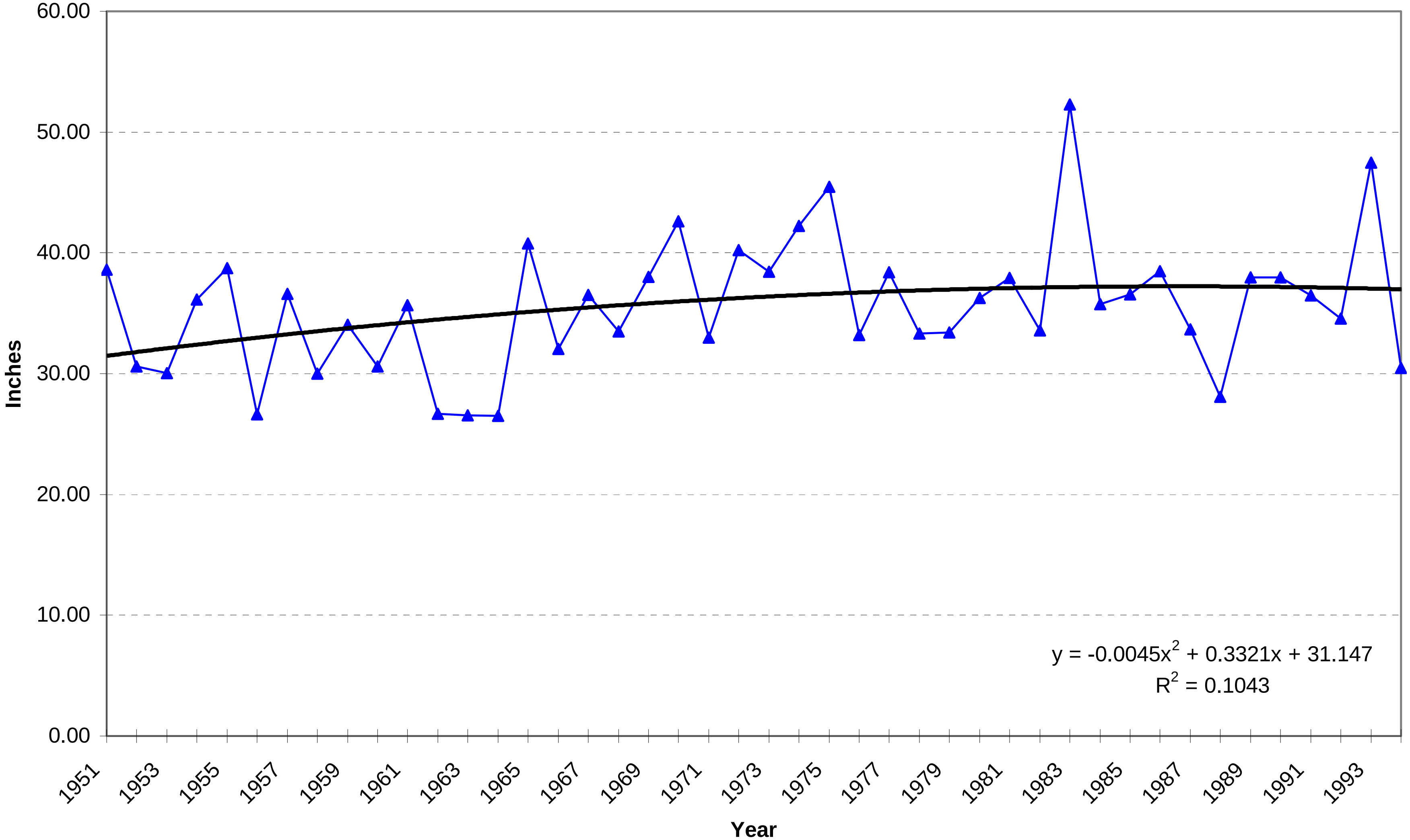


Figure B-2.4 Midway: 5-Year Average Rainfall

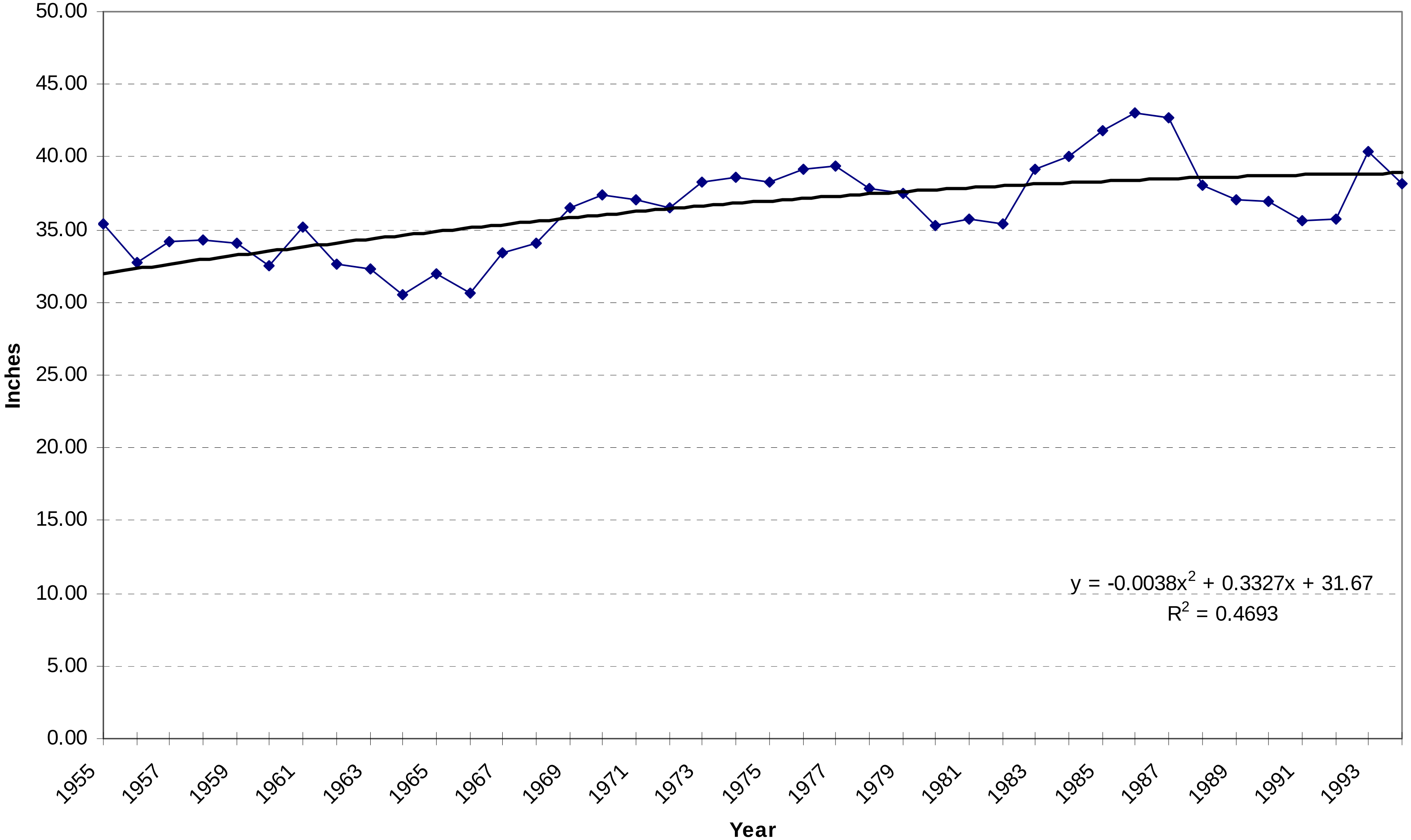


Figure B-2.5 O'Hare: 5-Year Average Rainfall

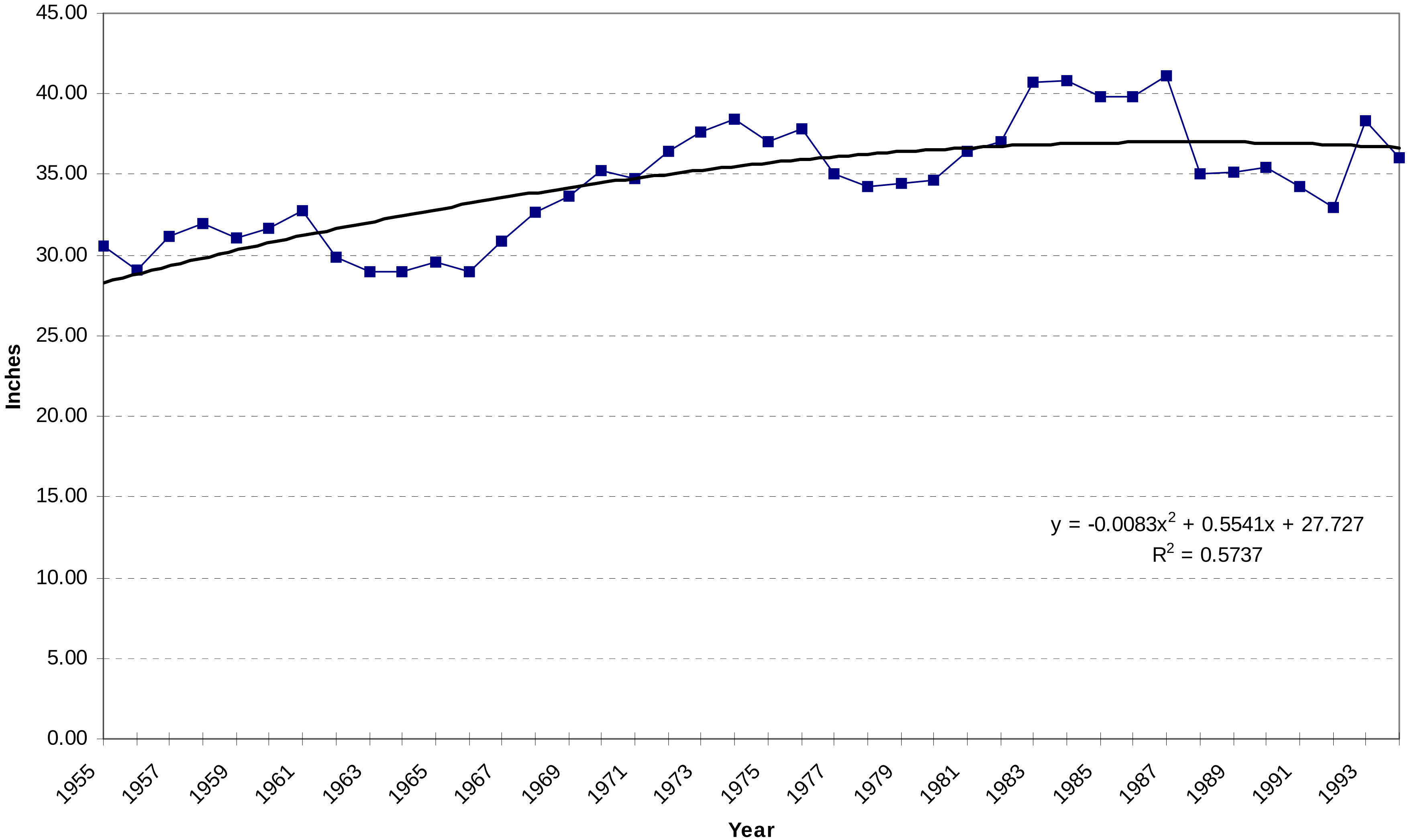
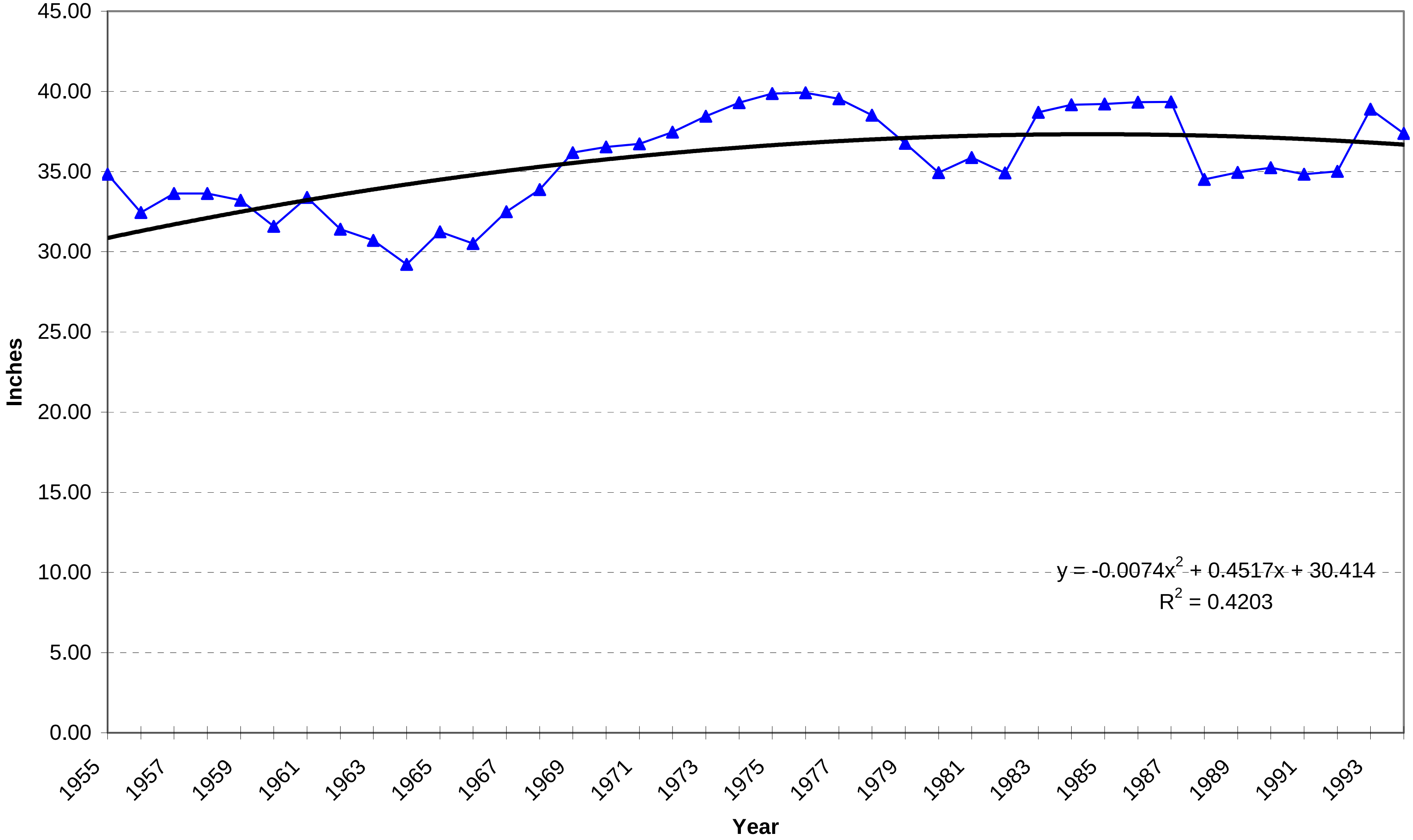


Figure B-2.6 University of Chicago: 5-Year Average Rainfall



Attachment B-3

Watershed Rainfall

Table B-3.1 Watershed Rainfall

Water Year	Gaged (inches)	Simulated (inches)	Total (inches)
1951	37.91	38.20	38.12
1952	31.00	31.18	31.13
1953	28.30	28.55	28.48
1954	35.15	35.40	35.33
1955	36.81	37.30	37.16
1956	26.79	26.77	26.77
1957	38.76	38.69	38.71
1958	29.62	29.69	29.67
1959	32.67	33.11	32.99
1960	32.10	31.92	31.97
1961	36.52	36.95	36.83
1962	26.42	26.45	26.44
1963	26.53	26.77	26.70
1964	26.79	26.69	26.72
1965	38.98	39.08	39.05
1966	31.92	32.15	32.09
1967	37.82	38.10	38.02
1968	32.40	32.33	32.35
1969	36.80	37.12	37.03
1970	43.57	43.44	43.47
1971	30.95	31.26	31.17
1972	40.15	39.76	39.87
1973	39.32	39.40	39.38
1974	39.96	40.18	40.12
1975	41.77	41.99	41.93
1976	33.95	34.24	34.16
1977	36.04	36.57	36.43
1978	33.48	33.34	33.38
1979	36.73	36.73	36.73
1980	34.81	34.34	34.48
1981	38.88	38.76	38.79
1982	34.69	34.94	34.87
1983	52.09	51.95	51.99
1984	39.37	39.60	39.53
1985	37.18	37.51	37.42
1986	41.44	41.68	41.61
1987	36.10	35.86	35.93
1988	26.46	26.77	26.69
1989	38.10	37.99	38.02
1990	37.96	38.20	38.13
1991	36.26	36.48	36.41
1992	34.70	34.86	34.82
1993	49.72	49.79	49.77
1994	27.82	27.75	27.77
Minimum	26.42	26.45	26.44
Maximum	52.09	51.95	51.99
Average	35.56	35.68	35.65
Stan Dev	5.68	5.67	5.67

5-Year Running Average

Water Year	Gaged (inches)	Simulated (inches)	Total (inches)
1955	33.83	34.12	34.04
1956	31.61	31.84	31.77
1957	33.16	33.34	33.29
1958	33.42	33.57	33.53
1959	32.93	33.11	33.06
1960	31.99	32.04	32.02
1961	33.93	34.07	34.03
1962	31.47	31.62	31.58
1963	30.85	31.04	30.99
1964	29.67	29.76	29.73
1965	31.05	31.19	31.15
1966	30.13	30.23	30.20
1967	32.41	32.56	32.52
1968	33.58	33.67	33.65
1969	35.58	35.76	35.71
1970	36.50	36.63	36.59
1971	36.31	36.45	36.41
1972	36.77	36.78	36.78
1973	38.16	38.20	38.19
1974	38.79	38.81	38.80
1975	38.43	38.52	38.49
1976	39.03	39.11	39.09
1977	38.21	38.48	38.40
1978	37.04	37.26	37.20
1979	36.39	36.57	36.52
1980	35.00	35.05	35.03
1981	35.99	35.95	35.96
1982	35.72	35.62	35.65
1983	39.44	39.35	39.37
1984	39.97	39.92	39.93
1985	40.44	40.55	40.52
1986	40.95	41.14	41.09
1987	41.24	41.32	41.30
1988	36.11	36.28	36.24
1989	35.86	35.96	35.93
1990	36.01	36.10	36.08
1991	34.98	35.06	35.04
1992	34.70	34.86	34.81
1993	39.35	39.46	39.43
1994	37.29	37.42	37.38
Minimum	29.67	29.76	29.73
Maximum	41.24	41.32	41.30
Average	35.61	35.72	35.69
Stan Dev	3.11	3.09	3.10

Figure B-3.1 Gaged Watershed - Rainfall

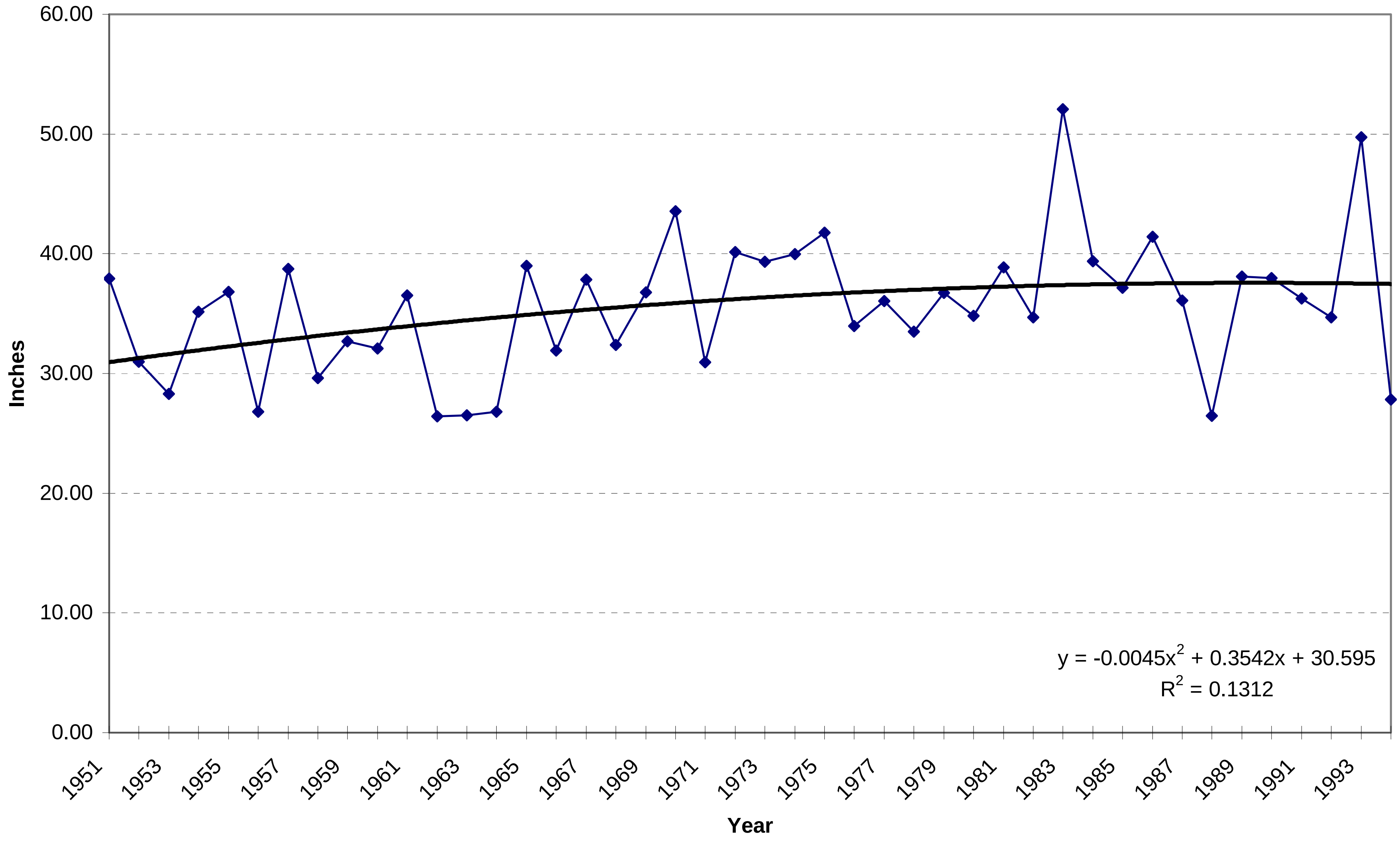


Figure B-3.2 Simulated Watershed - Rainfall

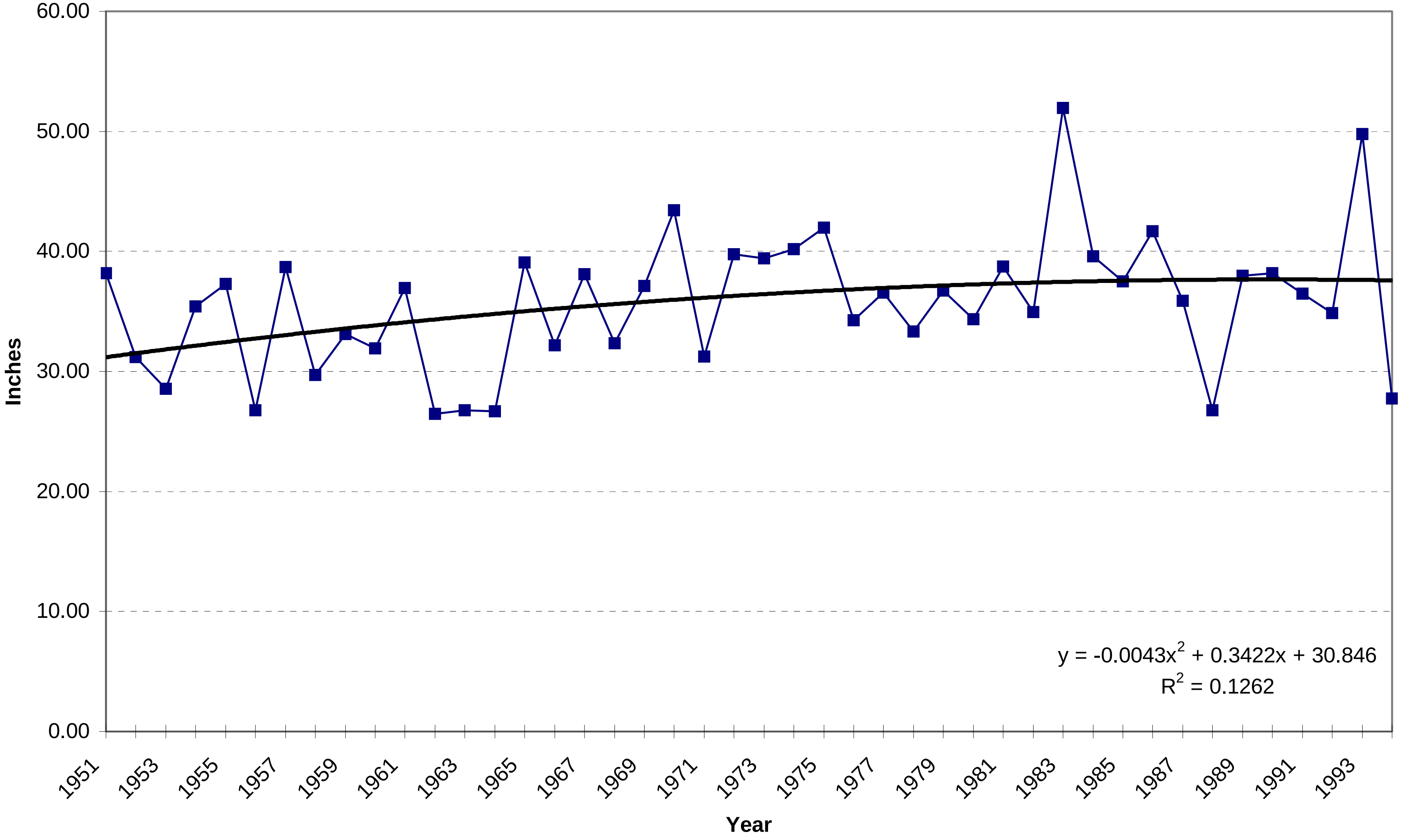


Figure B-3.3 Total Watershed - Rainfall

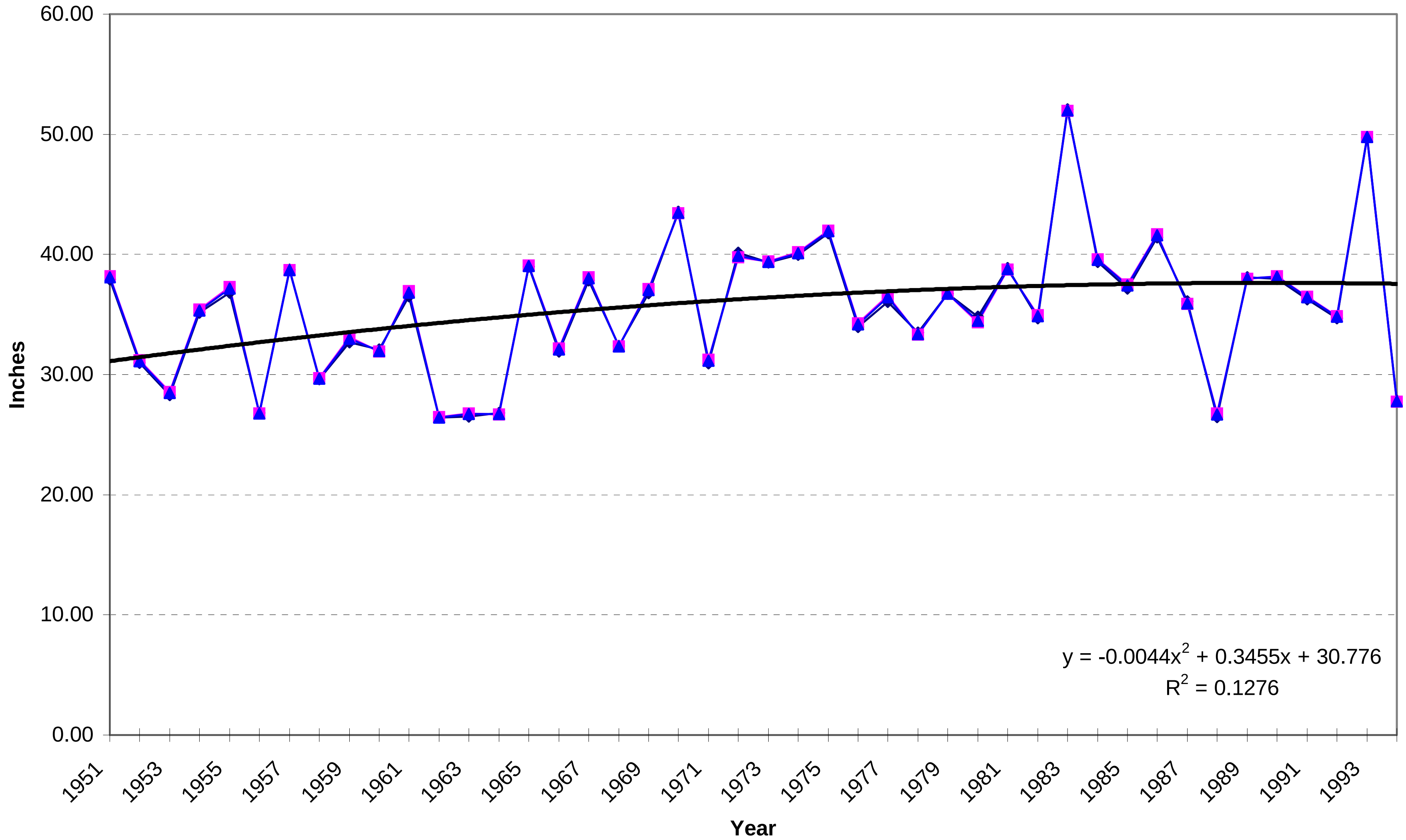


Figure B-3.4 Gaged Watershed: 5-Year Average Rainfall

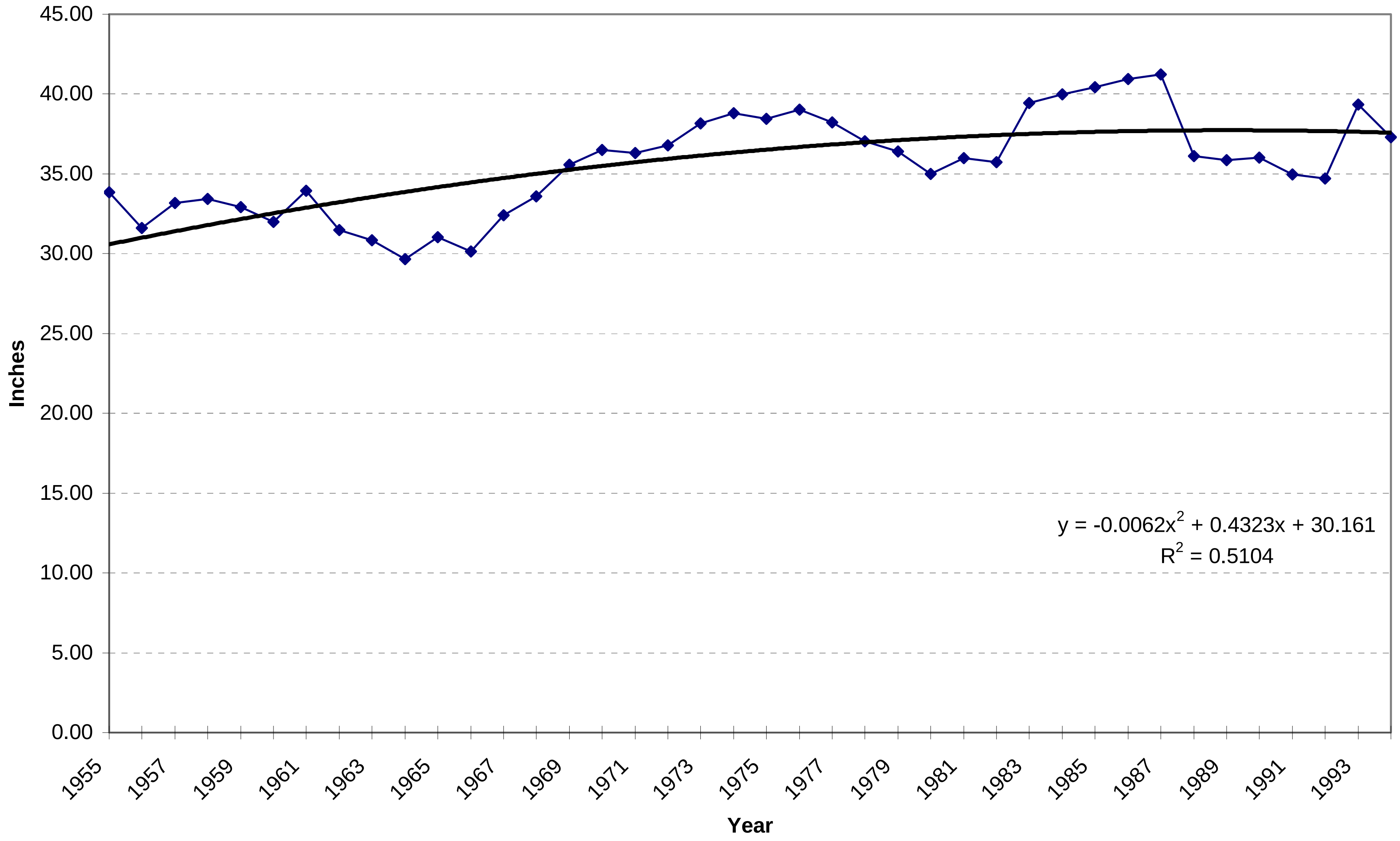


Figure B-3.5 Simulated Watershed: 5-Year Average Rainfall

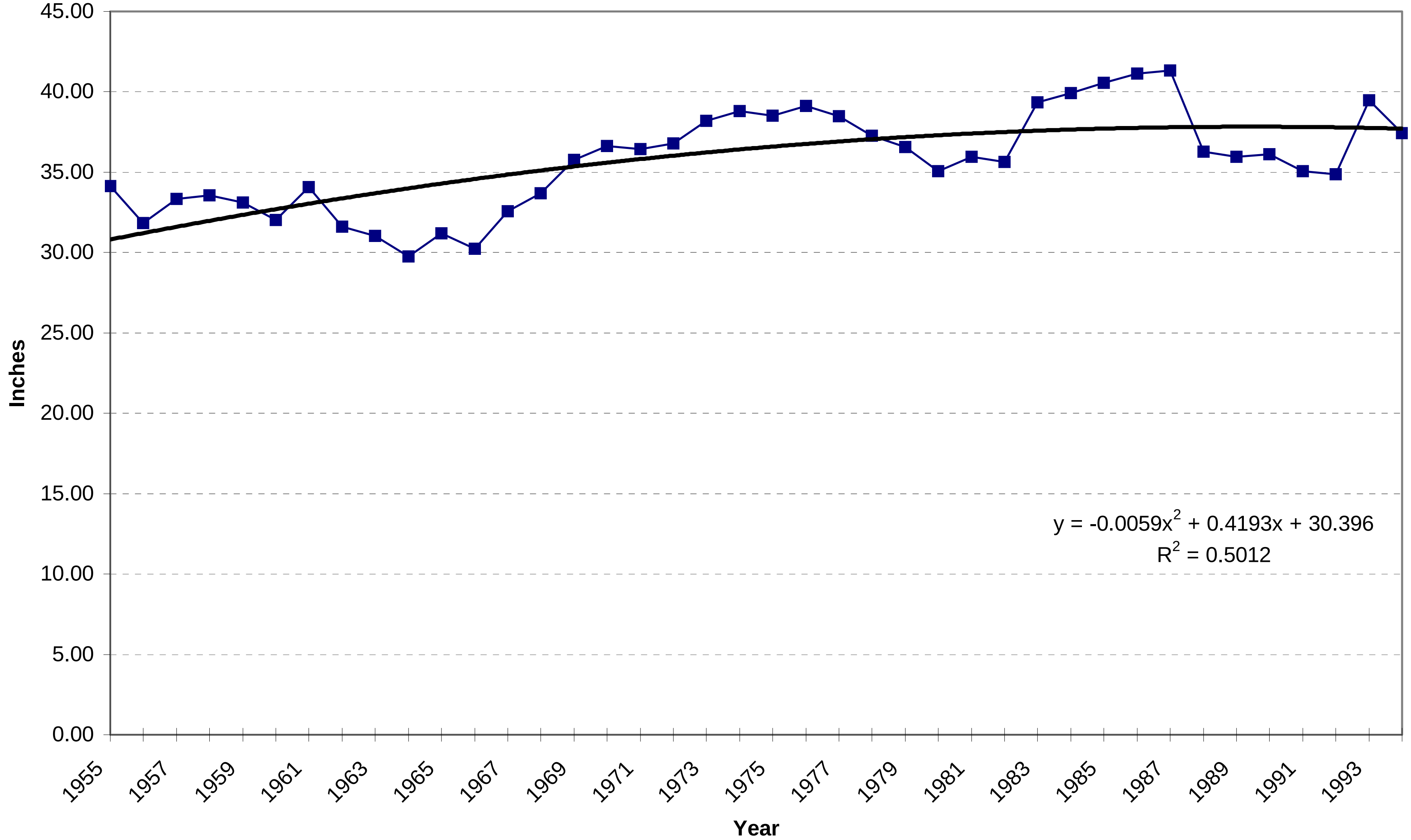
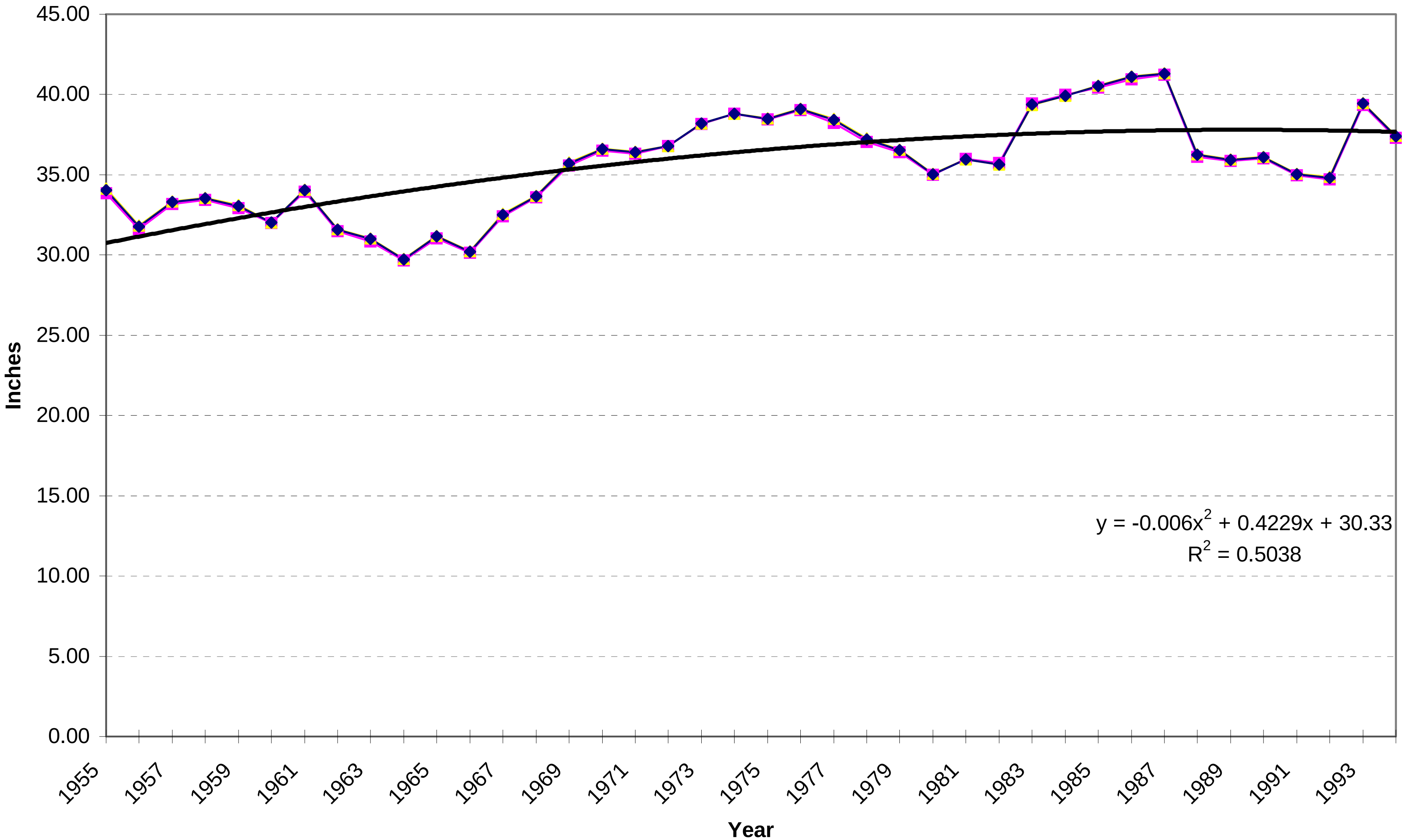


Figure B-3.6 Total Watershed: 5 Year Average



Attachment B-4

Watershed Runoff

Table B-4.1 Watershed Runoff (in cfs)

Water	Gaged	Simulated	Total					
Year	(cfs)	(cfs)	(cfs)					
1951	284.0	553.5	837.5		5-Year Running Average			
1952	358.5	462.7	821.2					
1953	176.8	318.6	495.5		Water	Gaged	Simulated	Total
1954	189.1	418.8	607.9		Year	(cfs)	(cfs)	(cfs)
1955	291.0	510.3	801.3		1955	259.9	452.8	712.7
1956	160.6	292.7	453.2		1956	235.2	400.6	635.8
1957	206.4	519.8	726.3		1957	204.8	412.0	616.8
1958	164.5	341.0	505.5		1958	202.3	416.5	618.8
1959	235.1	414.4	649.5		1959	211.5	415.6	627.1
1960	307.2	456.6	763.8		1960	214.8	404.9	619.7
1961	166.4	443.1	609.5		1961	215.9	435.0	650.9
1962	228.4	392.7	621.1		1962	220.3	409.5	629.9
1963	75.8	245.0	320.8		1963	202.6	390.4	592.9
1964	94.7	277.0	371.7		1964	174.5	362.9	537.4
1965	252.5	574.0	826.6		1965	163.6	386.4	549.9
1966	275.9	536.6	812.4		1966	185.5	405.1	590.5
1967	235.5	586.7	822.1		1967	186.9	443.9	630.7
1968	226.0	451.2	677.2		1968	216.9	485.1	702.0
1969	300.8	560.7	861.5		1969	258.1	541.8	800.0
1970	291.2	660.7	951.8		1970	265.9	559.2	825.0
1971	230.4	437.6	668.0		1971	256.8	539.4	796.1
1972	347.5	559.5	907.0		1972	279.2	534.0	813.1
1973	494.0	679.1	1173.0		1973	332.8	579.5	912.3
1974	474.6	689.7	1164.3		1974	367.5	605.3	972.8
1975	372.4	660.1	1032.4		1975	383.8	605.2	989.0
1976	332.8	464.4	797.2		1976	404.3	610.6	1014.8
1977	139.9	389.6	529.6		1977	362.7	576.6	939.3
1978	295.3	456.4	751.7		1978	323.0	532.0	855.0
1979	342.6	602.7	945.3		1979	296.6	514.6	811.2
1980	299.0	411.8	710.8		1980	281.9	465.0	746.9
1981	348.4	490.7	839.1		1981	285.0	470.3	755.3
1982	326.7	536.8	863.5		1982	322.4	499.7	822.1
1983	424.2	877.4	1301.6		1983	348.2	583.9	932.1
1984	355.0	607.4	962.4		1984	350.6	584.8	935.5
1985	311.9	573.1	885.0		1985	353.2	617.1	970.3
1986	378.5	597.4	975.9		1986	359.3	638.4	997.7
1987	345.9	498.0	843.9		1987	363.1	630.7	993.8
1988	217.9	353.8	571.7		1988	321.8	525.9	847.8
1989	243.4	495.9	739.3		1989	299.5	503.6	803.2
1990	279.4	526.4	805.8		1990	293.0	494.3	787.3
1991	343.1	526.6	869.7		1991	285.9	480.1	766.1
1992	237.9	478.0	715.9		1992	264.3	476.1	740.5
1993	484.2	859.2	1343.4		1993	317.6	577.2	894.8
1994	241.1	375.2	616.4		1994	317.2	553.1	870.2
Minimum	75.8	245.0	320.8		Minimum	163.6	362.9	537.4
Maximum	494.0	877.4	1343.4		Maximum	404.3	638.4	1014.8
Average	281.5	503.7	785.2		Average	279.7	503.0	782.7
Stan Dev	94.9	133.8	219.0		Stan Dev	65.1	79.0	140.9

Figure B-4.2 Watershed Runoff

Water Year	Gaged (inches)	Simulated (inches)	Total (inches)
1951	12.4	20.8	16.9
1952	15.6	17.4	16.6
1953	7.7	12.0	10.0
1954	8.2	15.8	12.3
1955	12.7	19.2	16.2
1956	7.0	11.0	9.1
1957	9.0	19.6	14.7
1958	7.2	12.8	10.2
1959	10.2	15.6	13.1
1960	13.4	17.2	15.4
1961	7.2	16.7	12.3
1962	9.9	14.8	12.5
1963	3.3	9.2	6.5
1964	4.1	10.4	7.5
1965	11.0	21.6	16.7
1966	12.0	20.2	16.4
1967	10.3	22.1	16.6
1968	9.8	17.0	13.7
1969	13.1	21.1	17.4
1970	12.7	24.9	19.2
1971	10.0	16.5	13.5
1972	15.1	21.1	18.3
1973	21.5	25.6	23.7
1974	20.7	26.0	23.5
1975	16.2	24.8	20.8
1976	14.5	17.5	16.1
1977	6.1	14.7	10.7
1978	12.9	17.2	15.2
1979	14.9	22.7	19.1
1980	13.0	15.5	14.3
1981	15.2	18.5	16.9
1982	14.2	20.2	17.4
1983	18.5	33.0	26.3
1984	15.5	22.9	19.4
1985	13.6	21.6	17.9
1986	16.5	22.5	19.7
1987	15.1	18.7	17.0
1988	9.5	13.3	11.5
1989	10.6	18.7	14.9
1990	12.2	19.8	16.3
1991	14.9	19.8	17.6
1992	10.4	18.0	14.5
1993	21.1	32.3	27.1
1994	10.5	14.1	12.4
Minimum	3.3	9.2	6.5
Maximum	21.5	33.0	27.1
Average	12.3	19.0	15.8
Stan Dev	4.1	5.0	4.4

5-Year Running Average

Water Year	Gaged (inches)	Simulated (inches)	Total (inches)
1955	11.3	17.0	14.4
1956	10.2	15.1	12.8
1957	8.9	15.5	12.4
1958	8.8	15.7	12.5
1959	9.2	15.6	12.7
1960	9.4	15.2	12.5
1961	9.4	16.4	13.1
1962	9.6	15.4	12.7
1963	8.8	14.7	12.0
1964	7.6	13.7	10.8
1965	7.1	14.5	11.1
1966	8.1	15.2	11.9
1967	8.1	16.7	12.7
1968	9.4	18.3	14.2
1969	11.2	20.4	16.1
1970	11.6	21.0	16.7
1971	11.2	20.3	16.1
1972	12.2	20.1	16.4
1973	14.5	21.8	18.4
1974	16.0	22.8	19.6
1975	16.7	22.8	20.0
1976	17.6	23.0	20.5
1977	15.8	21.7	19.0
1978	14.1	20.0	17.3
1979	12.9	19.4	16.4
1980	12.3	17.5	15.1
1981	12.4	17.7	15.2
1982	14.0	18.8	16.6
1983	15.2	22.0	18.8
1984	15.3	22.0	18.9
1985	15.4	23.2	19.6
1986	15.6	24.0	20.1
1987	15.8	23.7	20.1
1988	14.0	19.8	17.1
1989	13.0	19.0	16.2
1990	12.8	18.6	15.9
1991	12.4	18.1	15.5
1992	11.5	17.9	14.9
1993	13.8	21.7	18.1
1994	13.8	20.8	17.6
Minimum	7.1	13.7	10.8
Maximum	17.6	24.0	20.5
Average	12.2	18.9	15.8
Stan Dev	2.8	3.0	2.8

Conversion (cfs-years to inches/sq mile):

Gaged (Area=312 sq mi):	0.043537
Simulated (Area=361 sq mi):	0.037628
Total (Area=673 sq mi):	0.020184

Figure B-4.1 Gaged Watershed - Runoff

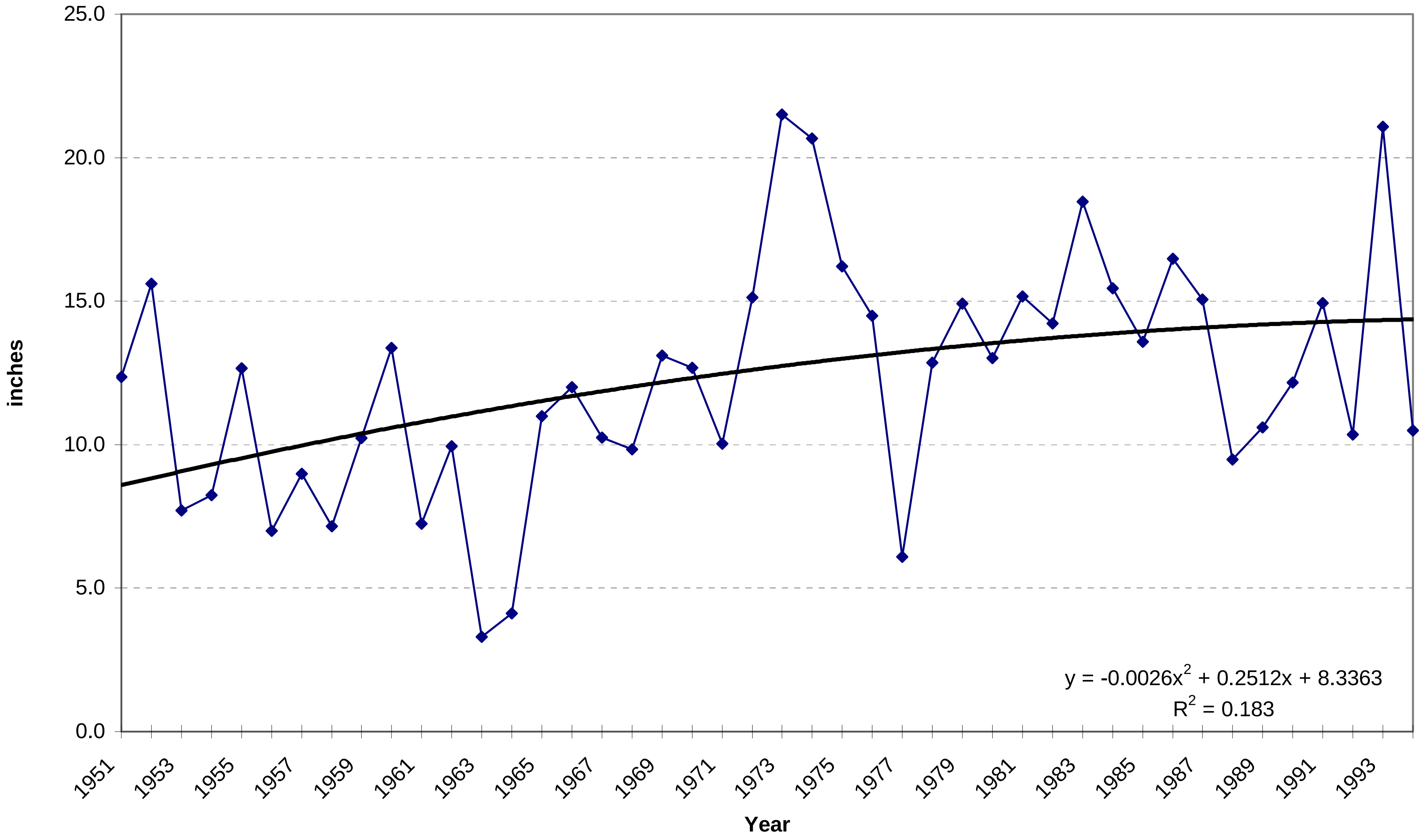
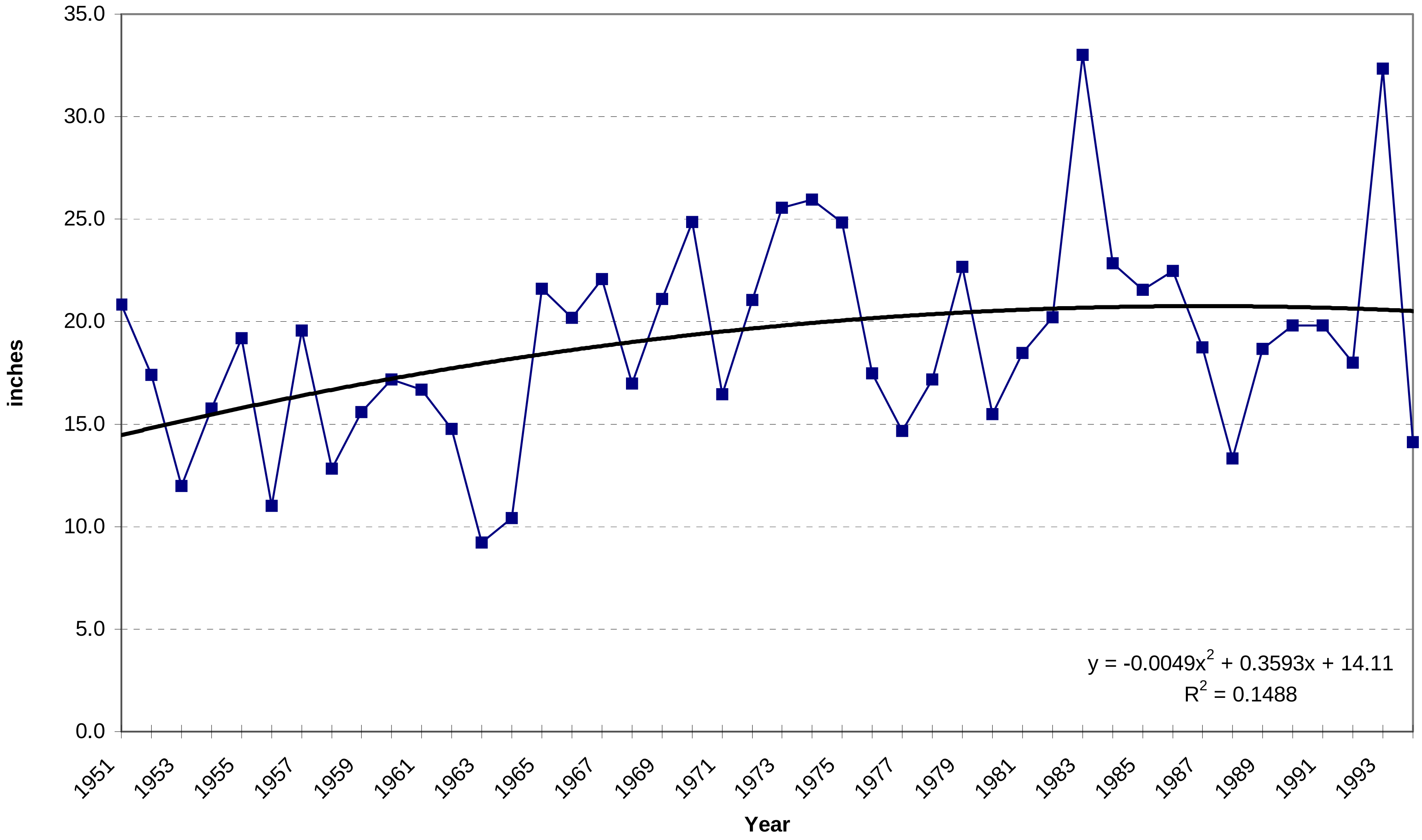


Figure B-4.2 Simulated Watershed - Runoff



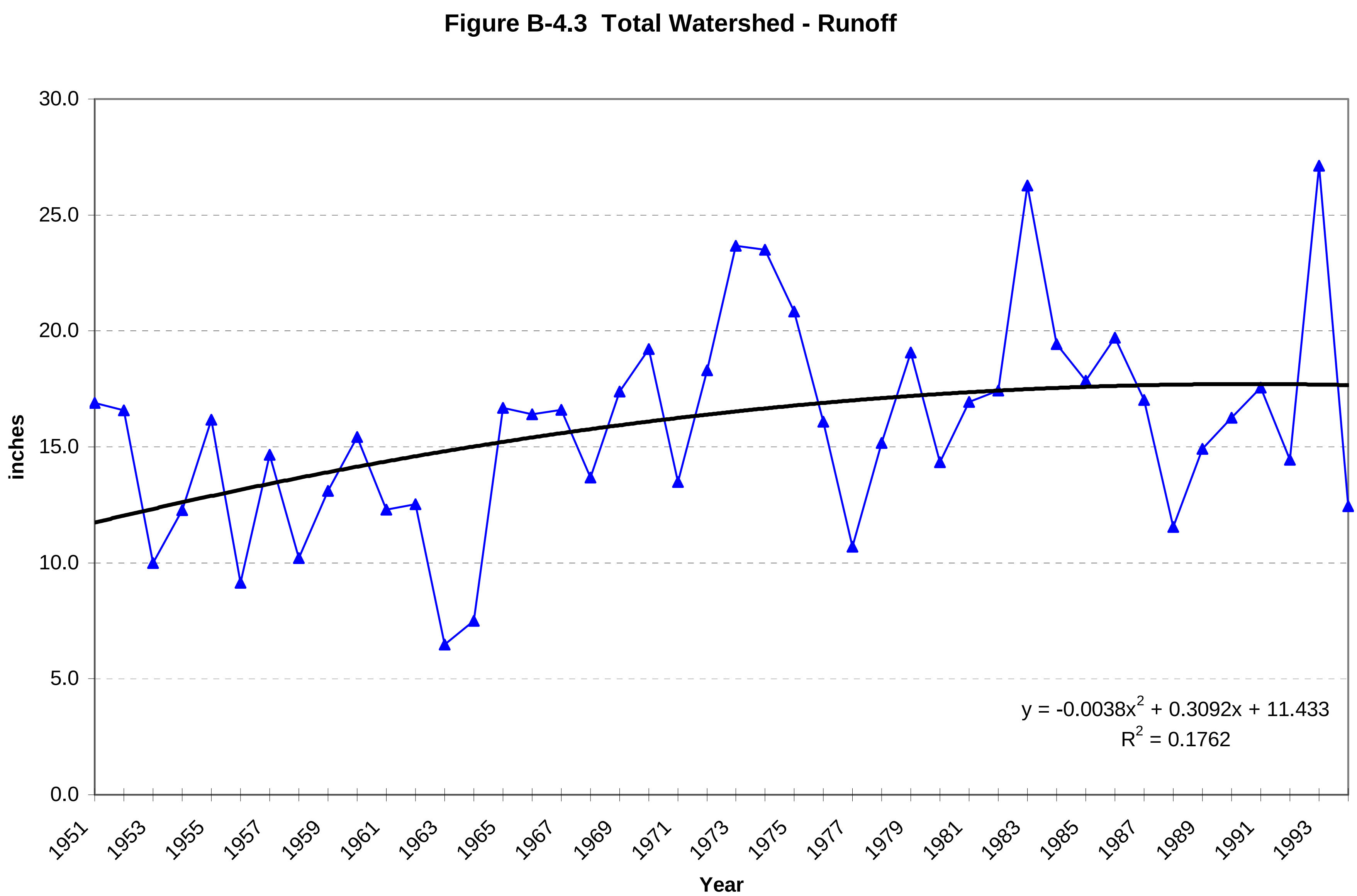


Figure B-4.3 Total Watershed - Runoff
30.0
25.0
20.0
15.0
10.0
5.0
0.0
inches
1951
1953
1955
1957
1959
1961
1963
1965
1967
1969
1971
1973
1975
1977
1979
1981
1983
1985
1987
1989
1991
1993
Year
y = -0.0038x2 + 0.3092x + 11.433
R2 = 0.1762

Figure B-4.4 Gaged Watershed: 5-Year Average Runoff

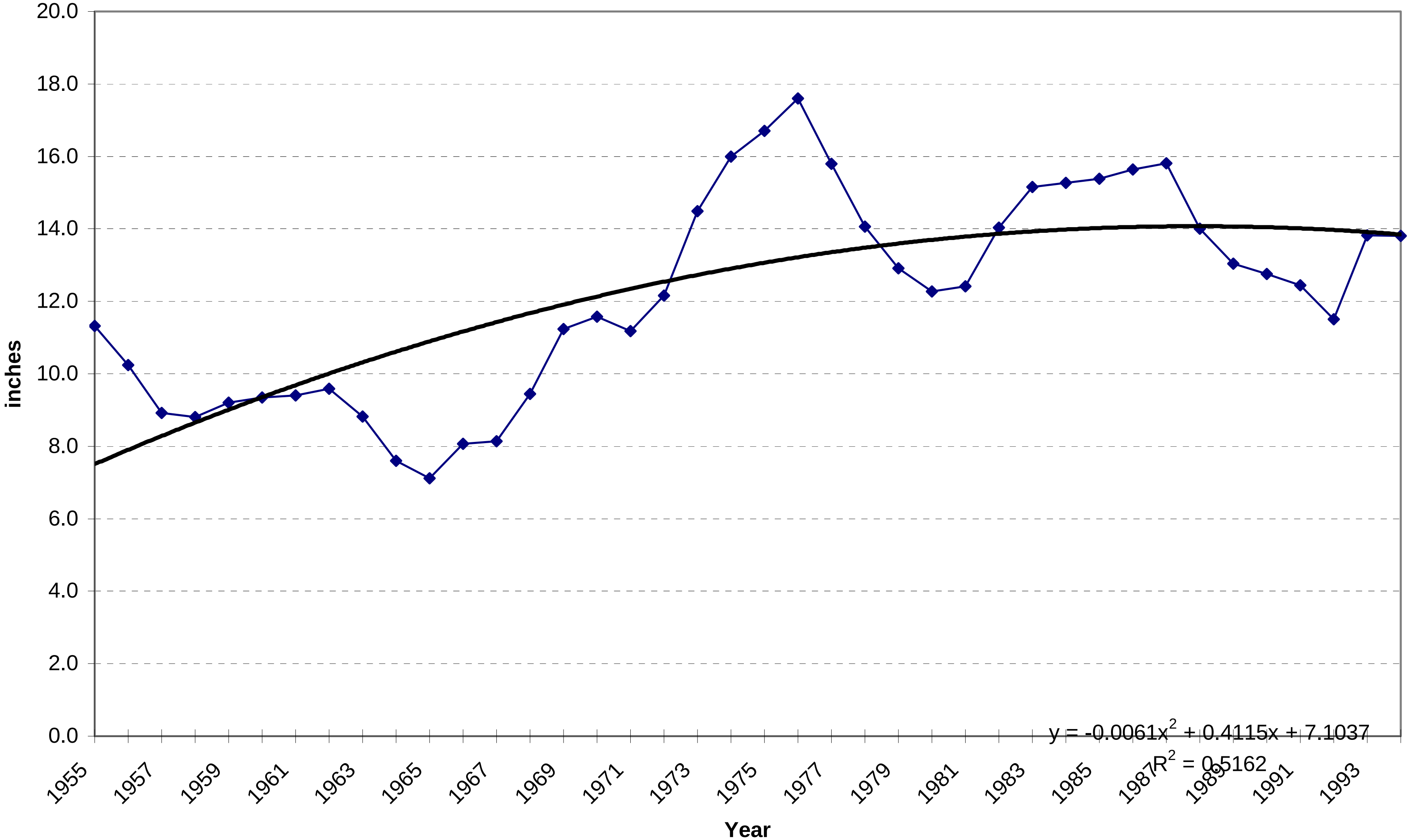


Figure B-4.5 Simulated Watershed: 5-Year Average Runoff

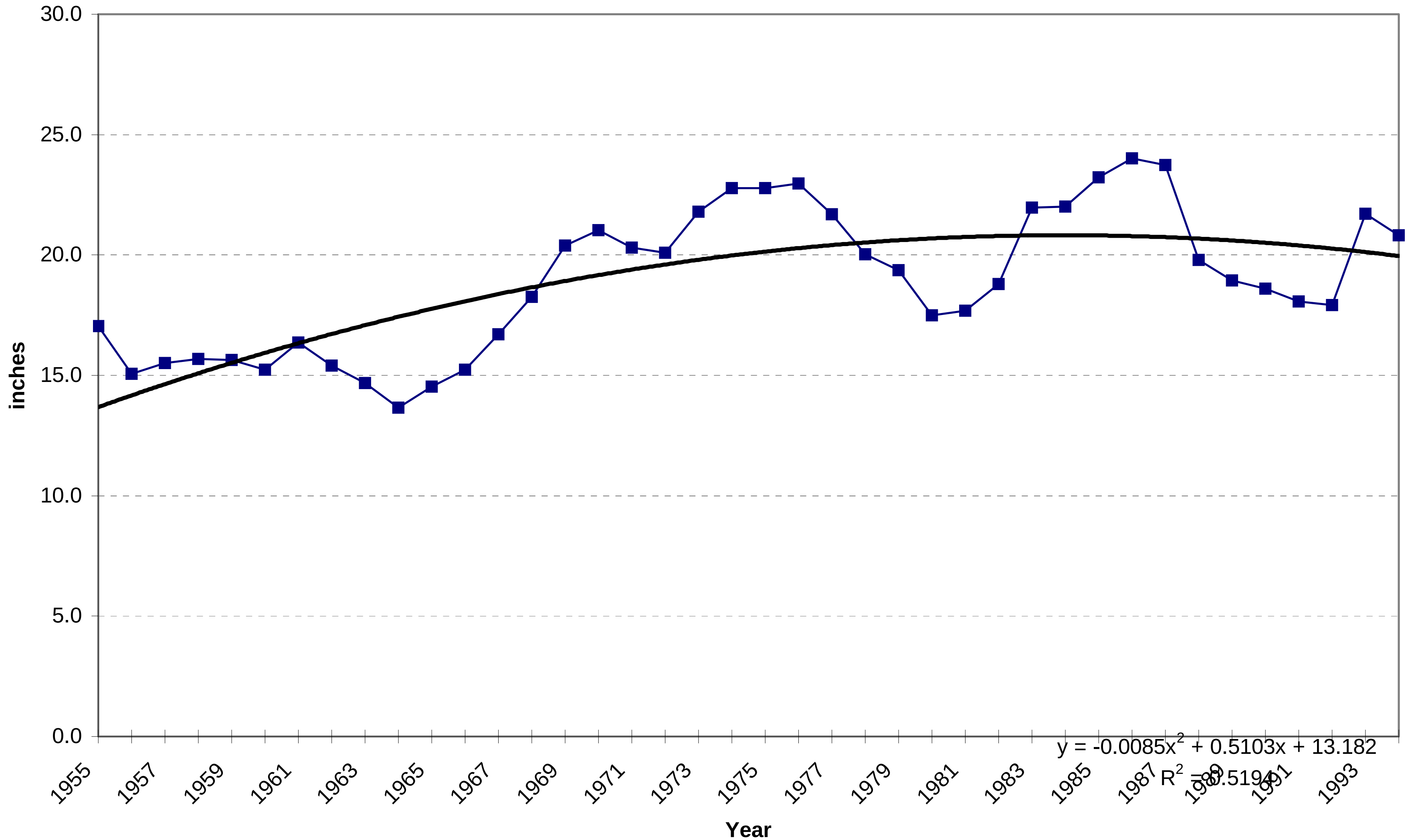
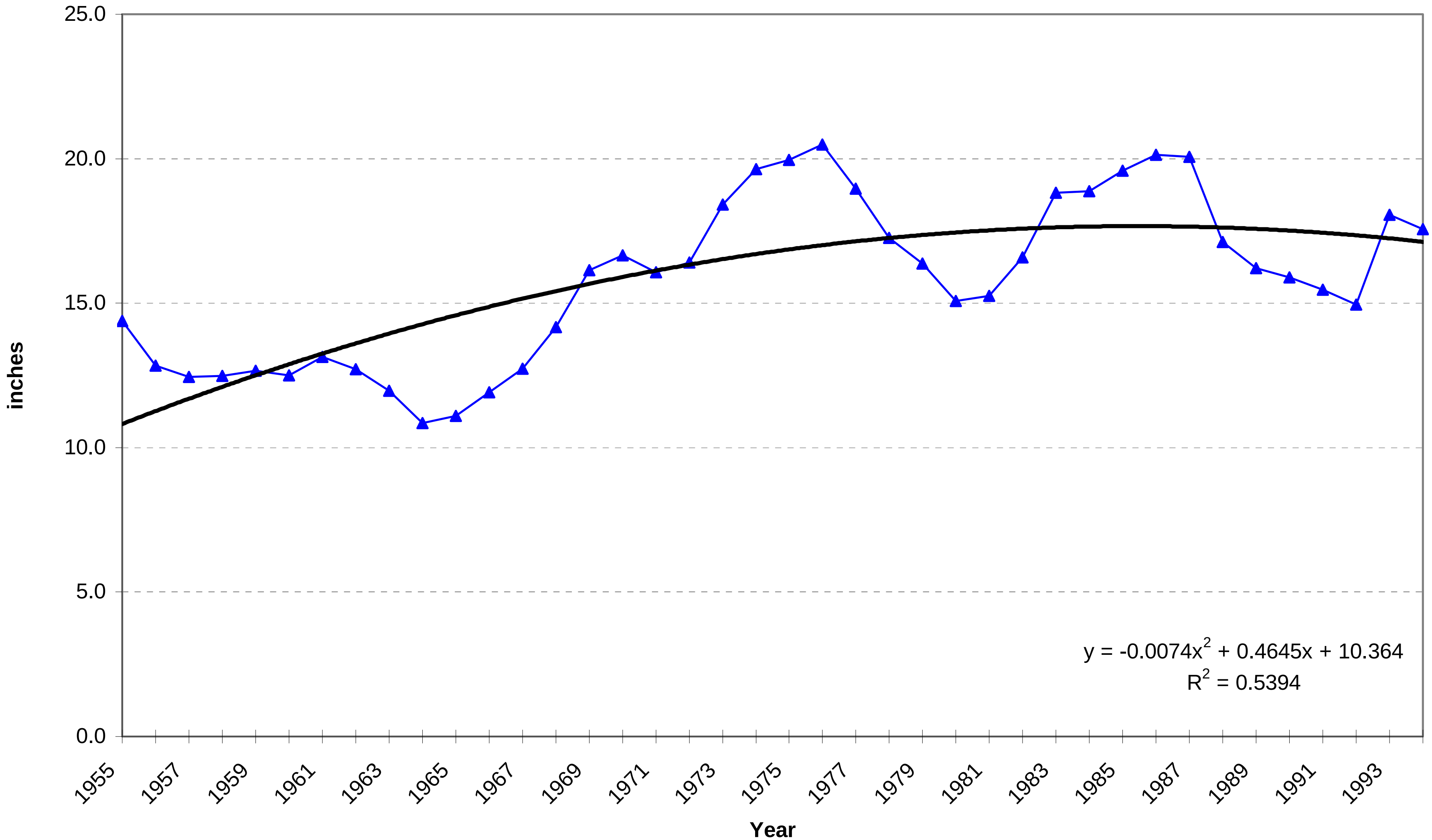


Figure B-4.6 Total Watershed: 5-Year Average Runoff



Attachment B-5

Watershed Rainfall-Runoff

Table B-5.1 Watershed Rainfall-Runoff

Water	Gaged (inches)			Simulated (inches)			Total (inches)		
Year	Rainfall	Runoff	Losses	Rainfall	Runoff	Losses	Rainfall	Runoff	Losses
1951	37.9	12.4	25.5	38.2	20.8	17.4	38.1	16.9	21.2
1952	31.0	15.6	15.4	31.2	17.4	13.8	31.1	16.6	14.6
1953	28.3	7.7	20.6	28.5	12.0	16.6	28.5	10.0	18.5
1954	35.1	8.2	26.9	35.4	15.8	19.6	35.3	12.3	23.1
1955	36.8	12.7	24.1	37.3	19.2	18.1	37.2	16.2	21.0
1956	26.8	7.0	19.8	26.8	11.0	15.8	26.8	9.1	17.6
1957	38.8	9.0	29.8	38.7	19.6	19.1	38.7	14.7	24.0
1958	29.6	7.2	22.5	29.7	12.8	16.9	29.7	10.2	19.5
1959	32.7	10.2	22.4	33.1	15.6	17.5	33.0	13.1	19.9
1960	32.1	13.4	18.7	31.9	17.2	14.7	32.0	15.4	16.6
1961	36.5	7.2	29.3	36.9	16.7	20.3	36.8	12.3	24.5
1962	26.4	9.9	16.5	26.4	14.8	11.7	26.4	12.5	13.9
1963	26.5	3.3	23.2	26.8	9.2	17.5	26.7	6.5	20.2
1964	26.8	4.1	22.7	26.7	10.4	16.3	26.7	7.5	19.2
1965	39.0	11.0	28.0	39.1	21.6	17.5	39.1	16.7	22.4
1966	31.9	12.0	19.9	32.2	20.2	12.0	32.1	16.4	15.7
1967	37.8	10.3	27.6	38.1	22.1	16.0	38.0	16.6	21.4
1968	32.4	9.8	22.6	32.3	17.0	15.3	32.3	13.7	18.7
1969	36.8	13.1	23.7	37.1	21.1	16.0	37.0	17.4	19.6
1970	43.6	12.7	30.9	43.4	24.9	18.6	43.5	19.2	24.3
1971	31.0	10.0	20.9	31.3	16.5	14.8	31.2	13.5	17.7
1972	40.2	15.1	25.0	39.8	21.1	18.7	39.9	18.3	21.6
1973	39.3	21.5	17.8	39.4	25.6	13.9	39.4	23.7	15.7
1974	40.0	20.7	19.3	40.2	26.0	14.2	40.1	23.5	16.6
1975	41.8	16.2	25.6	42.0	24.8	17.1	41.9	20.8	21.1
1976	34.0	14.5	19.5	34.2	17.5	16.8	34.2	16.1	18.1
1977	36.0	6.1	30.0	36.6	14.7	21.9	36.4	10.7	25.7
1978	33.5	12.9	20.6	33.3	17.2	16.2	33.4	15.2	18.2
1979	36.7	14.9	21.8	36.7	22.7	14.1	36.7	19.1	17.7
1980	34.8	13.0	21.8	34.3	15.5	18.8	34.5	14.3	20.1
1981	38.9	15.2	23.7	38.8	18.5	20.3	38.8	16.9	21.9
1982	34.7	14.2	20.5	34.9	20.2	14.7	34.9	17.4	17.4
1983	52.1	18.5	33.6	51.9	33.0	18.9	52.0	26.3	25.7
1984	39.4	15.5	23.9	39.6	22.9	16.7	39.5	19.4	20.1
1985	37.2	13.6	23.6	37.5	21.6	16.0	37.4	17.9	19.6
1986	41.4	16.5	25.0	41.7	22.5	19.2	41.6	19.7	21.9
1987	36.1	15.1	21.0	35.9	18.7	17.1	35.9	17.0	18.9
1988	26.5	9.5	17.0	26.8	13.3	13.5	26.7	11.5	15.1
1989	38.1	10.6	27.5	38.0	18.7	19.3	38.0	14.9	23.1
1990	38.0	12.2	25.8	38.2	19.8	18.4	38.1	16.3	21.9
1991	36.3	14.9	21.3	36.5	19.8	16.7	36.4	17.6	18.9
1992	34.7	10.4	24.3	34.9	18.0	16.9	34.8	14.5	20.4
1993	49.7	21.1	28.6	49.8	32.3	17.5	49.8	27.1	22.7
1994	27.8	10.5	17.3	27.8	14.1	13.6	27.8	12.4	15.3
Minimum	26.4	3.3	15.4	26.4	9.2	11.7	26.4	6.5	13.9
Maximum	52.1	21.5	33.6	51.9	33.0	21.9	52.0	27.1	25.7
Average	35.6	12.3	23.3	35.7	19.0	16.7	35.6	15.8	19.8
Stan Dev	5.7	4.1	4.2	5.7	5.0	2.3	5.7	4.4	3.0

Figure B-5.1 Gaged Rainfall-Runoff

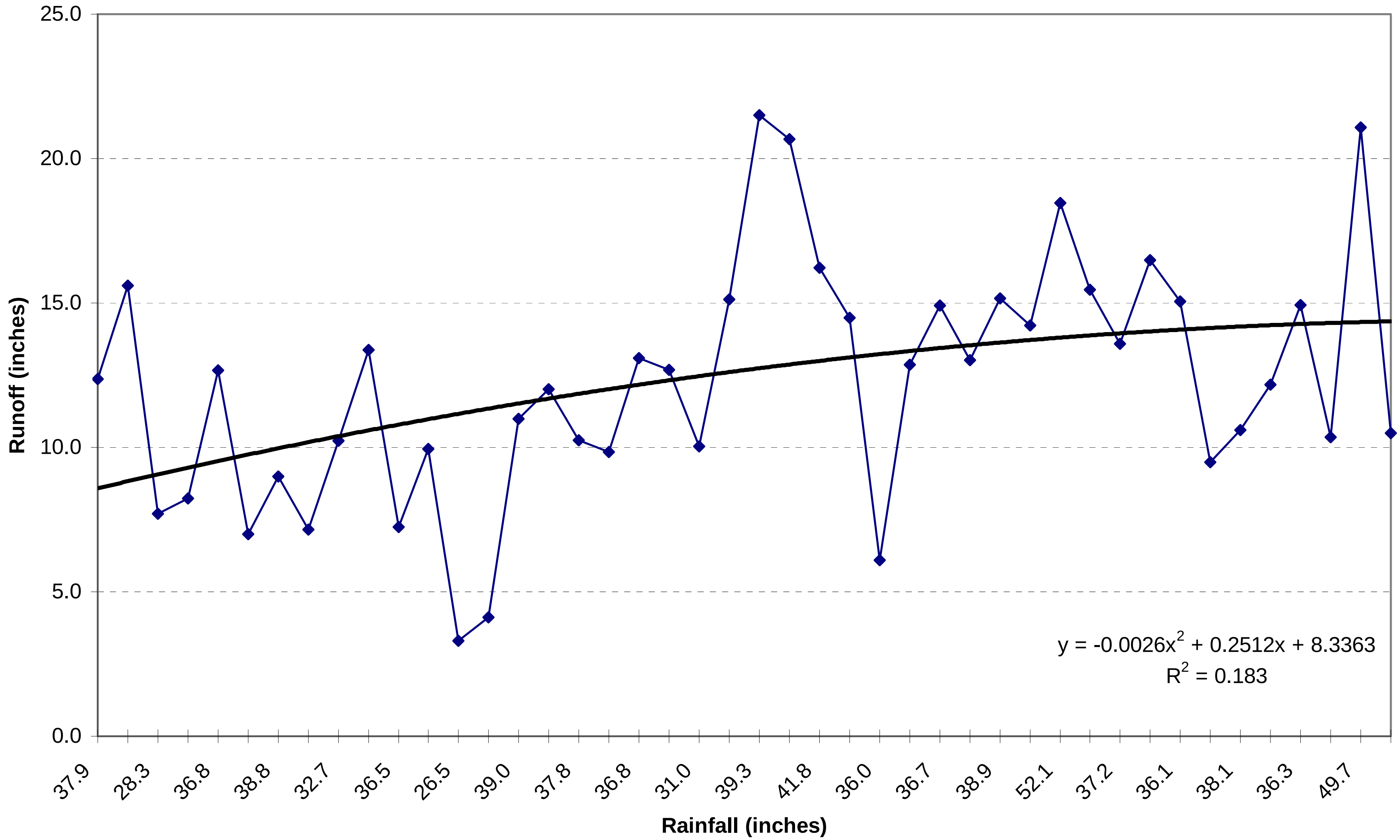


Figure B-5.2 Simulated Rainfall-Runoff

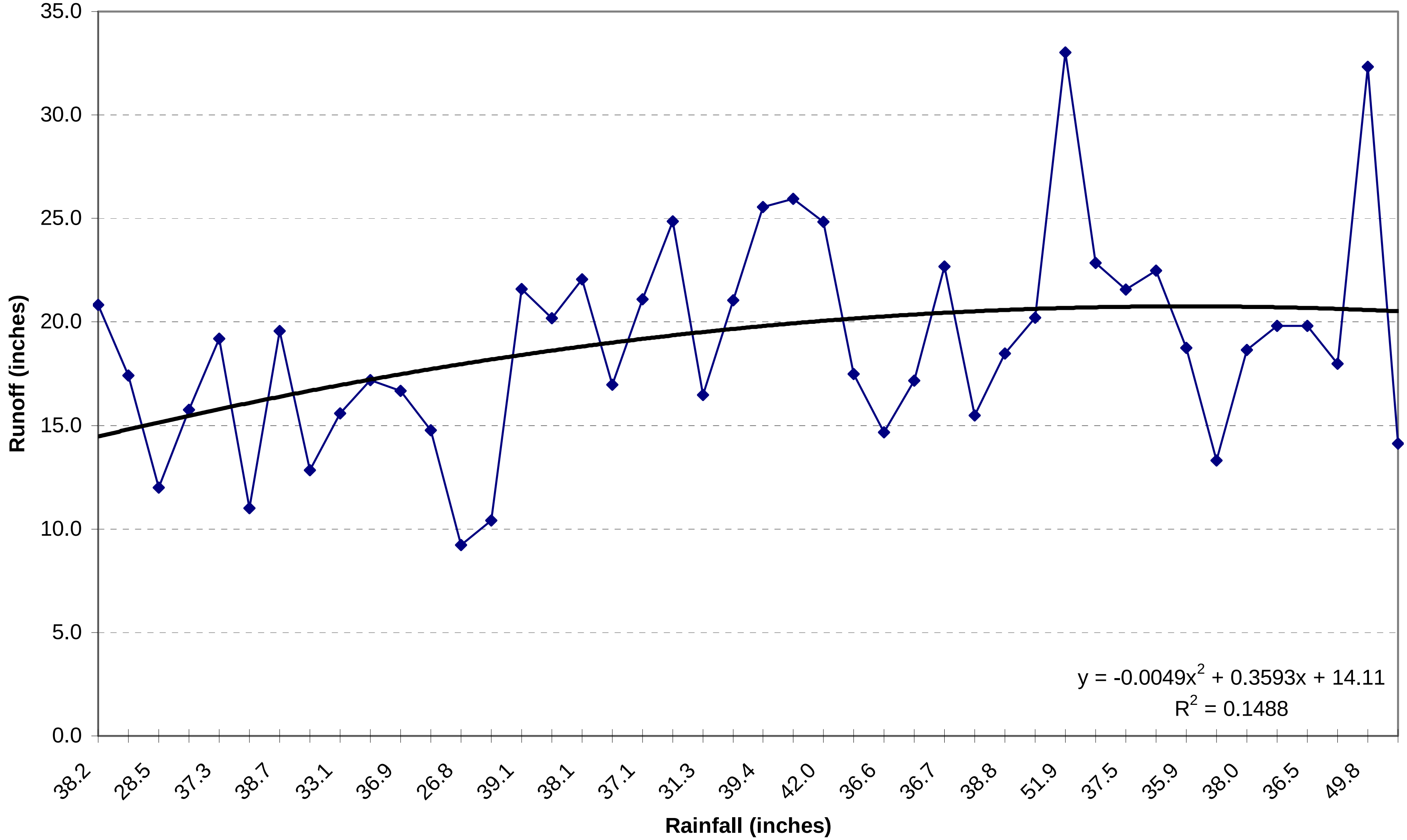
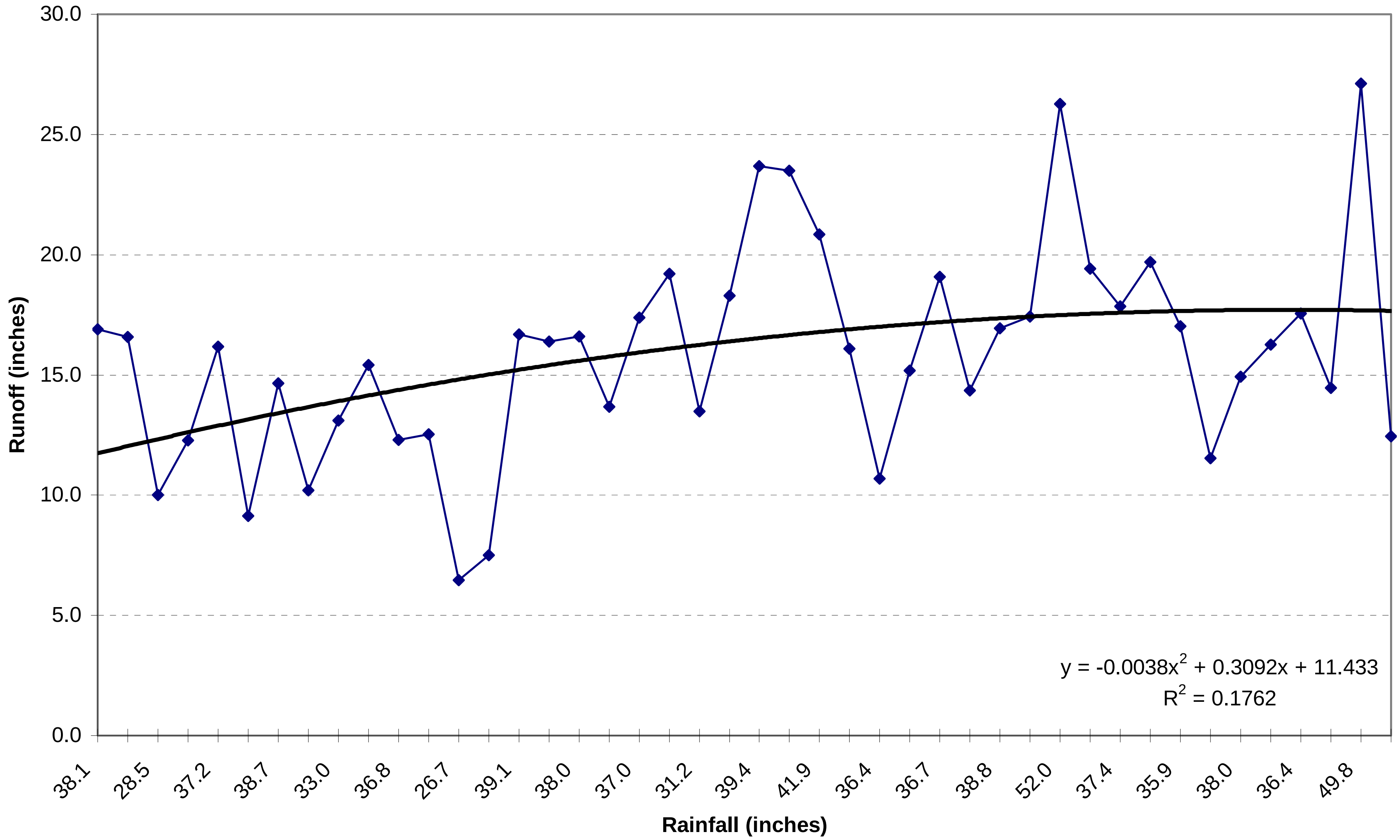


Table B-5.3 Total Rainfall-Runoff



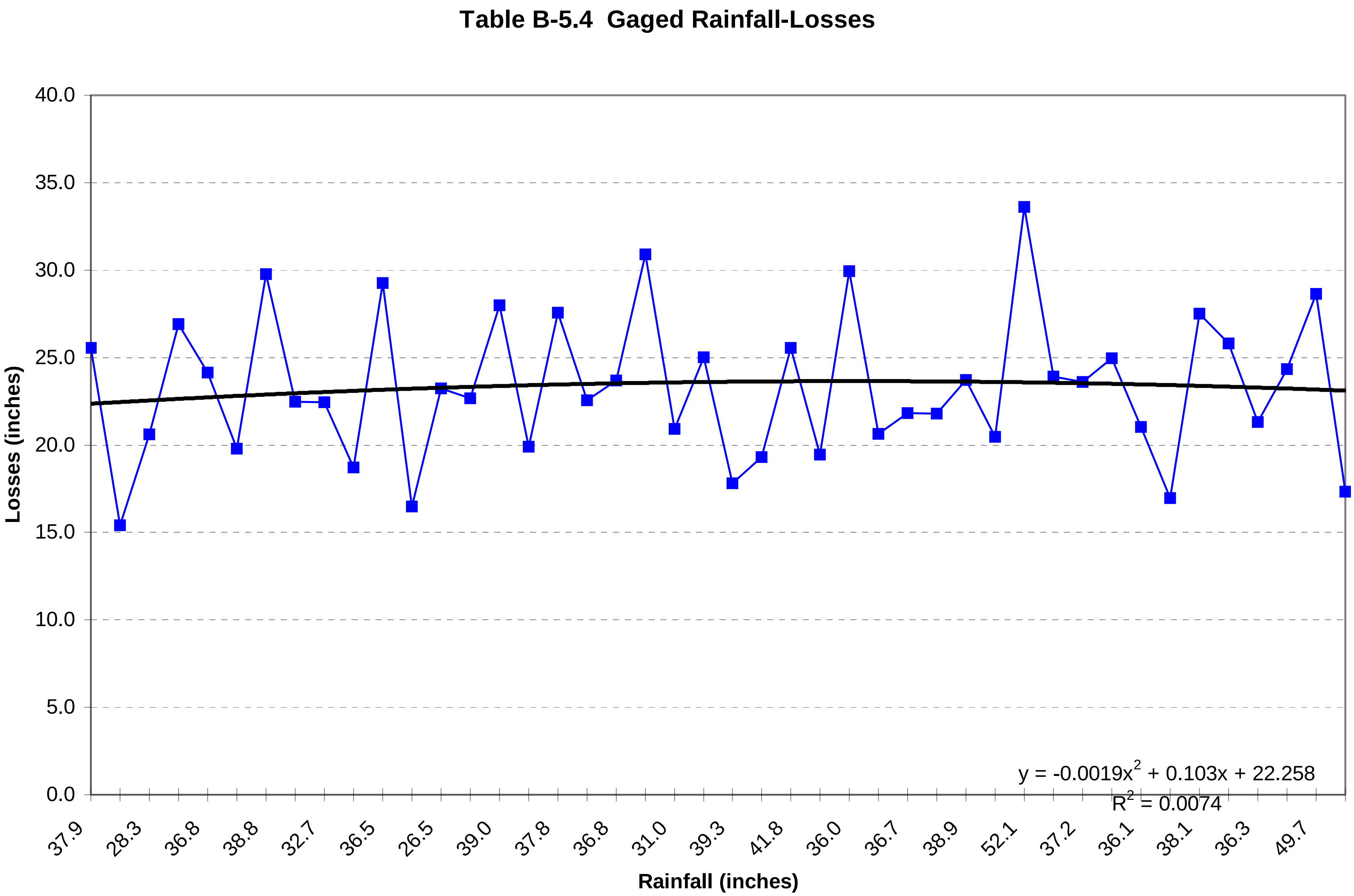


Table B-5.4 Gaged Rainfall-Losses
Losses (inches)
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
37.9
28.3
36.8
38.8
32.7
36.5
26.5
39.0
37.8
36.8
31.0
39.3
41.8
36.0
36.7
38.9
52.1
37.2
36.1
38.1
36.3
49.7
Rainfall (inches)
y = -0.0019x^2 + 0.103x + 22.258
R^2 = 0.0074

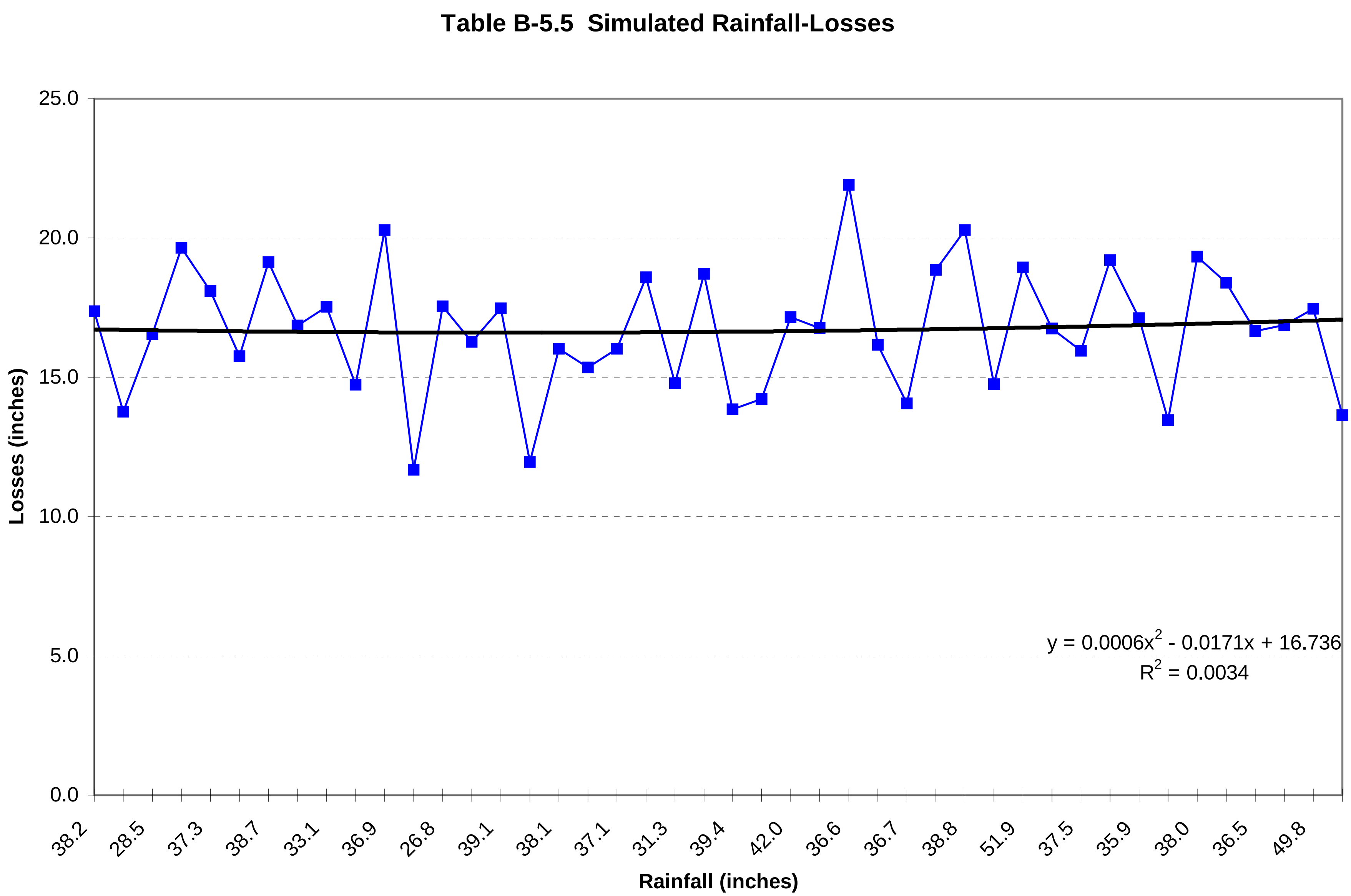
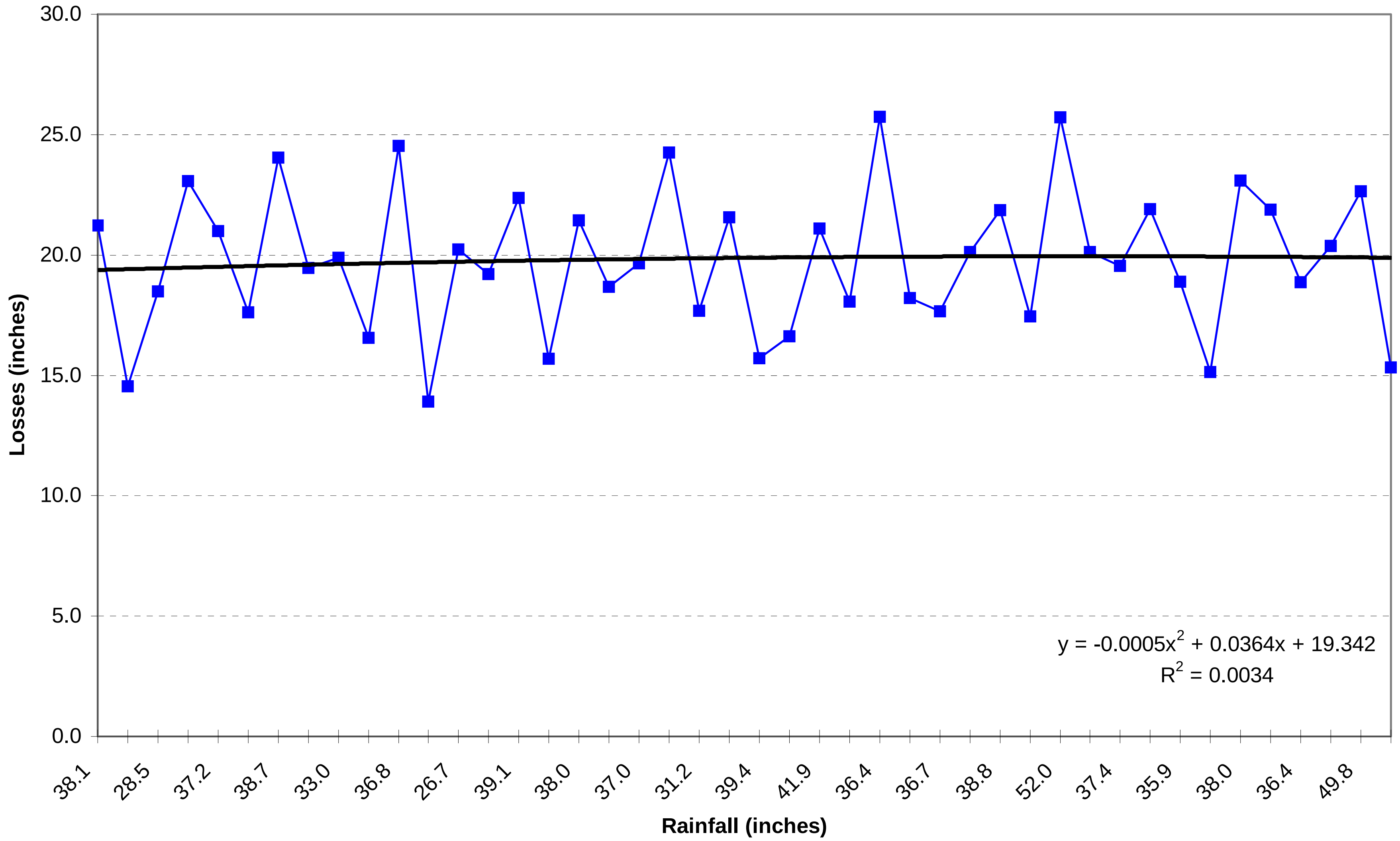


Table B-5.5 Simulated Rainfall-Losses
Losses (inches)
25.0
20.0
15.0
10.0
5.0
0.0
38.2
28.5
37.3
38.7
33.1
36.9
26.8
39.1
38.1
37.1
31.3
39.4
42.0
36.6
36.7
38.8
51.9
37.5
35.9
38.0
36.5
49.8
Rainfall (inches)
$y = 0.0006x^2 - 0.0171x + 16.736$
$R^2 = 0.0034$

Table B-5.6 Total Rainfall-Losses



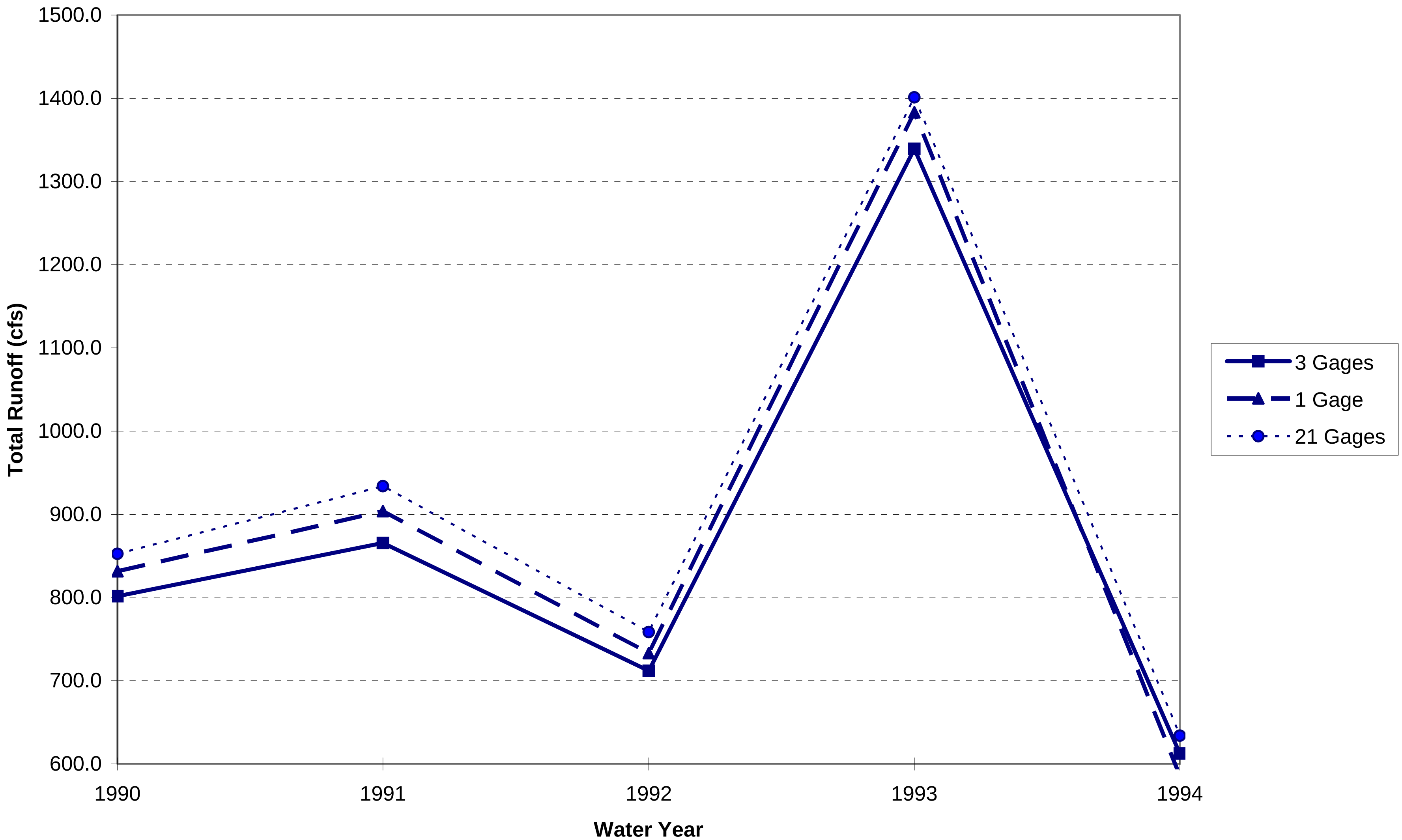
Attachment B-6

Precipitation Gage Analysis

Table B-6.1 PRECIPITATION GAGE SENSITIVITY ANALYSIS				
WATER YEAR	RUNOFF ELEMENT	3-GAGE cfs	1-GAGE cfs	21-GAGE cfs
1990	Simulated	524.4	554.2	575.2
	Streams	277.4	277.4	277.4
	Total	801.8	831.6	852.6
1991	Simulated	524.6	562.6	592.8
	Streams	341.1	341.1	341.0
	Total	865.7	903.7	933.8
1992	Simulated	476.0	497.1	522.8
	Streams	235.9	235.9	235.8
	Total	711.9	733.0	758.6
1993	Simulated	857.2	901.1	919.2
	Streams	482.2	482.1	482.1
	Total	1339.4	1383.2	1401.2
1994	Simulated	373.2	346.1	394.8
	Streams	239.1	239.2	239.1
	Total	612.4	585.3	633.9
5-Year Total Averages		866.2	887.4	916.0

3-GAGE Method used for 44-year Period of Record (WY51-94)
3-GAGE Method utilized O'Hare, Midway, and Univ. of Chicago precip.
1-GAGE Method utilized only the Midway precip.
21-GAGE Method utilized 21 of 25 precip. gages employed for
Lake Michigan Diversion Accounting

Figure B-6.1 Precipitation Sensitivity Analysis



Attachment B-7

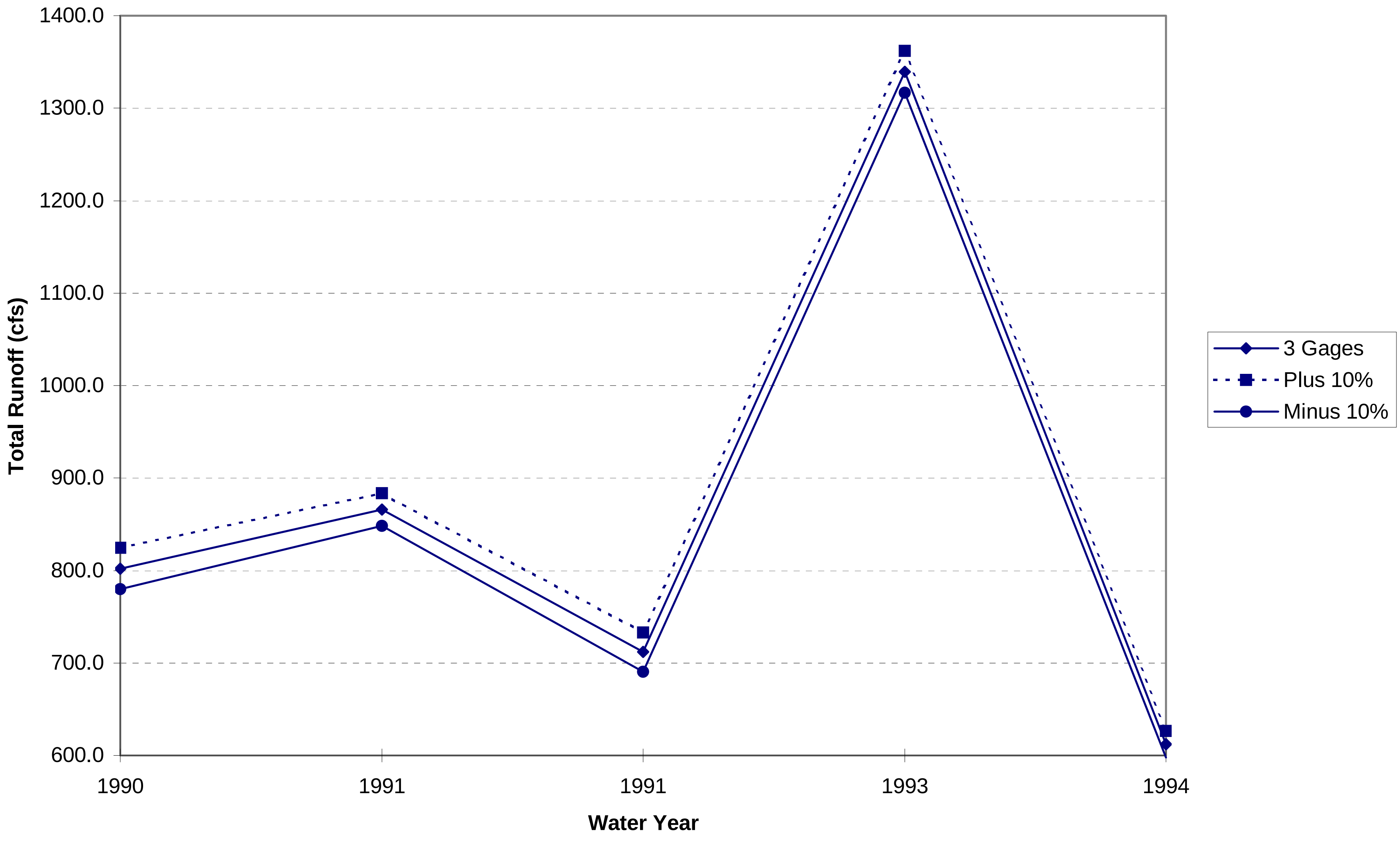
Imperviousness Analysis

Table B-7.1 IMPERVIOUSNESS SENSITIVITY ANAL

WATER YEAR	RUNOFF ELEMENT	3-GAGE	PLUS 10% IMPERV.	MINUS 10% IMPERV.
		cfs	csf	cfs
1990	Simulated	524.4	546.9	502.2
	Streams	277.4	277.4	277.4
	Total	801.8	824.3	779.6
1991	Simulated	524.6	542.6	507.0
	Streams	341.1	341.1	341.1
	Total	865.7	883.7	848.1
1992	Simulated	476.0	497.2	454.5
	Streams	235.9	235.9	235.9
	Total	711.9	733.1	690.4
1993	Simulated	857.2	879.6	834.4
	Streams	482.2	482.3	482.1
	Total	1339.4	1361.9	1316.6
1994	Simulated	373.2	387.5	358.9
	Streams	239.1	239.1	239.1
	Total	612.4	626.7	598.0
5-Year Total Averages		866.2	885.9	846.5

3-GAGE Method used for 44-year Period of Record (WY51-94)
3-GAGE Method utilized O'Hare, Midway, and Univ. of Chicago precip.
PLUS 10% IMPERV. increases impervious area of modeled SCAs
by 10% while using the same 3 precip. gages
MINUS 10% IMPERV. decreases impervious area of modeled SCAs
by 10% while using the same 3 precip. gages

Figure B-7.1 Imperviousness Sensitivity Analysis



Attachment B-8

USGA AVM Flows

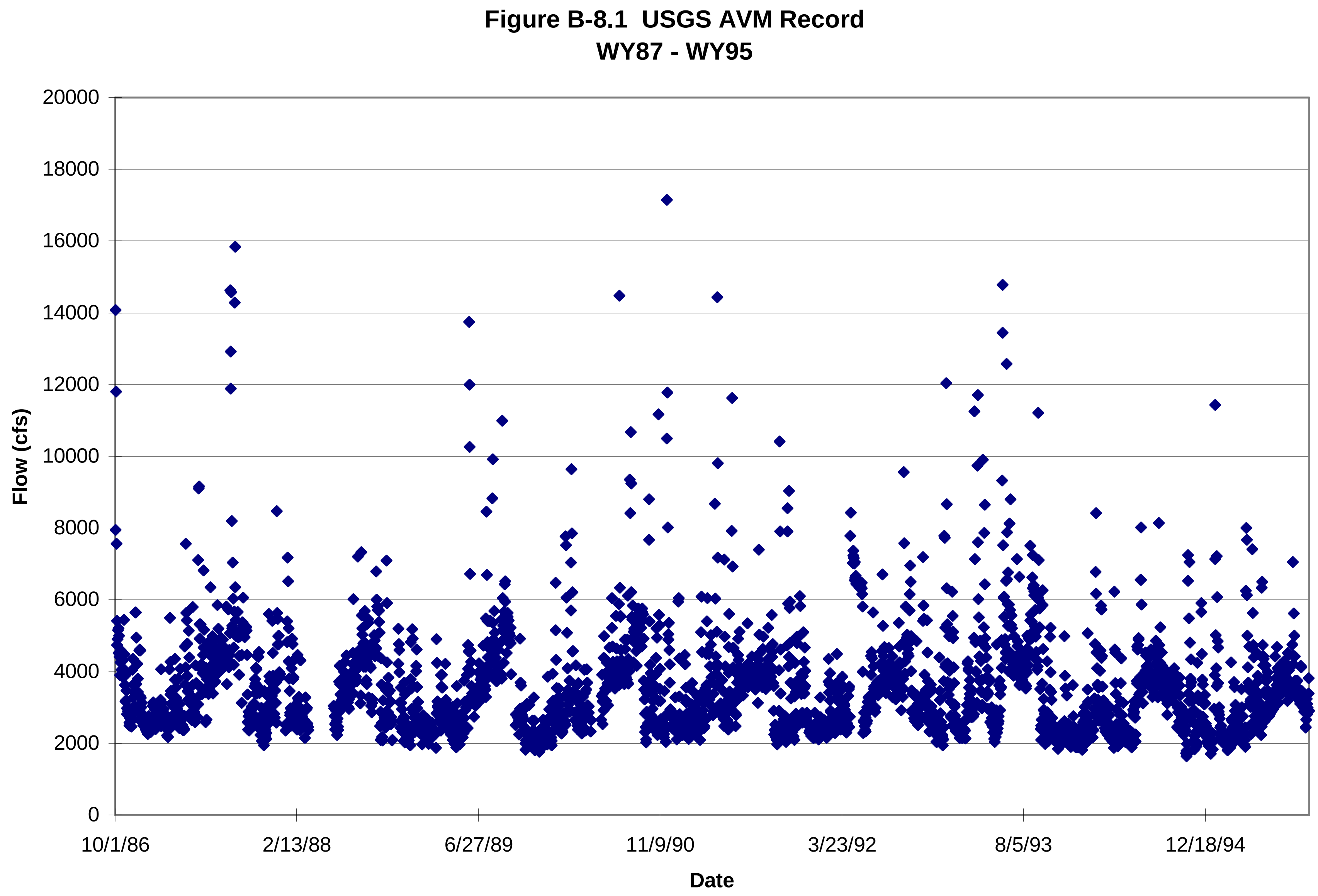


Figure B-8.1 USGS AVM Record
WY87 - WY95
Flow (cfs)
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
10/1/86
2/13/88
6/27/89
11/9/90
3/23/92
8/5/93
12/18/94
Date

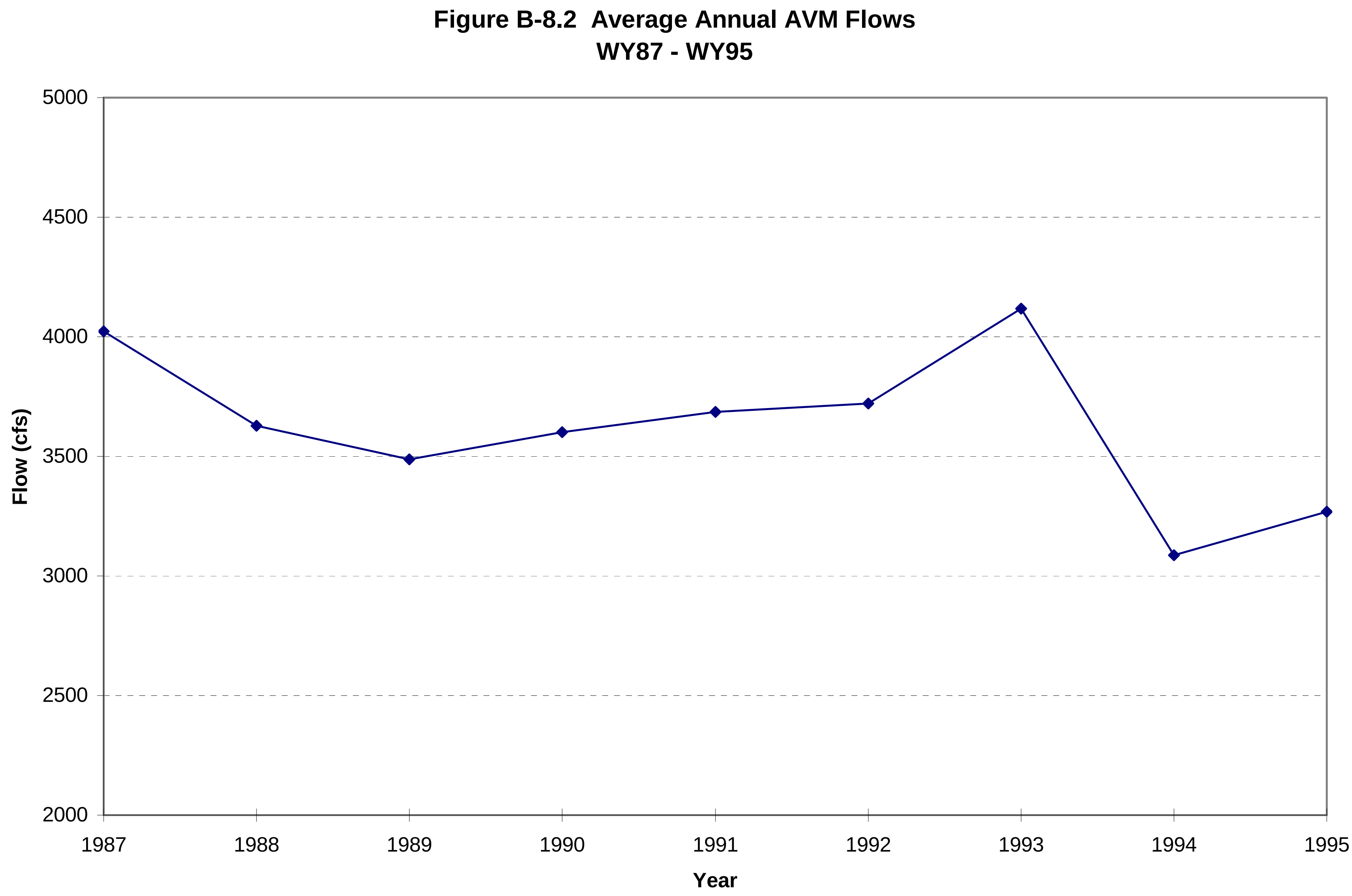


Figure B-8.2 Average Annual AVM Flows
WY87 - WY95
Flow (cfs)
5000
4500
4000
3500
3000
2500
2000
1987
1988
1989
1990
1991
1992
1993
1994
1995
Year

Attachment B-9

Turbine Corrections

Figure B-9.1 Ratio of TLL to AVM Flows

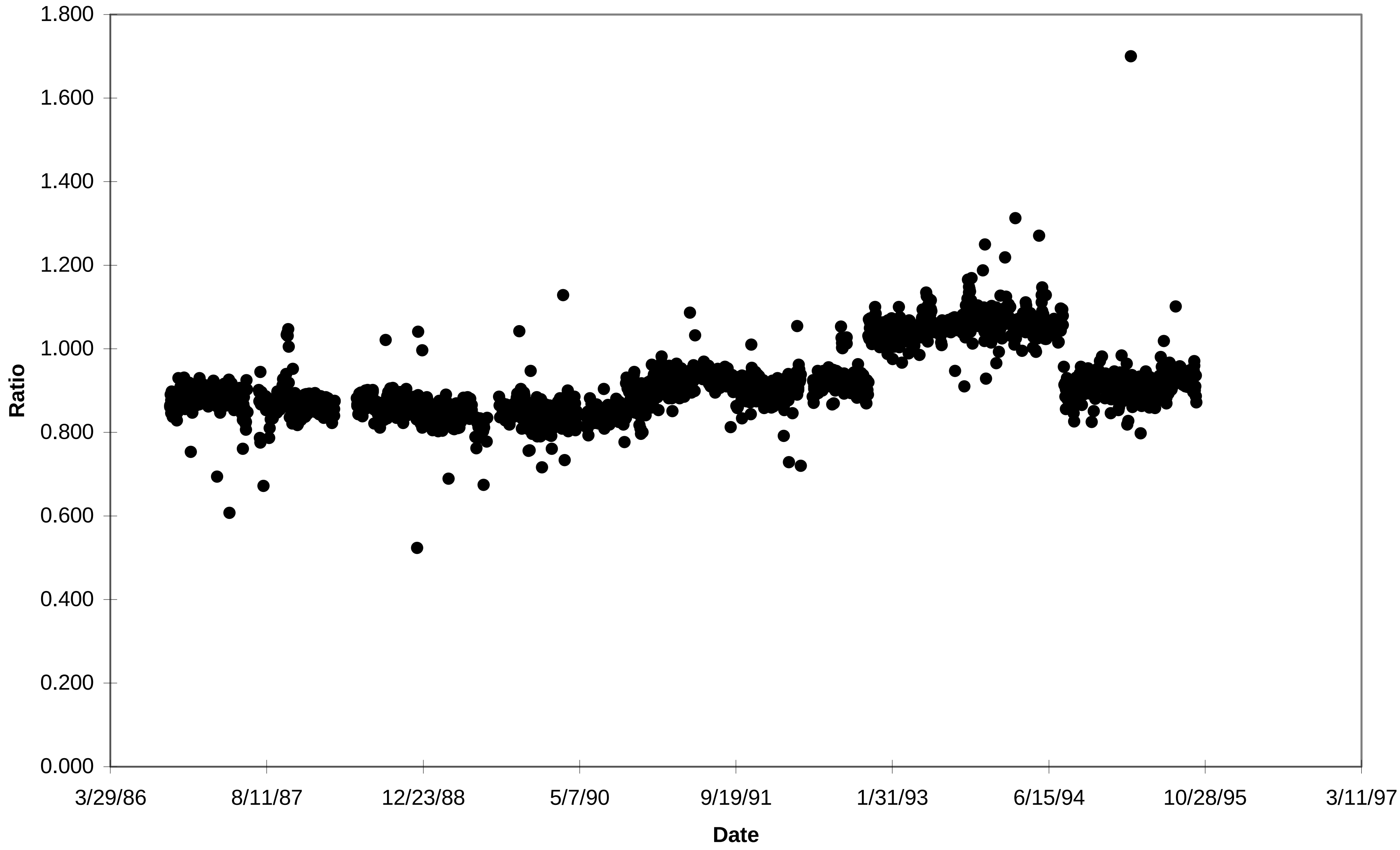


Figure B-9.2 Difference between AVM and TLL Flows

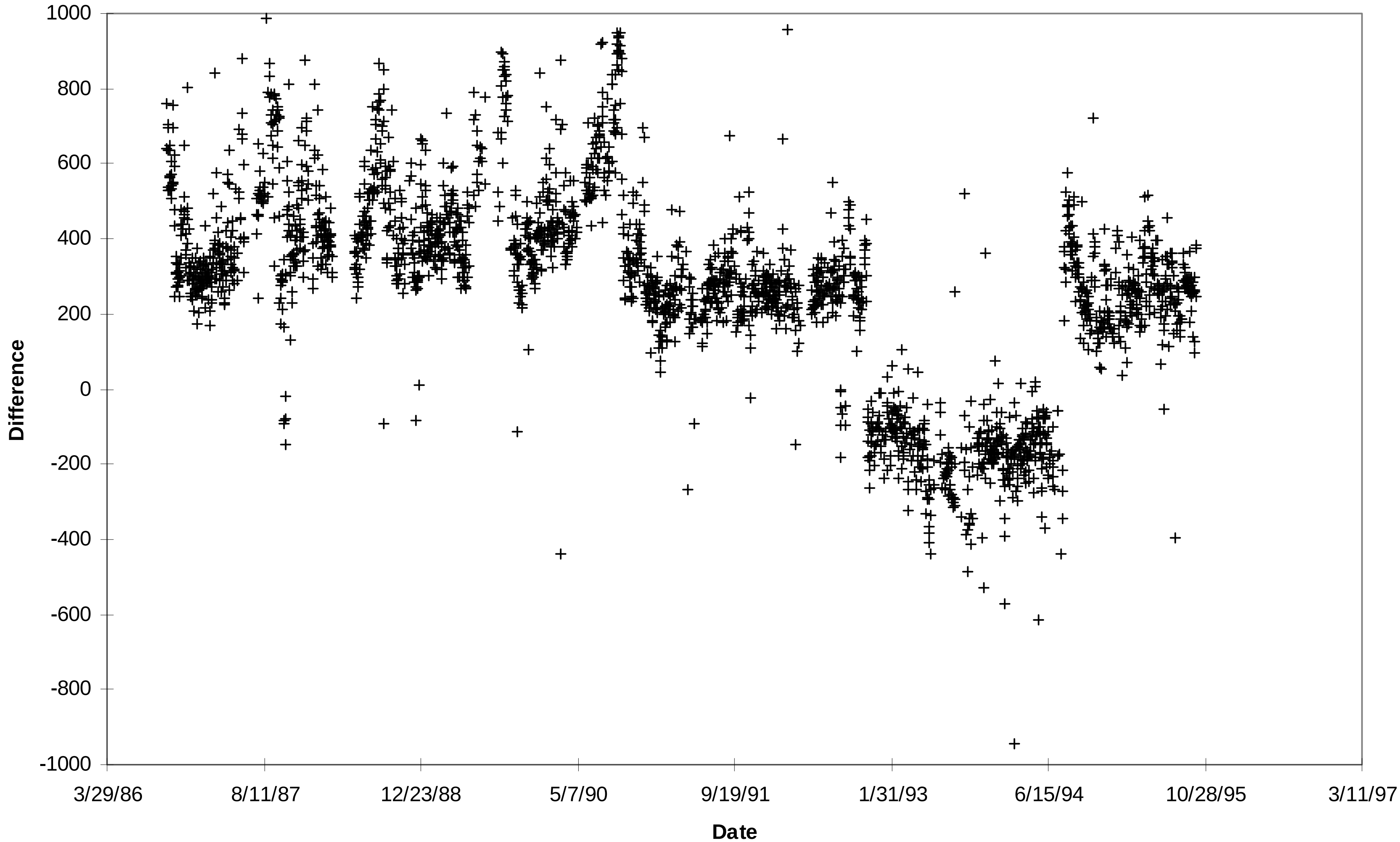


Table B-9.1 Lockport Turbines: Ratings Curves versus Turbine AVM's

MWRD's Period for AVM's: 11/17/92 - 7/31/94

	TLL	AVM		TLL	AVM		TLL	AVM		Ratio	Difference
				(Rating)			(AVM)				
Average:	2896	3171		2849	3221		3120	2953		0.92	275.11
Std Dev:	788	863		767	864		851	827		0.08	275.63
Min:	1364	1640		1364	1640		1924	1821		0.52	-1812.00
Max:	5314	7556		4857	7556		5314	5755		1.70	2967.00
Intrcpt:					82.06			-53.82			
Slope:					1.1020			0.9635			
R Sq:					0.96			0.98			
Std Err:					177			105			
Equating AVM equations, and solving for TLL (Rating) in Terms of TLL (AVM) gives:											
Intrcpt:	-123.31										
Slope:	0.8744										
Avg TLL (Rating):		2605									

Attachment B-10

Lockport and Romeoville AVM Flows

Table B-10.1 Lockport and AVM Flows					
Date	Turbine	Turbine	Lockage	Leakage	
	(Rating)	(AVM)			
1984	2826	0	250	100	
1985	2604	0	283	100	
1986	2951	0	311	100	
1987	2706	0	296	100	
1988	2578	0	315	100	
1989	2208	0	306	100	
1990	2522	0	313	100	
1991	2619	0	301	100	
1992	2620	0	300	100	
1993	2522	3251	274	100	
1994	2901	2419	303	100	
1995	2462	0	236	100	
Average:	2613	2845	291	100	
Date	TLL	Sluice	Controlling	MWRD	
		Gate	Works	Total	
1984	3177	1647	2159	3528	
1985	2987	2195	2432	3601	
1986	3362	2276	663	3725	
1987	3100	2530	3460	3780	
1988	2993	1108	0	3102	
1989	2614	2494	2339	3334	
1990	2935	3808	4000	3557	
1991	2972	623	168	3764	
1992	3020	580	25	3625	
1993	3031	1039	164	4234	
1994	2511	305	12	2828	
1995	2797	323	54	3174	
Average:	2958	923	190	3521	
Date	USGS	Turbine	Sluice Gate	Control Work	AVM
	AVM	Regression	Regression	Regression	Total
1984	0	3459	5527	7748	3895
1985	0	3347	4730	7730	3826
1986	0	3695	6009	7949	4113
1987	4023	3723	0	7329	4028
1988	3628	2946	5175	0	3537
1989	3487	3309	5248	0	3515
1990	3601	3271	6562	11063	3749
1991	3685	3542	5223	0	3713
1992	3720	4297	6899	0	3778
1993	4118	2914	4075	0	4074
1994	3086	3168	0	0	3088
1995	3268	2375	3895	3041	3235
Average:	3625	3435	5385	8098	3713

Figure B-10.1 Lockport vs USGS AVM

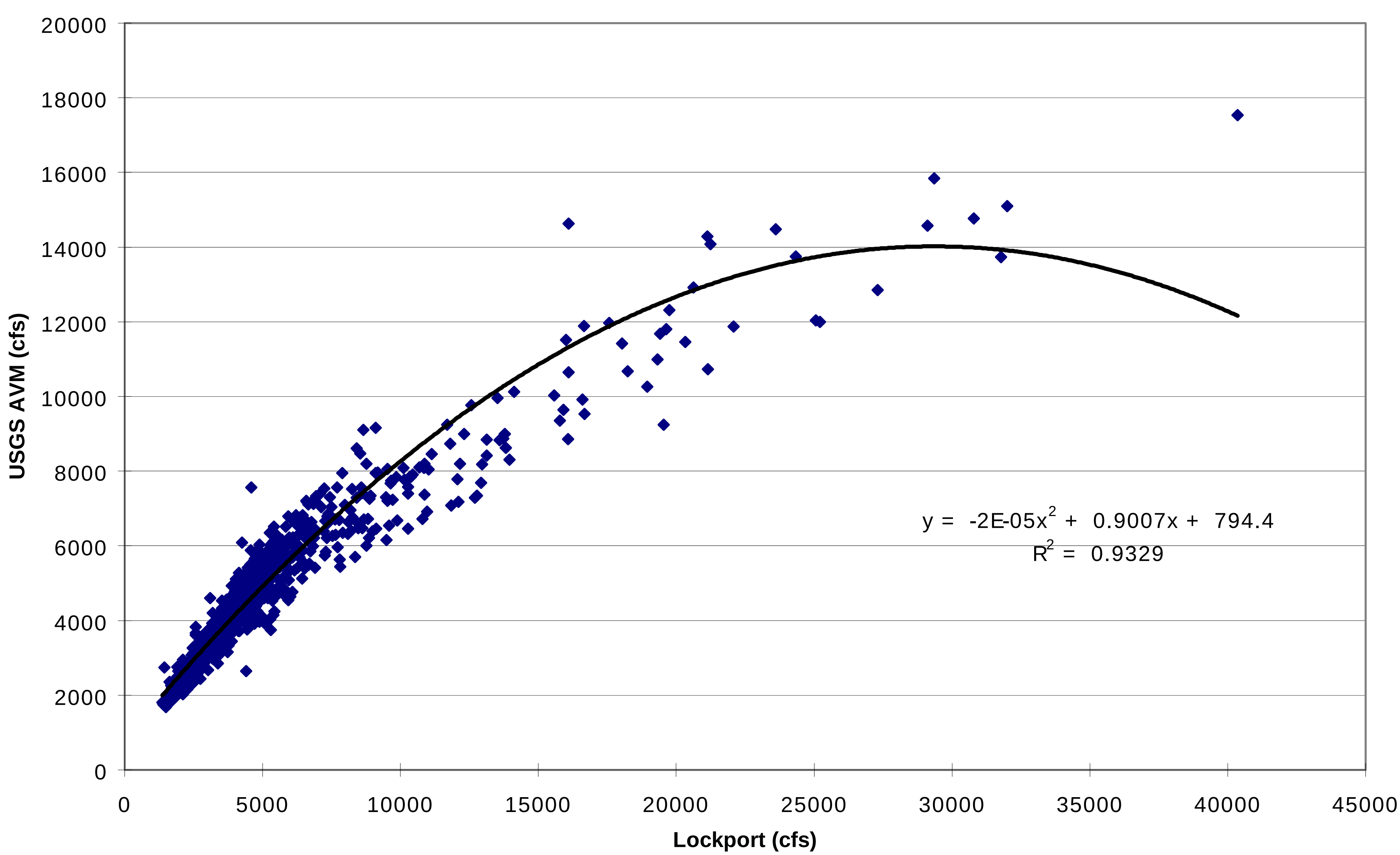
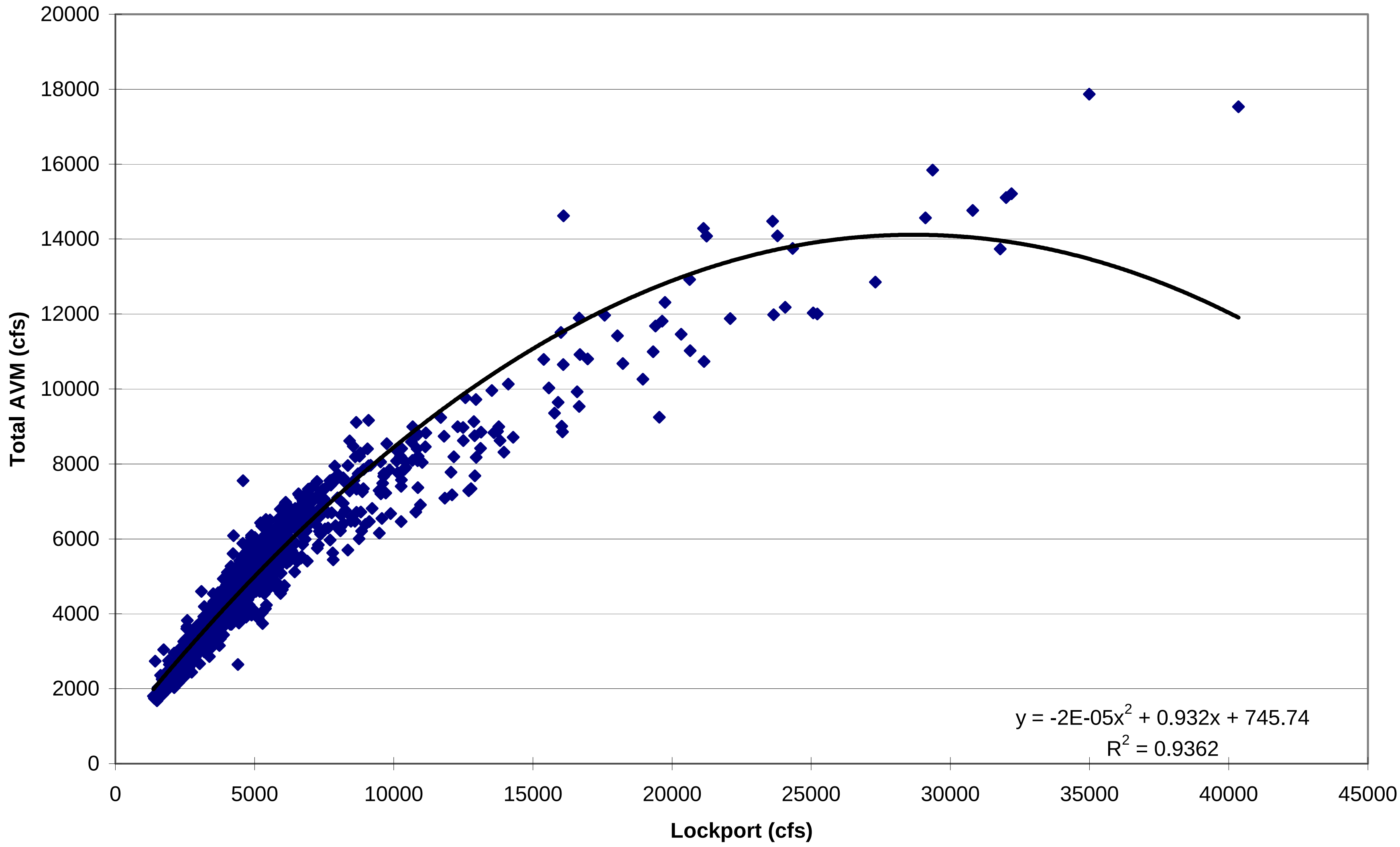


Figure B-10.2 Lockport vs Total AVM



Attachment B-11

Lockport versus Romeoville AVM
Regression Analysis

Table B-11.1 Frequency Analysis				
Flow	TLL	TLL+SG	TLL+SG+CW	Case
3000	2125	54	2	
3200	2358	86	3	1
3400	2609	122	3	
3600	2829	154	3	
3800	3010	188	3	
4000	3158	216	3	2
4200	3277	250	3	
4400	3378	281	4	
4600	3444	315	6	
4800	3498	351	7	
5000	3508	395	9	3
5200		431	11	
5400		464	16	
5600		489	20	
5800		511	22	
6000		533	22	
6200		547	24	
6400		563	27	
6600		586	29	
6800		604	31	
7000		621	34	
7500		644	37	
8000		658	39	
8500		669	43	
9000		679	50	
9500		687	52	
10000		688	59	4
11000		696	75	
12000		698	79	
14000		703	95	
16000		705	101	
18000			110	
20000			118	
25000			130	
30000			135	
35000			140	
40400			141	5

Table B-11.2 Regression Equations / Estimated AVM Flows					
	MWRD	AVM	Est AVM		
Average:	3544	3718	3798		
Std Deviation:	2539	1565	1810		
Case	Limit	Intercept	Slope	R Square	Std Error
1	<=3200	161.7	1.0754	0.94	115
2	<=4000	41.1	1.1082	0.68	173
3	<=5000	871.6	0.9135	0.32	363
4	<=10,000	2753.5	0.5257	0.52	629
5	>10,000	4288.9	0.3503	0.83	1042
WY Year	Avg AVM	Est AVM	% Error		
1984	3895	3853	-1.1		
1985	3826	3850	0.6		
1986	4113	4058	-1.3		
1987	4028	4003	-0.6		
1988	3537	3483	-1.5		
1989	3515	3571	1.6		
1990	3749	3712	-1.0		
1991	3731	3972	6.5		
1992	3780	3950	4.5		
1993	4072	4368	7.3		
1994	3116	3275	5.1		
1995	3235	3478	7.5		

Figure B-11.1 Lockport versus AVM
<=3,200 cfs

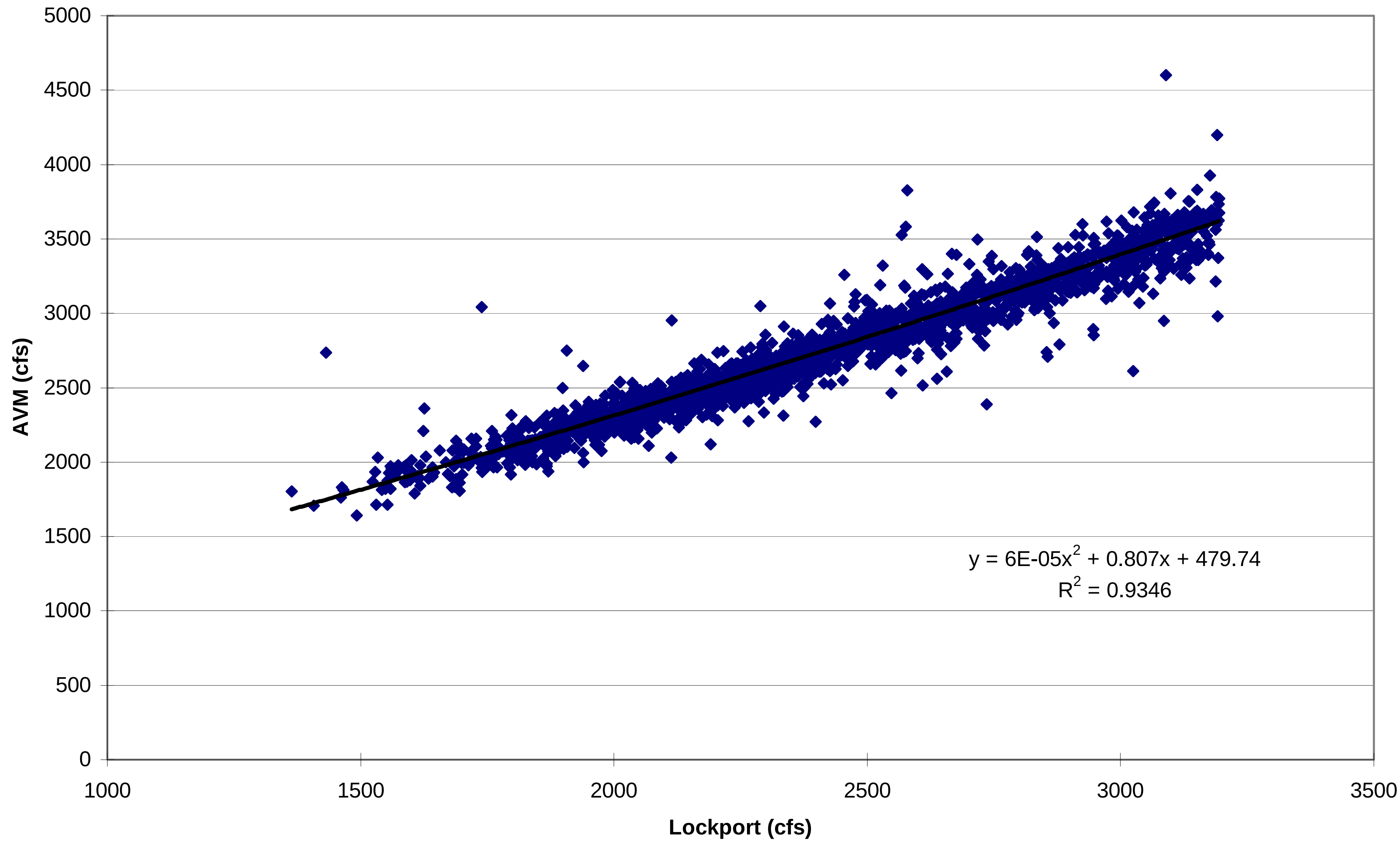


Figure B-11.2 Lockport versus AVM
>3,200 cfs, <=4,000 cfs

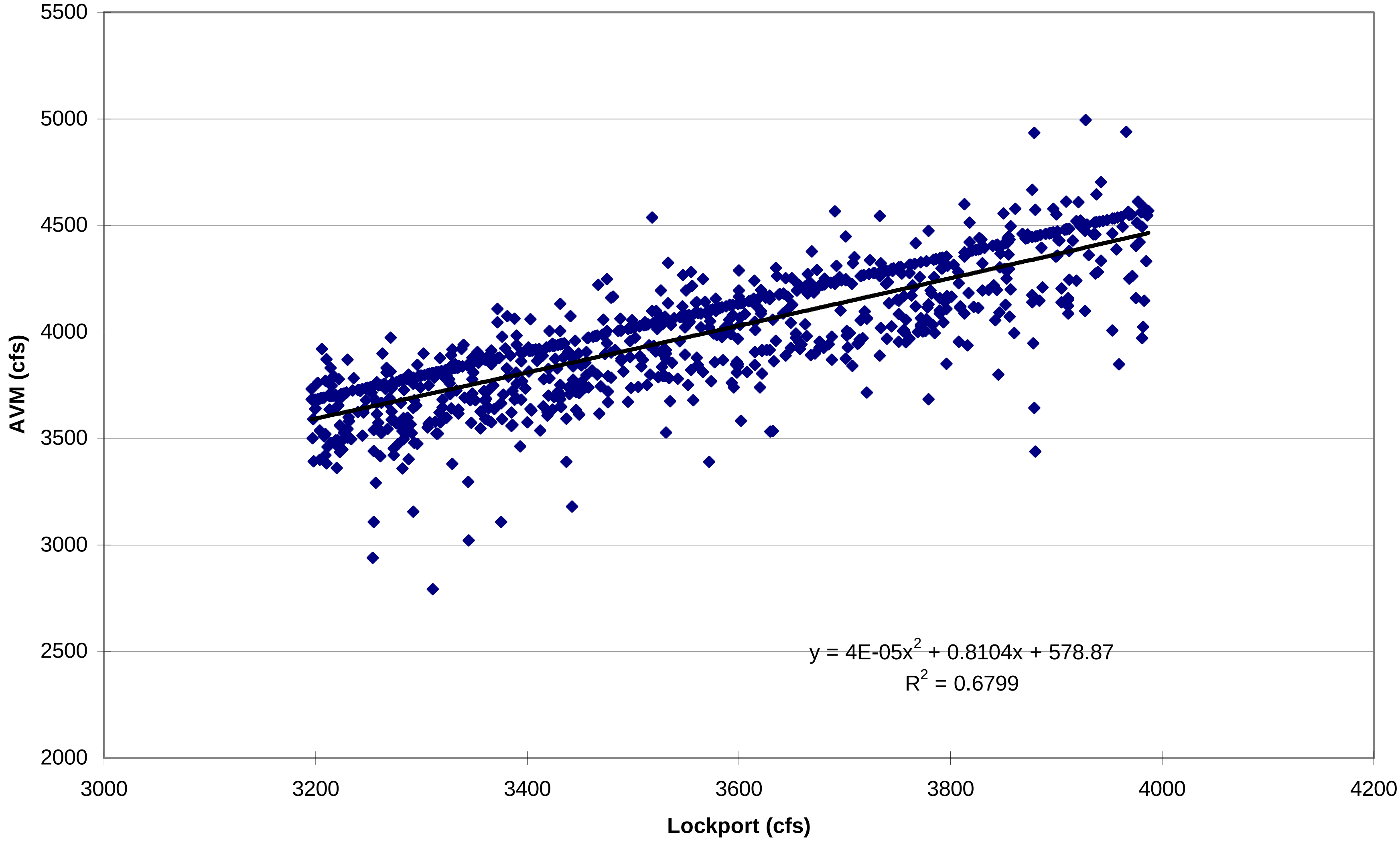


Figure B-11.3 Lockport versus AVM
>4,000 cfs, <=5,000 cfs

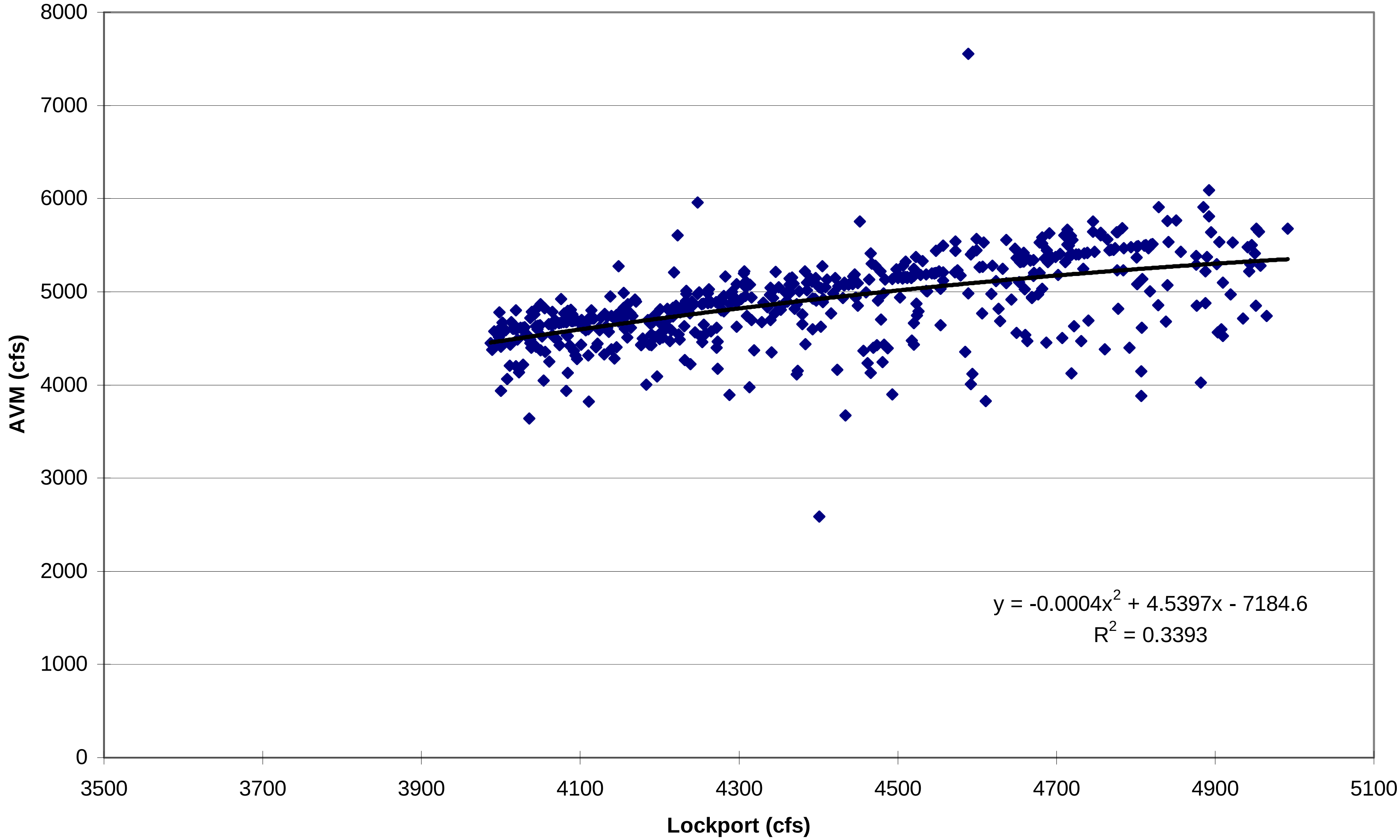
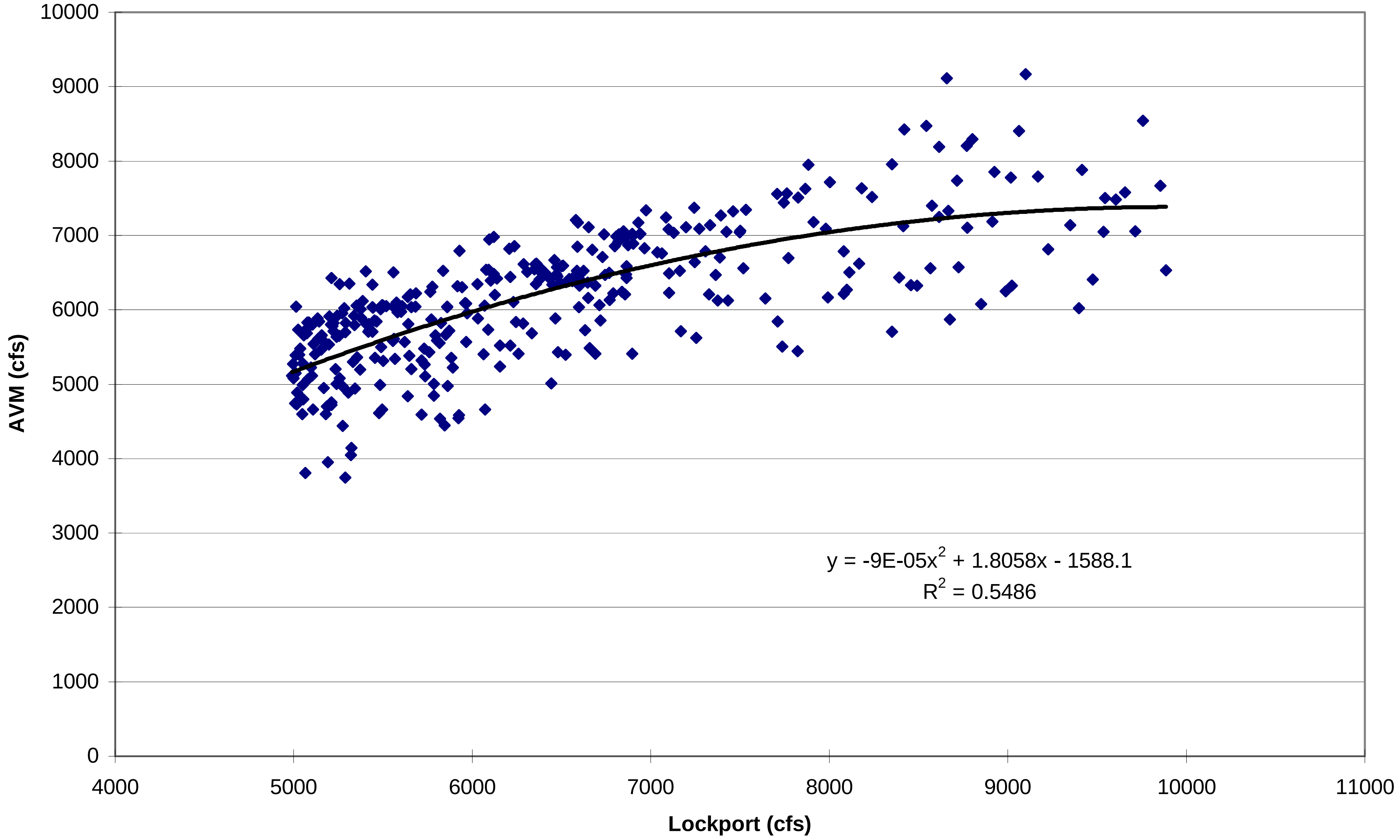


Figure B-11.4 Lockport versus AVM
>5,000 cfs, <=10,000 cfs



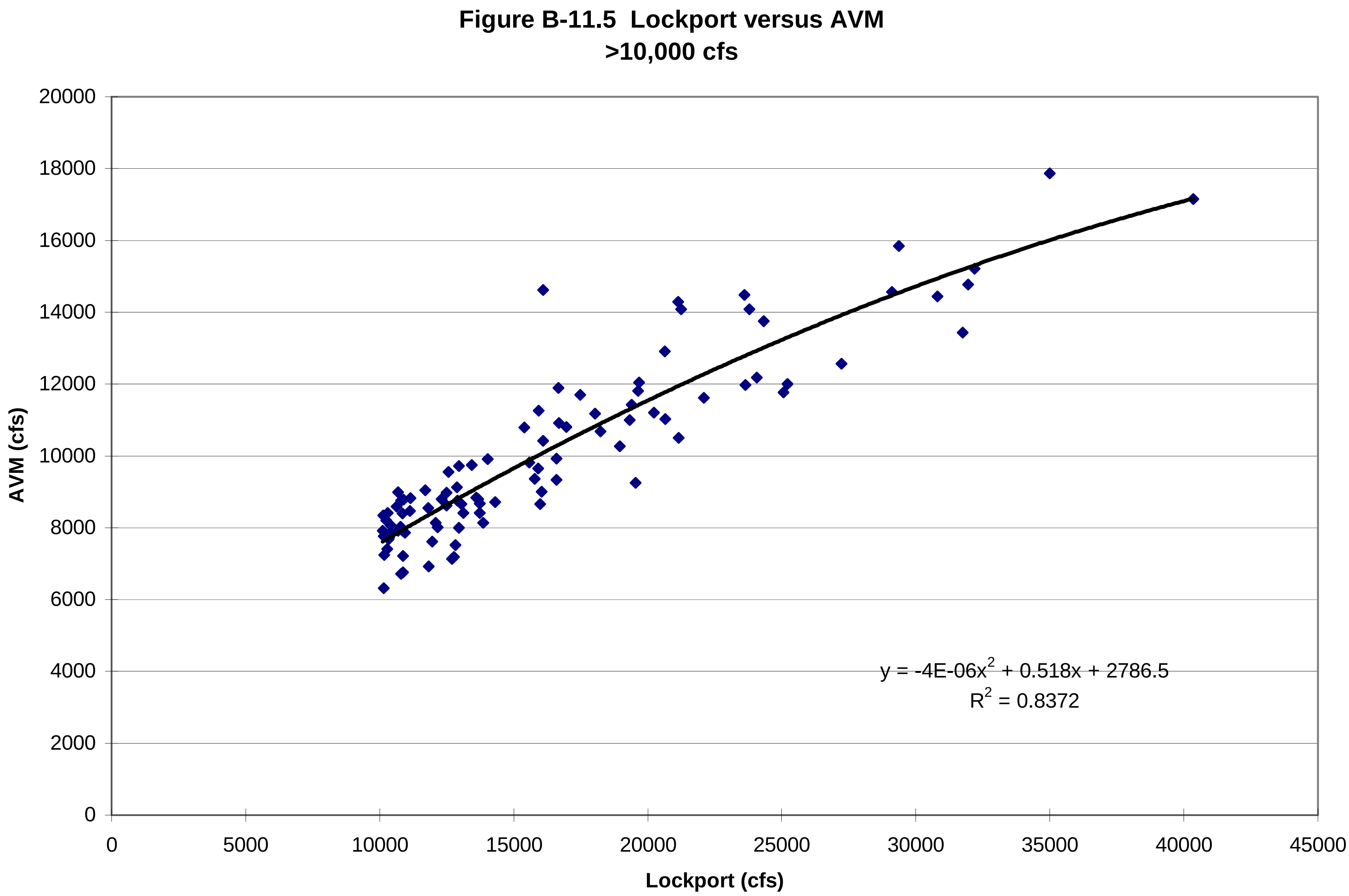


Figure B-11.5 Lockport versus AVM
>10,000 cfs
AVM (cfs)
Lockport (cfs)
y = -4E-06x2 + 0.518x + 2786.5
R2 = 0.8372
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
0
5000
10000
15000
20000
25000
30000
35000
40000
45000

Attachment B-12

Diversion Accounting Estimates

Table B-12.1 Diversion Accounting Estimates

Description	WY 86	WY 87	WY 88	WY 89	WY 90	WY 91	WY 92	WY 93	WY 94	WY 95
AVM Record	4113	4028	3537	3515	3749	3713	3452	4074	3088	3235
AVM Record (estimate)	4058	4003	3483	3571	3712	3972	3624	4368	3275	3478
Diversion Above Gage	0	2	1	1	1	1	2	1	1	1
Total Canal Flow	4113	4030	3538	3516	3750	3714	3454	4075	3089	3236
Groundwater	128	120	110	82	102	116	110	110	110	110
Indiana Pumpage	82	82	31	28	28	29	30	44	44	44
Des Plaines Runoff	180	146	106	135	192	200	177	162	162	162
Federal Facilities	2	4	23	2	2	2	2	5	5	5
Total Deductions	392	352	270	247	324	347	319	321	321	321
By-Passed Flows	30	96	109	108	106	117	192	192	192	192
Accountable Flows	3751	3774	3377	3377	3532	3484	3327	3946	2960	3107
Illinois Pumpage	1724	1805	1906	1792	1755	1819	1785	1794	1875	1806
Consumptive Use	150	157	166	156	153	158	155	156	163	157
Watershed Runoff	877	812	520	707	873	1041	848	1354	642	745
Period of Record Runoff	976	844	572	740	806	870	716	1343	616	722
Lockages	179	146	97	84	72	89	83	92	118	107
Leakages	311	271	271	271	356	357	342	255	204	204
Navigation Makeup	142	157	73	52	46	37	43	59	34	71
Discretionary Flow	302	314	352	264	305	315	293	331	308	309
Direct Diversions	934	888	793	671	779	798	761	737	664	691
Component Flows	3385	3348	3053	3014	3254	3500	3239	3729	3018	3085
Imbalance	366	426	324	363	278	-16	88	217	-58	22

Notes: 92: adjusted for tunnel flood, less 326 cfs
93-95: deductions 86-91 averages used, 92 by-passed flows used
86-95: leakage values are estimates
93-95: watershed runoff values are estimates

93-94: pumpage and deductions are preliminary values
95: pumpage and deductions, 86-94 averages used
86-94: consumptive loss is 8.7% of pumpage

Table B-12.2 Diversion Estimate Regression Analysis

Regression Statistics	
Multiple R	0.95
R Square	0.90
Adjusted R Square	0.87
Standard Error	111.19
Observations	10.00

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	744446	372223	30.11	0.00036
Residual	7	86542	12363		
Total	9	830988			

Regression Coefficients

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	-378.76	741.82	-0.51	0.63	-2132.89	1375.37
AVM	1.12	0.28	4.01	0.01	0.46	1.78
Runoff	-0.45	0.43	-1.06	0.33	-1.47	0.56

Attachment B-13

Lockport – Runoff – Romeoville Comparisons

Table B-13.1 Diversion Estimates							
Water Year	Lockport	AVM-EST	Runoff	%Ro/Lp	%Ro/AVM	Diversion	Deviation
1951	3540	3928	838	24	21	3651	451
1952	3530	3943	821	23	21	3675	926
1953	3338	3764	495	15	13	3622	1,348
1954	3425	3827	608	18	16	3641	1,789
1955	3563	3913	801	22	20	3650	2,239
1956	3488	3923	453	13	12	3820	2,859
1957	4554	4592	726	16	16	4446	4,105
1958	3460	3891	505	15	13	3759	4,664
1959	3571	3986	649	18	16	3801	5,265
1960	3483	3891	764	22	20	3643	5,708
1961	3562	3957	610	17	15	3786	6,294
1962	3302	3724	621	19	17	3520	6,614
1963	3416	3847	321	9	8	3793	7,207
1964	3330	3757	372	11	10	3669	7,676
1965	3414	3820	827	24	22	3535	8,011
1966	3322	3643	812	24	22	3342	8,152
1967	3351	3710	822	25	22	3413	8,365
1968	3356	3696	677	20	18	3463	8,629
1969	3796	4178	861	23	21	3920	9,349
1970	3419	3721	952	28	26	3367	9,516
1971	3342	3648	668	20	18	3413	9,729
1972	3738	3995	907	24	23	3695	10,224
1973	3590	3819	1173	33	31	3377	10,400
1974	3318	3452	1164	35	34	2968	10,169
1975	3539	3647	1032	29	28	3246	10,215
1976	3309	3471	797	24	23	3156	10,171
1977	3182	3471	530	17	15	3277	10,248
1978	3349	3613	752	22	21	3336	10,384
1979	3677	3771	945	26	25	3425	10,609
1980	3287	3612	711	22	20	3353	10,762
1981	3343	3587	839	25	23	3267	10,829
1982	3318	3589	863	26	24	3258	10,888
1983	3983	4011	1302	33	32	3533	11,221
1984	3528	3812	962	27	25	3463	11,484
1985	3601	3806	885	25	23	3492	11,776
1986	3725	4016	976	26	24	3687	12,263
1987	3780	3944	844	22	21	3666	12,728
1988	3102	3485	572	18	16	3274	12,802
1989	3334	3526	739	22	21	3244	12,846
1990	3557	3646	806	23	22	3349	12,995
1991	3825	3839	870	23	23	3536	13,331
1992	3625	3860	716	20	19	3629	13,761
1993	4733	4573	1343	28	29	4146	14,706
1994	3215	3528	616	19	17	3302	14,808
1995	3174	3444	722	23	21	3160	14,767
Average:	3520	3797	785	22	21	3537	337

Table B-13.2 Regression Errors						
Water Year	AVM	AVM-EST	% Error	Diversion	DIV-EST	% Error
1986	4113	4016	-2.4	3751	3687	-1.7
1987	4028	3944	-2.1	3774	3666	-2.9
1988	3537	3485	-1.5	3377	3274	-3.0
1989	3515	3526	0.3	3377	3244	-3.9
1990	3749	3646	-2.7	3532	3349	-5.2
1991	3713	3839	3.4	3484	3536	1.5
1992	3778	3860	2.2	3653	3629	-0.6
1993	4074	4573	12.3	3946	4146	5.1
1994	3088	3528	14.2	2960	3302	11.5
1995	3235	3444	6.5	3107	3160	1.7
Average:	3683	3786.2	3.0	3496	3499	0.2

Figure B-13.1 Lockport versus AVM Estimates

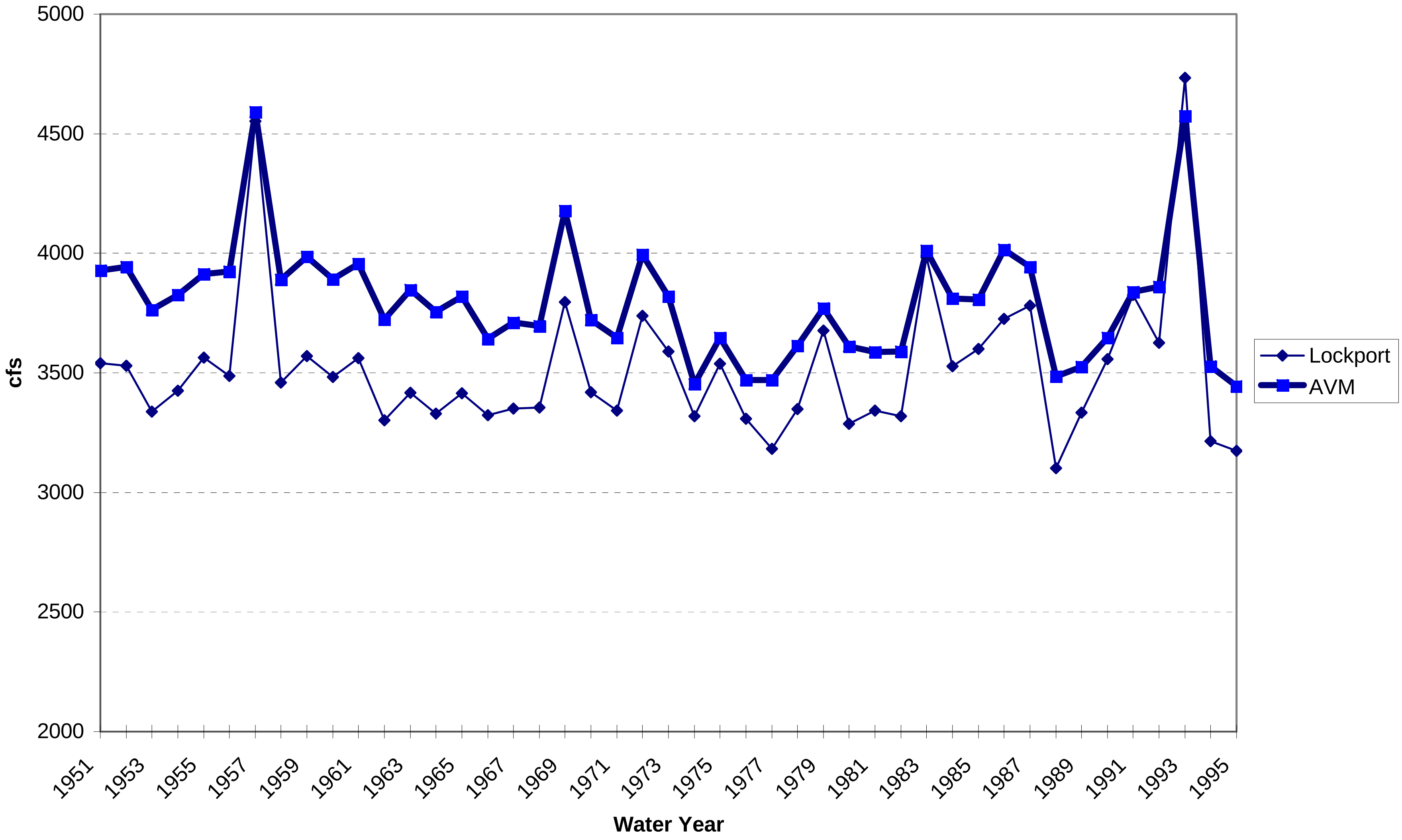
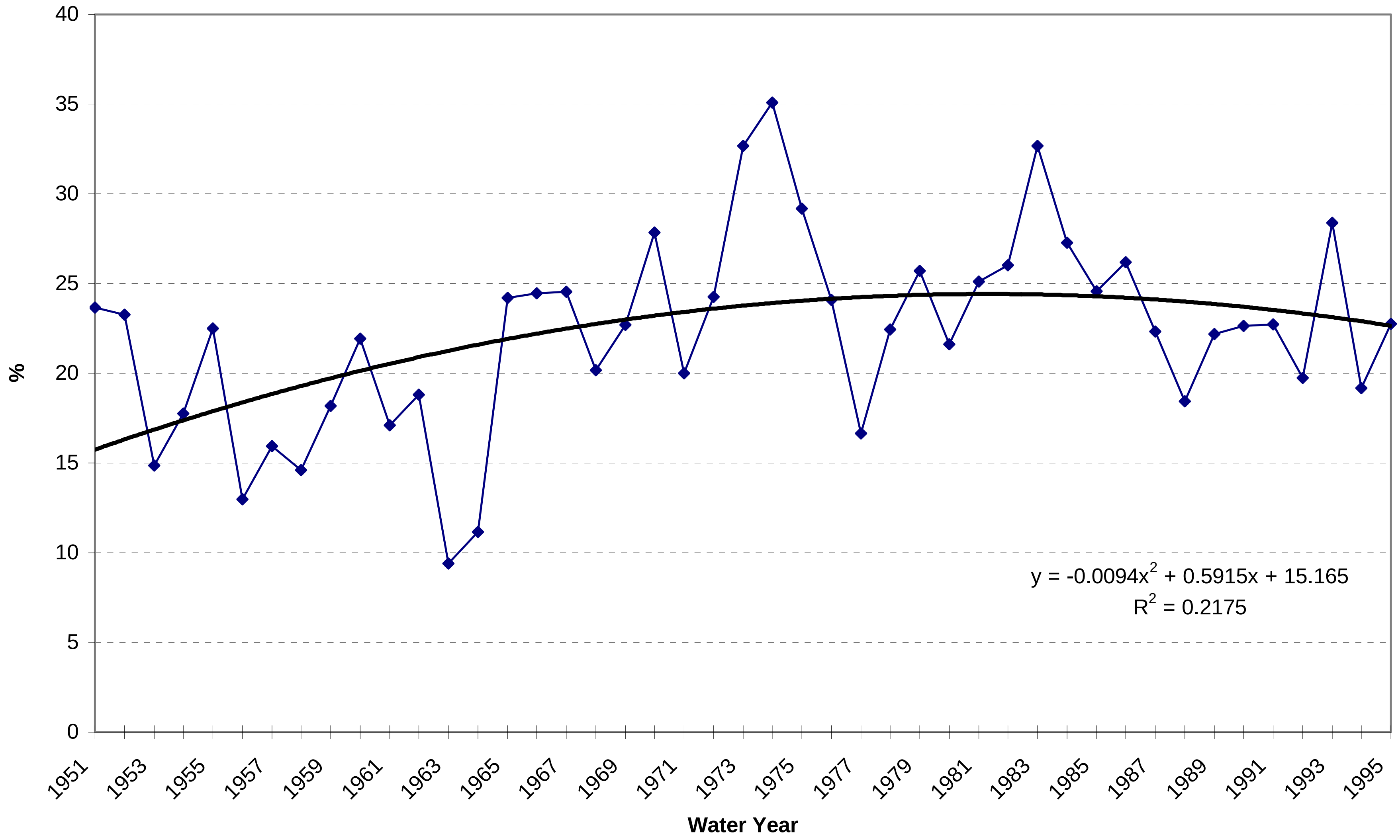


Figure B-13.2 Period of Record Runoff as % of Lockport Flow



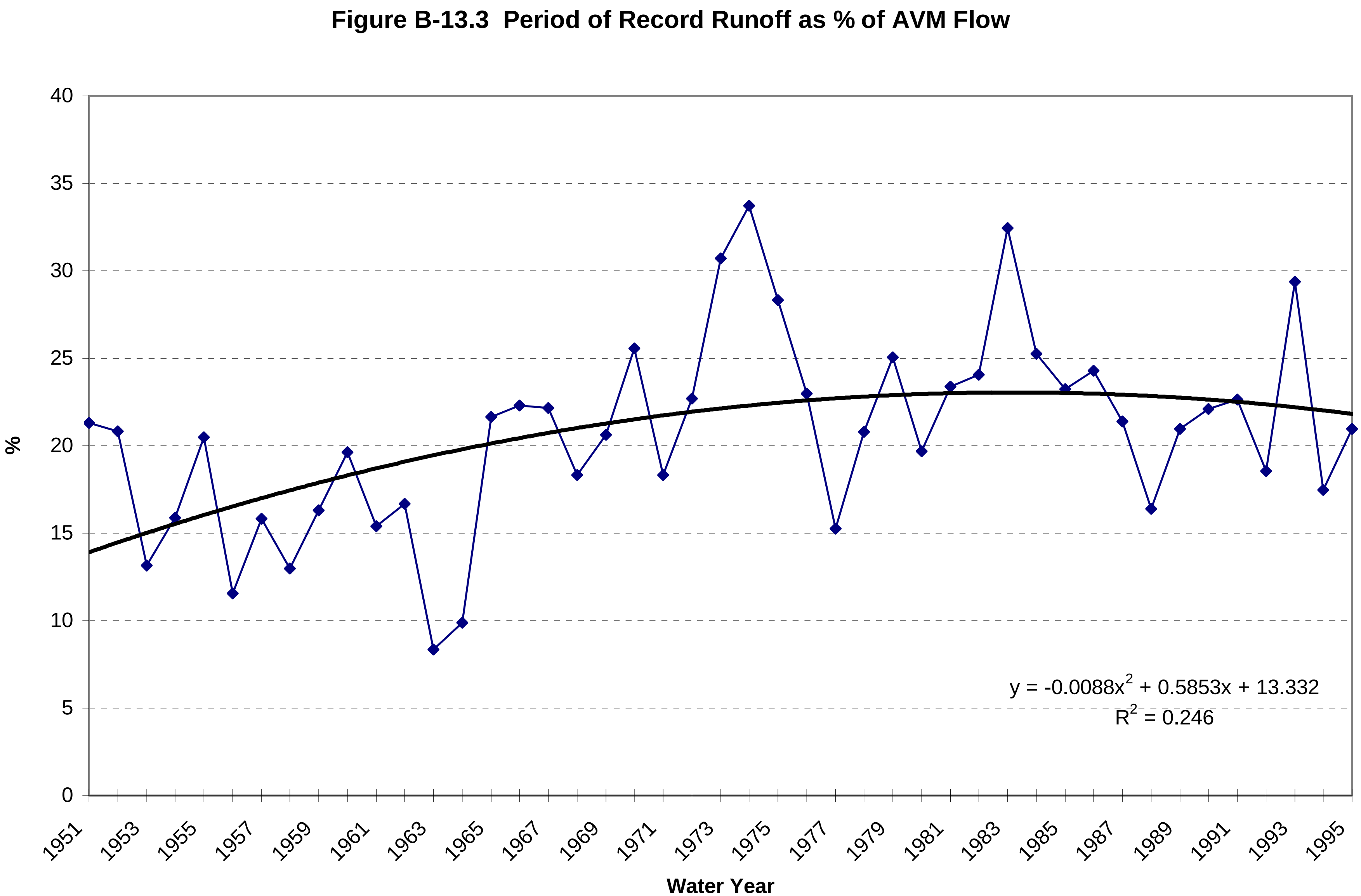


Figure B-13.3 Period of Record Runoff as % of AVM Flow
%
40
35
30
25
20
15
10
5
0
1951
1953
1955
1957
1959
1961
1963
1965
1967
1969
1971
1973
1975
1977
1979
1981
1983
1985
1987
1989
1991
1993
1995
Water Year
y = -0.0088x^2 + 0.5853x + 13.332
R^2 = 0.246

Figure B-13.4 Estimated Diversions

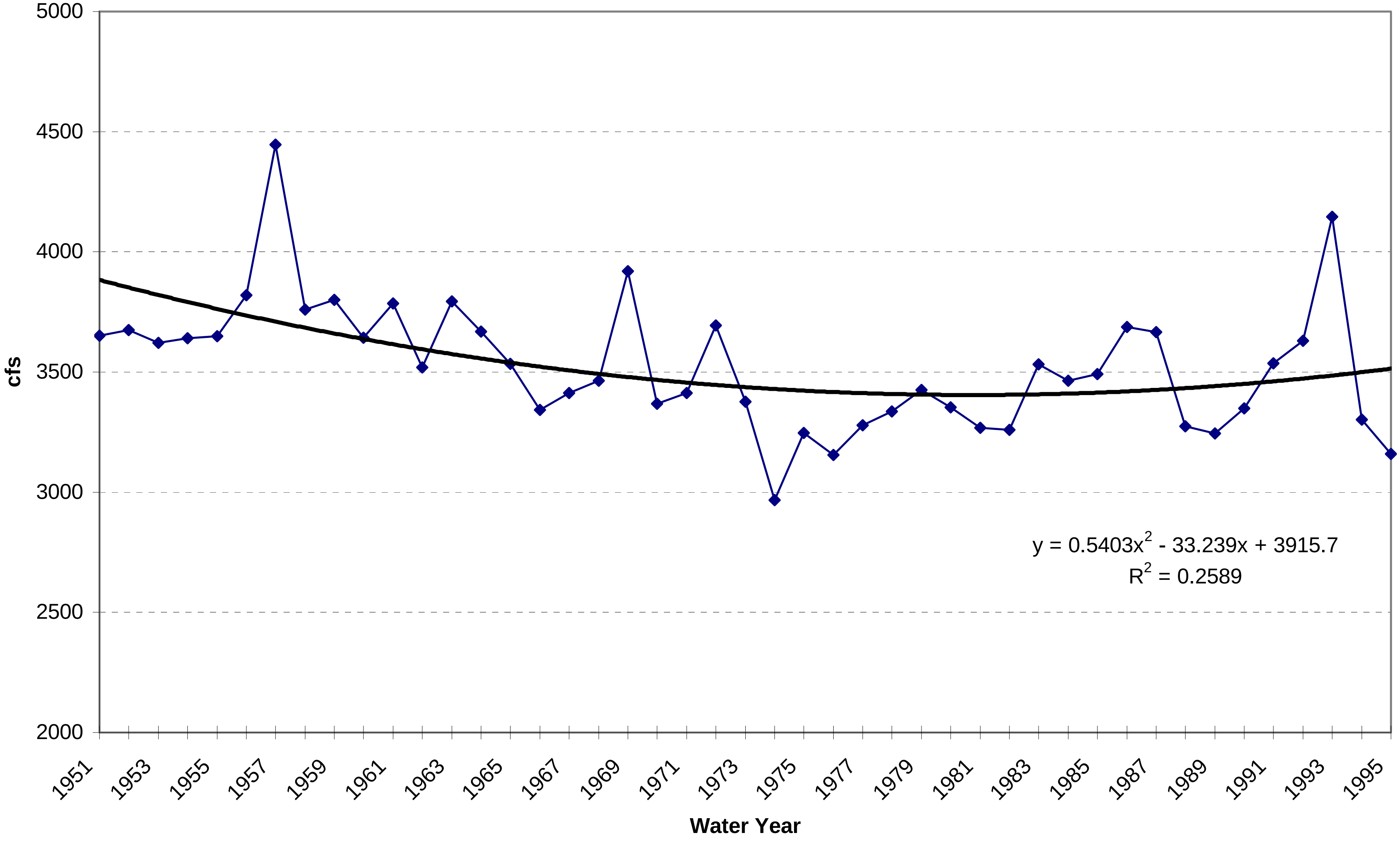
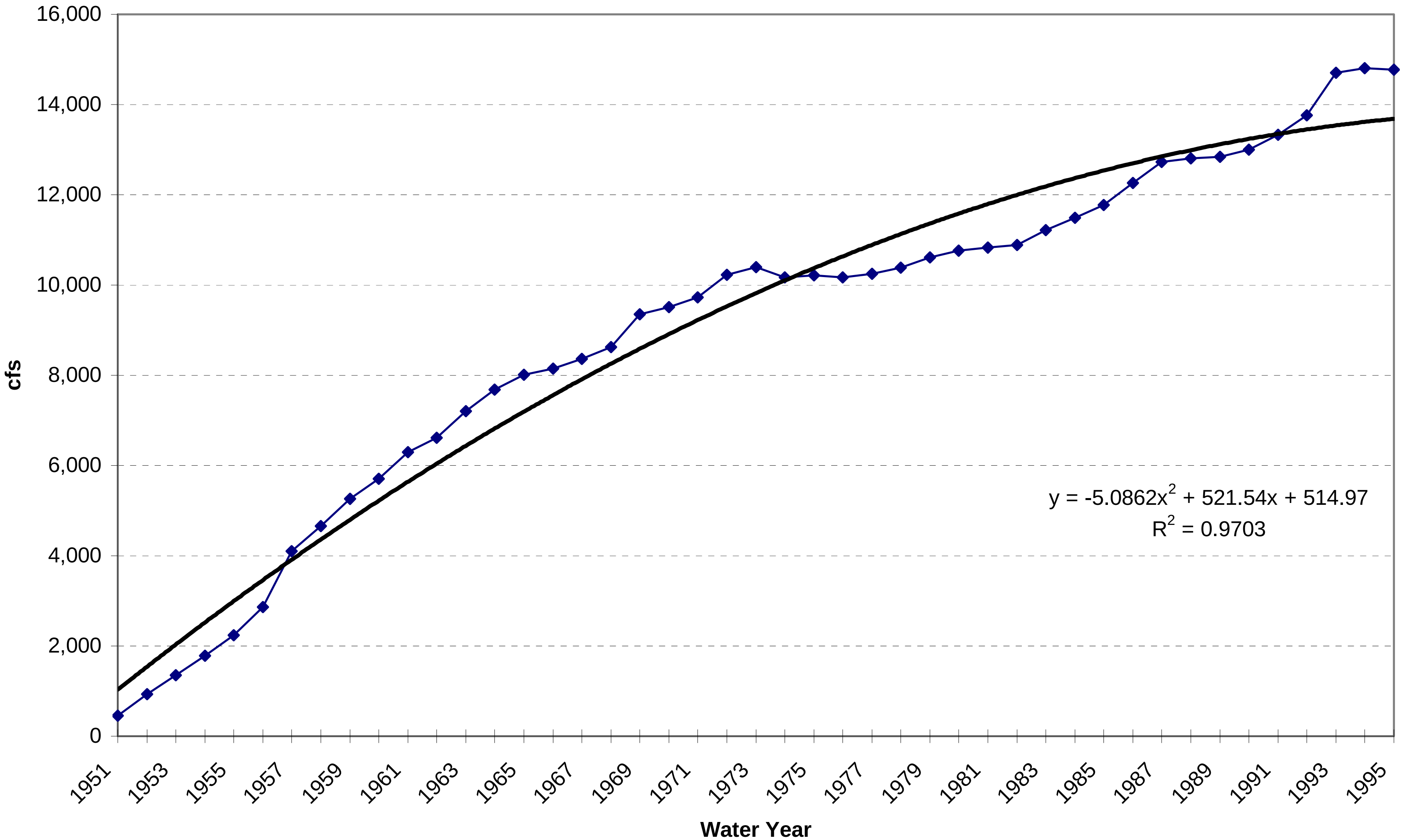


Figure B-13.5 Cummulative Deviations



Attachment B-14

Mass Balance of Rainfall-Runoff Components

Table B-14.1 Mass Balance - Rainfall					
W Year	Rainfall	Rainfall	Total Runoff	Deep Aquifer	Evapotranspiration
	(inches)	(cfs)	(cfs)	(cfs)	(cfs)
1990	38.13	1890.4	805.8	56.7	1027.9
1991	36.41	1805.2	869.7	54.2	881.3
1992	34.82	1721.6	715.9	51.6	954.1
1993	49.77	2467.5	1343.4	74.0	1050.1
1994	27.77	1376.8	616.4	41.3	719.1
Average	37.38	1852.3	870.2	55.6	926.5
% of Rain		100.0	47.0	3.0	50.0

Table B-14.2 Mass Balance - Runoff						
W Year	Total Runoff	Gaged Runoff	Sewer Runoff	Overflow	Ungaged Runoff	Baseflow
	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)
1990	805.8	277.4	216.5	210.1	97.8	4.0
1991	869.7	341.1	216.6	225.8	103.9	4.0
1992	715.9	235.9	222.3	179.8	81.3	4.0
1993	1343.4	482.2	366.3	319.4	171.5	4.0
1994	616.4	239.1	169.4	144.0	62.5	4.0
Average	870.2	315.1	238.2	215.8	103.4	4.0
% of Runoff	100.0	36.2	27.4	24.8	11.9	0.5

Table B-14.3 Mass Balance - Sewers

Inflows to Sewers

	Inflow from Overland Flow			Infiltration from Subsurface Flow		
W Year	Total (cfs)	Sewers (cfs)	TARP/Canal (cfs)	Total (cfs)	Sewers (cfs)	TARP/Canal (cfs)
1990	324.5	124.4	200.1	102.1	92.1	10.0
1991	331.3	117.8	213.5	111.2	98.9	12.3
1992	306.8	133.6	173.2	95.3	88.7	6.6
1993	480.5	186.9	293.6	205.2	179.4	25.8
1994	249.1	109.3	139.8	64.3	60.1	4.2
Average	338.5	134.4	204.1	115.6	103.8	11.8
% of Flow	100.0	39.7	60.3	100.0	89.8	10.2

Outflows from Sewers

	Sewer Flow to WRPs			Overflows to TARP or Canal		
W Year	Total (cfs)	Inflow (cfs)	Infiltration (cfs)	Total (cfs)	Inflow (cfs)	Infiltration (cfs)
1990	216.5	124.4	92.1	210.1	200.1	10.0
1991	216.7	117.8	98.9	225.8	213.5	12.3
1992	222.3	133.6	88.7	179.8	173.2	6.6
1993	366.3	186.9	179.4	319.4	293.6	25.8
1994	169.4	109.3	60.1	144.0	139.8	4.2
Average	238.2	134.4	103.8	215.8	204.1	11.8
% of Flow	100.0	56.4	43.6	100.0	94.5	5.5

Note: 91-94 overflows are approximations

Table B-14.4 Mass Balance - Overflows			
W Year	Overflow	TARP	Canal/River
	(cfs)	(cfs)	(cfs)
1990	210.1	55.0	155.1
1991	225.8	59.1	166.7
1992	179.8	47.1	132.7
1993	319.4	83.6	235.8
1994	144.0	37.7	106.3
Average	215.8	56.5	159.3
% of Flow	100.0	26.2	73.8
Note: 91-94 overflows are approximations			

Appendix C

Consumptive Loss of Domestic Water Supply

1. The methodology that will be used to estimate consumptive loss consists of subtracting the water reclamation plant (WRP) influent from the total water supply (the sum of the pumpage from Lake Michigan plus ground water). The major difficulty in accomplishing this computation is the isolation and removal of the stormwater discharge that flows through the sewer system. Stormwater discharge is a persistent problem in that the resulting inflow and infiltration (I&I) into the sewer system can last for long periods of time. The presence of I&I complicates the computation of consumptive loss because it is part of the runoff and not part of the water supply. Since it is intermingled with the sanitary flow that is influent to the WRPs, the accuracy of the consumptive loss estimate is dependent on the accuracy of the modeled I&I.

2. The procedures used to compute consumptive loss required the use of the rainfall - runoff models described in appendix A, “Period of Record Runoff Analysis.” With the use of these models a variety of techniques were employed in attempting to eliminate the effects of stormwater discharge. The main approach required limiting the comparisons of WRP influent to water supply (i.e. computing consumptive loss) for only dry weather periods. As a sensitivity analysis, a second approach involved merging the results of the dry weather analysis into the entire continuous period analysis. Finally, an additional series of sensitivity analyses were carried out to develop probable bounds on the value of consumptive loss.

Continuous Period Method

3. Estimating consumptive loss for domestic water supply involves subtracting sanitary treatment plant influent from total domestic water supply. Water Years 1991 and 1992 (WY91 and WY92) were analyzed for this study along with influent records at three Metropolitan Water Reclamation District (MWRD) Water Reclamation Plants (WRPs): West Southwest (Stickney); Northside; and Calumet. The total domestic water supply includes water pumped from either Lake Michigan or from groundwater to all users within the service area boundaries of the three WRPs.

4. The water supply from Lake Michigan and groundwater sources tributary to the three WRPs are based on service area boundaries for the three WRPs. Several communities and other entities were divided by the service boundaries. Those users were contacted to determine if all sewage, no sewage, or a portion of their sewage (based on area proportioning) was treated by one or more of the three MWRD WRPs. Annual water supply pumpages were used since most users are not primary diverters, and thus are not required to submit daily pumpage records to the State of Illinois on the State’s LMO-3 forms. Variances in daily and seasonal pumpages were accounted for by using Chicago’s Jardine water purification plant daily pumpages as an index whereby factors were applied

to the annual pumpages of individual users to convert the annual pumpages to daily values.

5. Daily influent, as measured by MWRD to the three WRPs was used in the analysis of consumptive loss. Pumpages from Mainstream and Calumet TARP were not included in the analysis since only sanitary flows are considered. However, Calumet TARP is also used as a conveyance system for sanitary sewage, the extent of which has not been determined, in some separately sewered areas. Since physical boundaries of these areas have not been identified, sanitary sewage flows into TARP are not currently quantifiable and have not been included in the analysis.

6. In estimating the consumptive loss, the total daily simulated sanitary flows (the total measured WRP influent, not including TARP, minus the simulated inflow and infiltration) were subtracted from the total daily derived water supply pumpages for users within the three WRP service areas. The process was carried out for fifteen defined dry weather periods (see next paragraph). Attachment C-1, table C-1.1 and figure C-1.1, show the influent, I&I, sewage, water supply and percent loss for month and each year of the continuous period. In reviewing the information provided in the attachment, it is noted that there are large variances in losses between months. Consumptive losses ranged from 18.2 percent to a negative 4.6 percent. The total water supply of all users within the three WRP service areas during WY91 and WY92 was 1621.9 cfs. The simulated sanitary sewage generated from those same users over the 2-year period was 1524.8 cfs. This results in an average consumptive loss of 6.0 percent (it should be noted that for this analysis the average consumptive loss for a period is computed using the total water supply and the total simulated sanitary flow, not the average of the daily or monthly values).

Dry Weather Procedure

7. To isolate and reduce the effects of stormwater runoff, the dry weather portion of the continuous period was examined in detail. As the effects of the I&I are minimized, the computation of consumptive loss during these periods are the most theoretically and technically sound. The dry weather periods were determined by analyzing output from the hydraulic sewer routing model SCALP (see appendix A). For the purpose of this analysis, dry weather periods were defined as those times when less than 100 cfs of I&I was simulated in the interceptors of the three WRPs. If the day preceding the start of a dry weather period showed a spike in the simulated I&I hydrograph then the first day of the dry weather period also was dropped from consideration. The hydrograph spike is the result of storm water inflow, and the first day after the spike was removed from the dry weather period as a measure of safety to allow for discrepancies in sewer travel times. Dry weather periods that were considered were limited to durations that were greater than or equal to five days. Fifteen separate dry weather periods were identified for the 2-year study period. The periods ranged from 5 to 33 days with an average duration of 11.4 days.

8. Attachment C-2, table C-2.1, shows the influent, I&I, sewage, water supply and percent loss for each of the periods of the dry weather period. The table C-2.2 and figure C-2.1 show the monthly averages. In reviewing the information provided in the attachment, it is noted that there are large variances in losses between individual dry weather periods. Consumptive losses ranged from 25.4 percent to a negative 0.4 percent. One of the fifteen dry weather periods resulted in negative losses. The average daily water supply pumpage during the dry weather periods was 1879.7 cfs, while the average daily sanitary sewage portion of the WRP influent was 1594.8 cfs. This results in an average consumptive loss of 15.2 percent.

Potential Inaccuracies in Consumptive Loss Modeling

9. As is the case with any consumptive loss study, measurement errors, simplifications, and assumptions can lead to potential inaccuracies. Prior to detailing the results of the sensitivity analyses it is appropriate to first discuss the limitations in the continuous period analyses, especially the dry weather variant. The wide variances between individual dry weather periods losses, as well as the monthly losses for the full continuous period, indicate that there are a variety of reasons why the actual consumptive loss could be greater or less than the computed values.

10. There are three main reasons that would indicate that the consumptive loss could be greater. The first is in regards to the actual interceptors that convey both sanitary sewage and storm water runoff (in the form of I&I) to the WRPs. Part of the actual flow conveyed to the WRPs, even during “dry weather” periods, is a result of I&I in the sewers. Review of the SCALP output reveals several days when the total I&I in the sewers is zero or approaches zero. However, it is likely that infiltration is constantly entering low level interceptors that are well below the groundwater table. Not fully accounting for this infiltration would result in larger estimates of the sanitary sewage portion of the WRP influent and thus a low biased estimate of consumptive loss. Potentially this could have a significant impact if infiltration into these low level interceptors is large. For instance, if the total low level infiltration is 18.8 cfs, the effect is an underestimation of consumptive loss of one percent for the dry weather period analysis. Additional information regarding the WRP influent pumpage records is also necessary since there is a wider than expected variance in daily pumpages during the dry weather periods.

11. The second reason why there may be a greater consumptive loss is the incomplete groundwater data used for this study. The Illinois State Water Survey (ISWS) collects groundwater withdrawal data from most, but not all users. By the ISWS’s own admission, on any given year up to ten percent of the withdrawals are not reported by the users (composed of communities and industries). Additionally, there is a threshold below which it is not required to submit groundwater withdrawal information to the ISWS. However, this is probably not a large source of error since the average reported groundwater pumpages that are tributary to the three WRPs were 17.4 cfs over WY91 and WY92.

12. The third reason why the consumptive loss may be greater is a result of the conversion of annual water supply values to daily values. It is likely that using Chicago's Jardine water purification plant as the basis for daily factors may tend to underestimate suburban water use during the warm weather months when demand is typically higher. This is the result of the Chicago suburbs having a larger percentage of residential land use. Since a majority of the dry weather periods for this study happened to occur in the warm weather months it is possible that the increased water use and losses are not fully accounted for by the daily factors.

13. There also are three primary reasons why the consumptive loss could be less. The first reason is that sanitary sewage flow into Calumet TARP was not considered in this study. This is the result of the lack of additional data surrounding the areas and connections of sewers to Calumet TARP in some separately sewered areas within the Calumet WRP service basin. The second reason is that the O'Hare flow transfer was not considered in this study. The O'Hare flow transfer is the transfer of sewer flows from the Kirie WRP (formerly the O'Hare WRP) and Egan WRP service areas to the Northside WRP via the Howard 6 interceptor. MWRD estimated this flow to be 25 cfs during WY91 and WY92. However, a breakdown of constituent flow is not available at this time and as a result it is not possible to know how much of this flow was a result of I&I or sanitary sewage. For this study the full 25 cfs was subtracted from the Northside record. However, it is quite possible that the transfer of flow during the dry weather periods was less than the annual estimate of 25 cfs. The third reason supporting a lower consumptive loss is that the dry weather periods happened to occur primarily during the warm weather months when domestic water use and consumptive losses are greater. This is addressed in an evaluation of the continuous period of record study presented later in this Appendix.

14. There are also two reasons that could support both the case for greater as well as reduced consumptive losses. The first is with respect to the actual WRP influent pumpages from interceptors during dry weather periods. If pumpages are regulated based on head within the interceptor, then there is a time delay in the measurement of influent as compared to the simulated flows. If this delay is significant it could result in additional I&I being recorded much later than it is modeled (i.e. I&I from a previous storm being measured during a defined dry weather period). This would support the notion of a greater consumptive loss. Conversely, this delay may also result in recorded influent pumpages that are deficient of sanitary sewage flow in the later stages of a dry weather period. This would support the notion of a reduced consumptive loss. The second item that could support either higher or lower losses is the fact that daily pumpages were derived values based on the Jardine daily record. Using the Jardine daily pumpages as an index, annual water supplies were converted to daily values. In some cases this may overestimate water supply while in other cases, especially during the warm weather months, this procedure may underestimate water supply.

Merged Period Sensitivity Analyses

15. In reviewing the dry weather and continuous period results there are a number of factors that become readily apparent:

- The dry weather results are technically more sound because the influence of I&I has been minimized.

- The dry weather analysis produces results almost exclusively for the "warm weather" months, May through September. There were too few dry weather observations outside of this period to draw any conclusions for the remainder of the year.

- For the months where there are sufficient observations, May through September, the dry weather results produced significantly higher consumptive losses.

16. In an effort to fully utilize the information from the dry weather procedures, two "merged period" analyses have been completed, and the results shown in attachment C-3. In the first of the merged period procedures, the warm month averages (May through September) for the continuous period have been replaced by the dry period monthly averages (see table C-3.1 and figure C-3.1). The results of this effort increased the average water supply to 1646.2 cfs, decreased the average simulated sewage to 1511.9 cfs, and increased the average consumptive loss to 8.2%.

17. The second of the merged records procedures involved extending the first method by updating the cold month averages (January through April and October through December) in the merged period method, using the percent increase between dry weather and continuous period results (140% increase in consumptive use - see attachment C-3, table C-3.2 and figure C-3.2). Because of the smaller water supply usage during cold weather, the results of this effort were minimal. The average water supply was not changed, however the average simulated sewage was reduced to 1502.6 cfs, and this increased the average consumptive loss to 8.7%.

Continuous Period Sensitivity Analyses

18. To further explore the potential effects of reducing the effects of I&I, and to eliminate the obvious inconsistencies of having negative consumptive losses, a number of sensitivity runs were performed using the continuous period results. All results are shown in attachment C-4. The following methods were applied to the continuos period, whereby the results were adjusted for all days (and only days) when the consumptive loss was negative:

- Negative loss were been eliminated from the record.

- The simulated sanitary flow was decreased so that the consumptive loss is equal to 0%.

- The simulated sanitary flow was decreased so that the consumptive loss is equal to 3%.

- The simulated sanitary flow was decreased so that the consumptive loss is equal to 5%.

Conclusions

19. Presented in this appendix is a computation of consumptive loss of municipal water for use in the Lake Michigan diversion accounting system. The computation is based on subtracting the water reclamation plant influent from the total water supply. The limitations and potential errors in utilizing the continuous period model in computing consumptive losses are fully recognized. When compared to the accepted "book" value (normally given as 10%), or the International Joint Commission determination (16% for 1975 and 10% for 2000), the computed value of consumptive use (6.0%) low. This conclusion is also supported by the negative values in some of the daily and monthly computations of consumptive loss. One of the rationales for this conclusion is that the continuous period model failed to separate out all of the inflow and infiltration (I&I due to stormwater runoff) from the sewage plant influent.

20. In an effort to resolve the high I&I rates a dry weather analysis was carried out. Additionally, a number of sensitivity analyses were also undertaken. The problem with all of the methodologies utilized in reducing the impacts of I&I, is that they are based on adjusting values without a specific consistent physical basis. For example, in the merged-ratio analysis the values of all of the monthly losses were adjusted based on a comparison with dry-weather flows. However, there is no way of knowing if the errors in the dry-weather months are uniform across the year. With this caveat, the table below enumerates the results from all of the evaluations:

Methodology	% Consumptive Loss
Continuous Period Analysis	6.0
Dry Weather Periods	15.2
Continuous - Dry Weather Merged Analysis	8.2
Continuous - Dry Weather Merged-Ratio Analysis	8.7
Continuous Period - Positive Values	16.1
Continuous Period - 0% Minimum Value	9.8
Continuous Period - 3% Minimum Value	11.1
Continuous Period - 5% Minimum Value	12.1

21. In general, it is difficult to select a potential range of consumptive use values from this analysis. However, if you discount the extreme values from the above table (6%, 15.2%, and 16.1%) a potential range of approximately 8% to 12% remains. Again, it

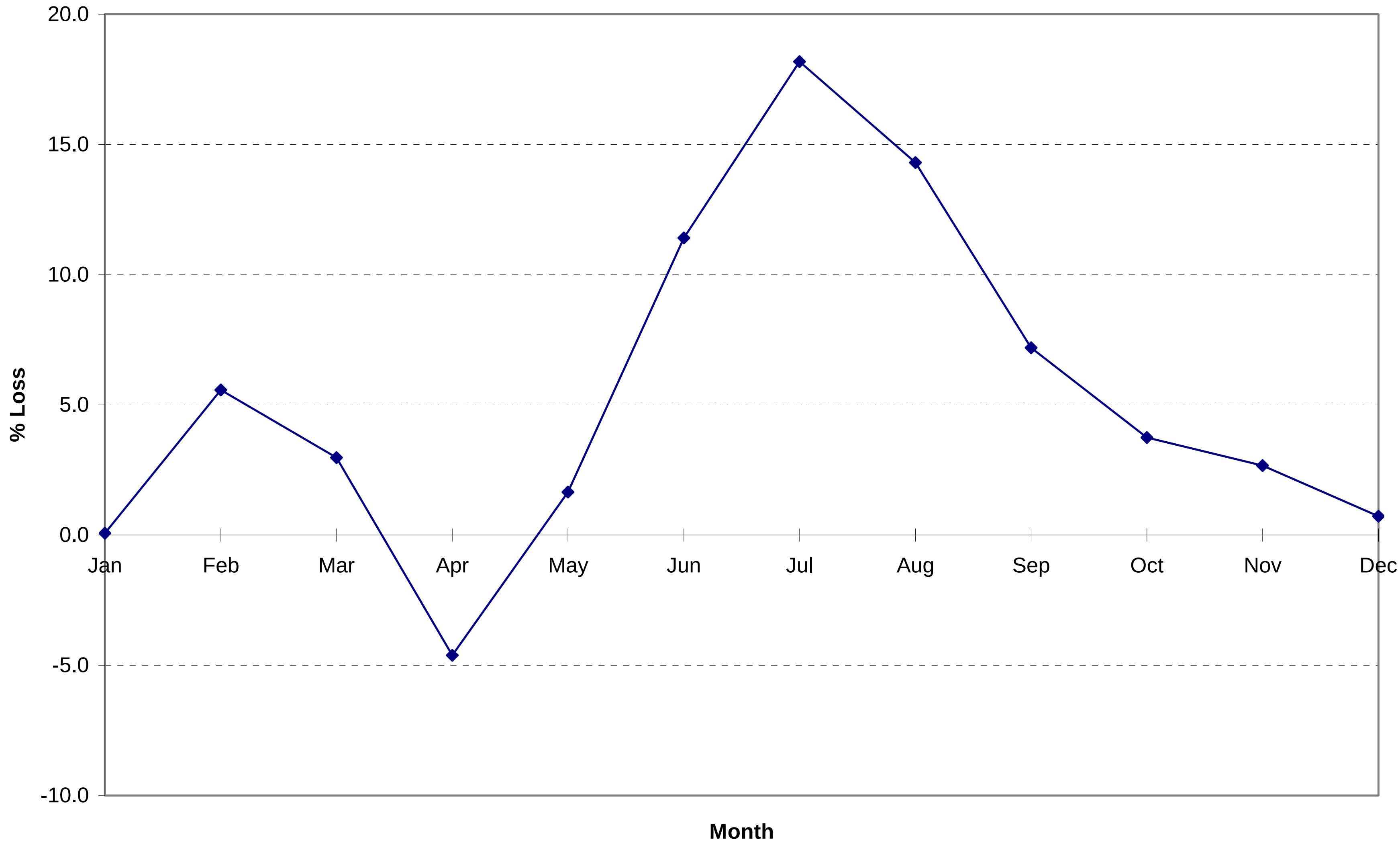
should be noted that this range is low compared to other “accepted” consumptive use values of 10% to 16%.

22. As a final note, a consumptive use estimate was required for the computation of the imbalance in the sensitivity analysis (appendix B). Without any recommendation, the value from the dry-weather merged-ratio analysis (8.7%) was utilized.

Attachment C-1
Continuous Period

Table C.1-1 CONTINUOUS PERIOD SENSITIVITY ANALYSIS					
Month	Measured	Simulated	Simulated	Measured	Consumptive
	WRP	WRP	WRP	Water	Loss
	Total Influent	I&I	San. Sewage	Supply	
	cfs	cfs	cfs	cfs	%
Jan	1832.4	342.5	1490.0	1491.0	0.1
Feb	1762.6	363.9	1398.7	1481.2	5.6
Mar	2036.9	617.6	1419.3	1462.9	3.0
Apr	2027.3	500.3	1527.0	1459.6	-4.6
May	1813.3	224.2	1589.1	1615.8	1.7
Jun	1738.8	73.3	1665.4	1879.9	11.4
Jul	1737.4	118.9	1618.5	1978.3	18.2
Aug	1745.5	115.2	1630.3	1902.4	14.3
Sep	1794.2	235.6	1558.5	1679.3	7.2
Oct	1990.9	506.3	1484.6	1542.3	3.7
Nov	2082.9	639.7	1443.2	1482.9	2.7
Dec	1972.6	507.1	1465.5	1476.1	0.7
91 Average	1921.0	358.6	1562.4	1660.5	5.9
92 Average	1835.8	348.6	1487.2	1583.3	6.1
91-92 Average	1878.3	353.6	1524.8	1621.9	6.0

Figure C.1-1 Continuous Period



Attachment C-2

Dry Weather Periods

Table C-2.1 DRY WEATHER PERIOD SENSITIVITY ANALYSIS					
Date	Measured	Simulated	Simulated	Measured	Consumptive
	WRP	WRP	WRP	Water	Loss
	Total Influent	I&I	San. Sewage	Supply	
	cfs	cfs	cfs	cfs	%
9-Jul-91	1744.5	3.4	1741.1	1926.3	9.6
10-Jul-91	1678.8	2.1	1676.7	2081.5	19.4
11-Jul-91	1731.0	19.4	1711.6	2053.7	16.7
12-Jul-91	1627.1	1.0	1626.1	2035.4	20.1
13-Jul-91	1465.1	0.7	1464.4	1902.7	23.0
14-Jul-91	1417.8	0.5	1417.3	1923.4	26.3
15-Jul-91	1614.3	0.3	1614.0	2134.2	24.4
16-Jul-91	1748.8	0.2	1748.6	2310.6	24.3
17-Jul-91	1764.1	0.1	1764.0	2346.0	24.8
18-Jul-91	1979.5	0.0	1979.5	2447.6	19.1
19-Jul-91	1893.6	0.0	1893.6	2584.1	26.7
20-Jul-91	1984.9	0.0	1984.9	2579.8	23.1
21-Jul-91	1809.8	0.1	1809.6	2191.8	17.4
Average	**1727.6**	**2.1**	**1725.5**	**2193.6**	**21.3**
24-Jul-91	1706.2	0.4	1705.8	2197.2	22.4
25-Jul-91	1568.5	0.1	1568.4	2105.6	25.5
26-Jul-91	1513.2	0.0	1513.1	2044.9	26.0
27-Jul-91	1342.9	0.0	1342.9	2012.2	33.3
28-Jul-91	1383.7	0.0	1383.7	1948.3	29.0
29-Jul-91	1476.9	0.6	1476.4	1963.3	24.8
30-Jul-91	1367.2	0.0	1367.2	2064.0	33.8
31-Jul-91	1665.8	0.0	1665.8	2201.9	24.3
1-Aug-91	1615.4	0.0	1615.4	2270.3	28.8
2-Aug-91	1677.0	0.3	1676.6	2370.6	29.3
3-Aug-91	1644.0	0.1	1643.9	2028.6	19.0
4-Aug-91	1429.7	0.0	1429.6	1918.3	25.5
5-Aug-91	1665.9	0.0	1665.9	1993.5	16.4
6-Aug-91	1403.2	0.0	1403.2	2024.5	30.7
7-Aug-91	1716.6	11.0	1705.5	1900.1	10.2
Average	**1545.1**	**0.8**	**1544.2**	**2069.6**	**25.4**
10-Aug-91	1662.3	16.0	1646.2	1762.2	6.6
11-Aug-91	1473.4	9.8	1463.6	1789.0	18.2
12-Aug-91	1717.8	6.0	1711.8	1931.7	11.4
13-Aug-91	1481.5	3.4	1478.1	2008.6	26.4
14-Aug-91	1625.2	1.7	1623.5	2040.7	20.4
15-Aug-91	1684.8	0.7	1684.0	2064.0	18.4
16-Aug-91	1688.0	0.3	1687.8	2027.3	16.7
17-Aug-91	1614.9	11.4	1603.5	1855.9	13.6

Table C.2-1 cont' DRY WEATHER PERIOD SENSITIVITY ANALYSIS					
Date	Measured	Simulated	Simulated	Measured	Consumptive
	WRP	WRP	WRP	Water	Loss
	Total Influent	I&I	San. Sewage	Supply	
	cfs	cfs	cfs	cfs	%
21-Aug-91	1448.1	11.3	1436.8	1836.1	21.7
22-Aug-91	1747.0	6.3	1740.7	1953.6	10.9
23-Aug-91	1622.2	3.4	1618.8	1956.4	17.3
24-Aug-91	1510.3	1.8	1508.5	1939.6	22.2
25-Aug-91	1589.6	0.9	1588.7	2001.9	20.6
26-Aug-91	1818.3	0.4	1817.9	2265.1	19.7
27-Aug-91	1621.3	0.2	1621.1	2315.2	30.0
28-Aug-91	1878.9	0.1	1878.8	2341.0	19.7
Average	**1654.4**	**3.1**	**1651.4**	**2076.1**	**20.5**
16-Sep-91	1429.2	16.8	1412.3	1720.1	17.9
17-Sep-91	1548.0	11.3	1536.7	1712.2	10.3
18-Sep-91	1645.1	8.2	1636.9	1641.9	0.3
19-Sep-91	1220.6	6.3	1214.2	1600.7	24.1
20-Sep-91	1422.2	5.0	1417.1	1633.0	13.2
21-Sep-91	1309.7	4.0	1305.7	1623.4	19.6
Average	**1429.1**	**8.6**	**1420.5**	**1655.2**	**14.2**
24-Sep-91	1549.0	3.9	1545.1	1595.9	3.2
25-Sep-91	1472.1	3.5	1468.6	1587.5	7.5
26-Sep-91	1477.9	2.8	1475.0	1606.5	8.2
27-Sep-91	1456.4	2.1	1454.3	1612.9	9.8
28-Sep-91	1467.1	1.6	1465.5	1612.2	9.1
29-Sep-91	1455.4	1.1	1454.3	1597.9	9.0
30-Sep-91	1517.7	0.7	1517.0	1693.3	10.4
1-Oct-91	1543.1	35.3	1507.8	1663.1	9.3
Average	**1492.3**	**6.4**	**1485.9**	**1621.2**	**8.3**
7-May-92	1517.5	98.7	1418.7	1549.9	8.5
8-May-92	1514.8	92.3	1422.6	1592.6	10.7
9-May-92	1488.6	85.9	1402.7	1611.5	13.0
10-May-92	1567.3	79.6	1487.7	1595.4	6.8
11-May-92	1540.5	73.5	1467.1	1678.4	12.6

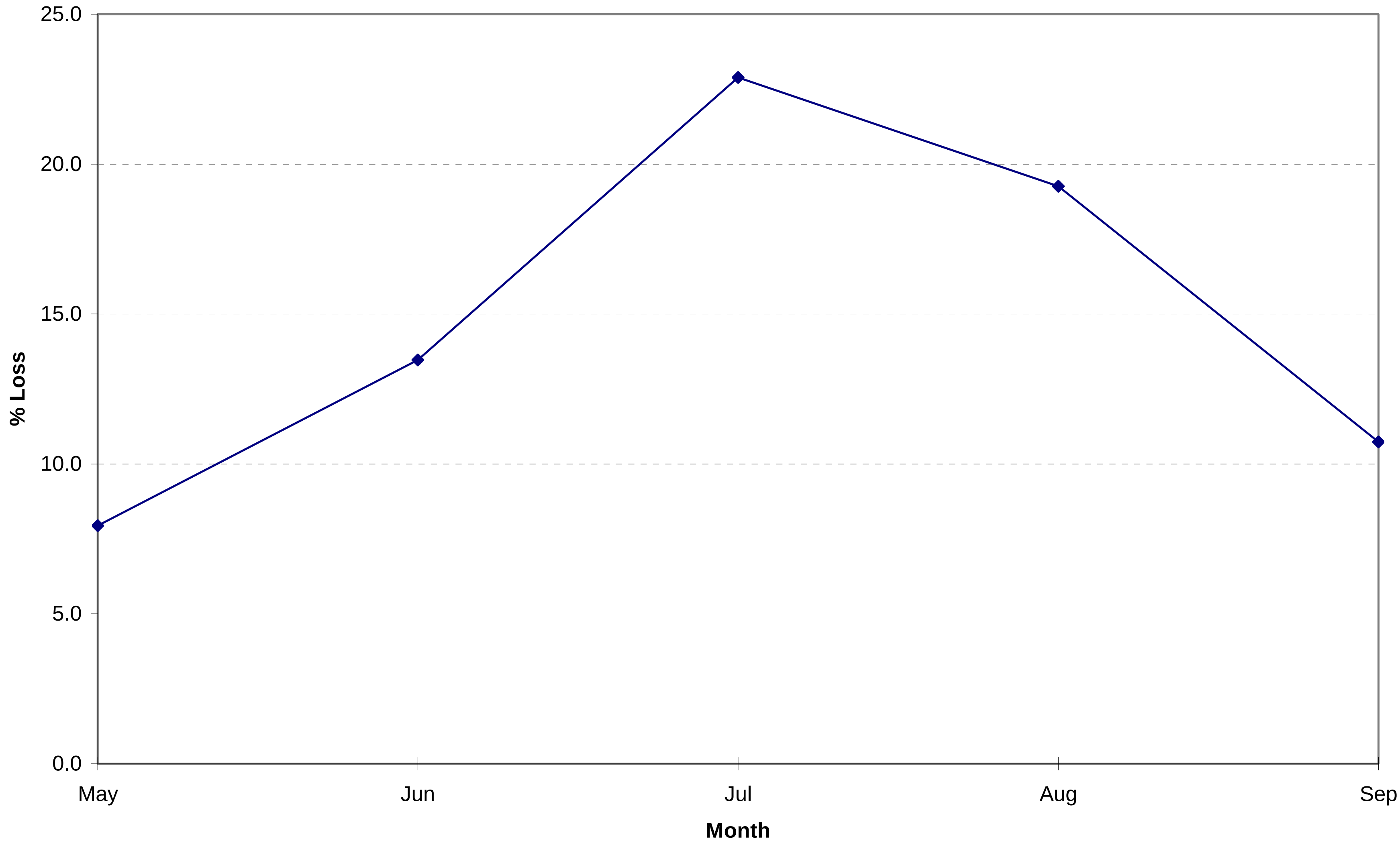
Table C-2.1 cont' DRY WEATHER PERIOD SENSITIVITY ANALYSIS					
Date	Measured	Simulated	Simulated	Measured	Consumptive
	WRP	WRP	WRP	Water	Loss
	Total Influent	I&I	San. Sewage	Supply	
	cfs	cfs	cfs	cfs	%
15-May-92	1289.5	58.6	1231.0	1653.3	25.5
16-May-92	1567.9	59.2	1508.7	1735.3	13.1
17-May-92	1551.9	48.9	1503.0	1691.9	11.2
18-May-92	1611.2	45.2	1565.9	1626.9	3.7
19-May-92	1506.8	41.5	1465.3	1735.2	15.6
20-May-92	1510.6	37.4	1473.2	1830.4	19.5
21-May-92	1567.1	33.5	1533.6	1907.5	19.6
22-May-92	1679.9	35.5	1644.4	1836.5	10.5
23-May-92	1747.0	56.2	1690.8	1546.6	-9.3
24-May-92	1561.6	28.3	1533.3	1450.8	-5.7
25-May-92	1529.4	26.6	1502.8	1483.9	-1.3
26-May-92	1673.9	25.3	1648.6	1500.6	-9.9
27-May-92	1554.9	23.7	1531.2	1638.6	6.6
28-May-92	1696.7	21.3	1675.4	1742.9	3.9
29-May-92	1684.0	18.6	1665.4	1759.5	5.3
30-May-92	1685.5	69.6	1615.8	1558.0	-3.7
31-May-92	1597.1	16.0	1581.1	1650.0	4.2
1-Jun-92	1703.2	14.1	1689.1	1778.4	5.0
2-Jun-92	1729.6	12.5	1717.1	1841.6	6.8
3-Jun-92	1546.3	10.2	1536.1	1908.3	19.5
4-Jun-92	1660.3	15.5	1644.7	1709.9	3.8
5-Jun-92	1534.3	9.7	1524.6	1690.4	9.8
6-Jun-92	1605.7	9.8	1595.9	1716.6	7.0
7-Jun-92	1594.3	6.0	1588.3	1707.5	7.0
8-Jun-92	1573.8	4.3	1569.5	1811.6	13.4
9-Jun-92	1607.2	3.2	1604.0	1857.4	13.6
10-Jun-92	1534.3	2.3	1532.1	1937.7	20.9
11-Jun-92	1696.1	1.5	1694.6	2005.0	15.5
12-Jun-92	1538.4	0.6	1537.8	2077.8	26.0
13-Jun-92	1529.7	0.2	1529.6	2115.1	27.7
14-Jun-92	1694.3	0.0	1694.2	2042.1	17.0
15-Jun-92	1523.4	0.0	1523.4	1964.1	22.4

Table C-2.1 cont' DRY WEATHER PERIOD SENSITIVITY ANALYSIS					
Date	Measured	Simulated	Simulated	Measured	Consumptive
	WRP	WRP	WRP	Water	Loss
	Total Influent	I&I	San. Sewage	Supply	
	cfs	cfs	cfs	cfs	%
19-Jun-92	1852.9	6.4	1846.5	1617.7	-14.1
20-Jun-92	1639.1	3.5	1635.6	1551.6	-5.4
21-Jun-92	1574.4	1.8	1572.6	1566.4	-0.4
22-Jun-92	1668.2	0.9	1667.3	1635.4	-2.0
23-Jun-92	1541.9	0.5	1541.4	1668.1	7.6
24-Jun-92	1643.4	0.2	1643.1	1733.9	5.2
25-Jun-92	1584.0	0.1	1583.8	1821.1	13.0
26-Jun-92	1460.9	0.1	1460.8	1864.4	21.6
27-Jun-92	1523.2	0.0	1523.2	1796.1	15.2
28-Jun-92	1369.5	0.0	1369.5	1817.5	24.6
29-Jun-92	1614.7	0.0	1614.7	1981.9	18.5
30-Jun-92	1212.2	0.0	1212.2	1978.2	38.7
1-Jul-92	1593.0	0.0	1593.0	2265.0	29.7
Average	**1559.8**	**1.0**	**1558.8**	**1792.1**	**13.0**
6-Jul-92	1604.8	5.3	1599.4	1884.0	15.1
7-Jul-92	1562.3	6.4	1555.9	1780.6	12.6
8-Jul-92	1568.1	8.7	1559.4	2000.6	22.1
9-Jul-92	1422.2	2.8	1419.4	2006.6	29.3
10-Jul-92	1541.2	5.1	1536.1	1982.7	22.5
Average	**1539.7**	**5.7**	**1534.0**	**1930.9**	**20.6**
16-Aug-92	1476.1	19.5	1456.6	1659.9	12.3
17-Aug-92	1509.5	16.2	1493.3	1773.9	15.8
18-Aug-92	1689.7	41.9	1647.8	1812.8	9.1
19-Aug-92	1617.6	14.7	1603.0	1848.0	13.3
20-Aug-92	1333.7	12.1	1321.6	1867.6	29.2
21-Aug-92	1620.7	10.1	1610.6	1929.1	16.5
22-Aug-92	1646.5	8.6	1637.9	1945.2	15.8
23-Aug-92	1466.5	7.2	1459.3	1986.1	26.5
24-Aug-92	1654.6	5.8	1648.7	2131.9	22.7
25-Aug-92	1778.2	8.1	1770.1	2203.6	19.7
Average	**1579.3**	**14.4**	**1564.9**	**1915.8**	**18.3**
31-Aug-92	1582.7	23.5	1559.2	1768.2	11.8
1-Sep-92	1608.4	18.0	1590.5	1783.6	10.8
2-Sep-92	1786.3	20.5	1765.8	1715.5	-2.9
3-Sep-92	1688.1	53.2	1634.9	1748.4	6.5
4-Sep-92	1485.6	12.5	1473.2	1782.7	17.4
5-Sep-92	1639.4	10.5	1628.9	1789.2	9.0
6-Sep-92	1491.2	9.0	1482.1	1707.8	13.2

Table C-2.2 DRY WEATHER MONTHLY SENSITIVITY ANALYSIS

Month	Measured WRP Total Influent cfs	Simulated WRP I&I cfs	Simulated WRP San. Sewage cfs	Measured Water Supply cfs	Consumptive Loss %
May	1571.8	49.8	1521.9	1653.2	7.9
Jun	1672.5	20.3	1652.2	1909.3	13.5
Jul	1641.9	6.5	1635.4	2121.0	22.9
Aug	1607.7	7.2	1600.5	1982.3	19.3
Sep	1508.9	11.6	1497.3	1677.3	10.7
Oct	1619.9	80.6	1539.3	1544.4	0.3
Nov	1570.1	68.5	1501.5	1521.3	1.3
Average	1616.4	21.6	1594.8	1879.7	15.2

Figure C-2.1 Dry Weather Period



Attachment C-3
Merged Period

Table C-3.1 CONTINUOUS PERIOD (Merged) ANALYSIS					
Month	Measured	Simulated	Simulated	Measured	Consumptive
	WRP	WRP	WRP	Water	Loss
	Total Influent	I&I	San. Sewage	Supply	
	cfs	cfs	cfs	cfs	%
Jan	1832.4	342.5	1490.0	1491.0	0.0
Feb	1762.6	363.9	1398.7	1481.2	5.6
Mar	2036.9	617.6	1419.3	1462.9	2.9
Apr	2027.3	500.3	1527.0	1459.6	-4.7
May	1571.8	49.8	1521.9	1653.2	7.9
Jun	1672.5	20.3	1652.2	1909.3	13.5
Jul	1641.9	6.5	1635.4	2121.0	22.9
Aug	1607.7	7.2	1600.5	1982.3	19.3
Sep	1508.9	11.6	1497.3	1677.3	10.7
Oct	1990.9	506.3	1484.6	1542.3	3.7
Nov	2082.9	639.7	1443.2	1482.9	2.7
Dec	1972.6	507.1	1465.5	1476.1	0.8
Average	1809.2	297.3	1511.9	1646.2	8.2

Figure C.3.1 Merged Period

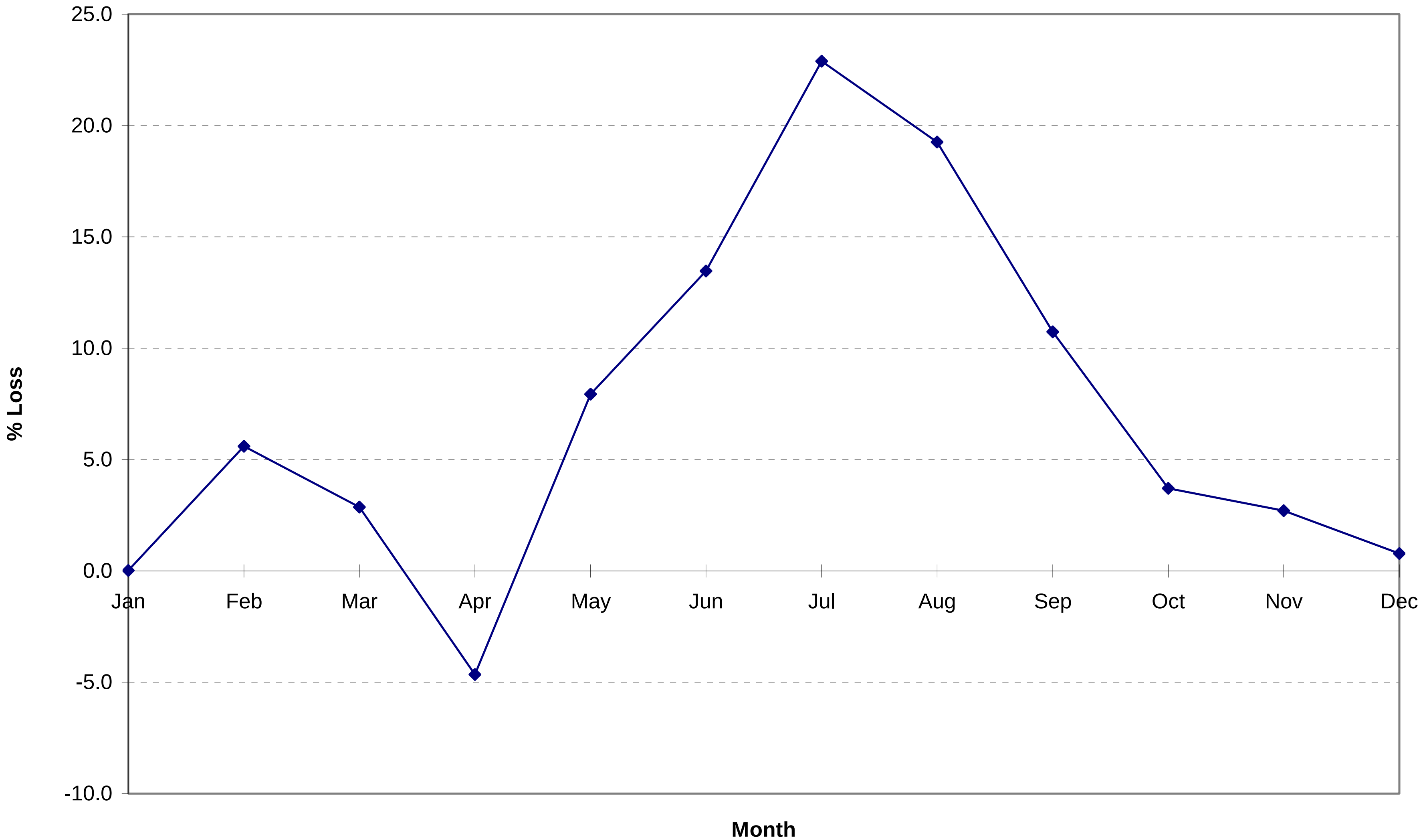


Table C-3.2 CONTINOUS PERIOD (Merged- Ratio) Analysis

Ratio of Continous to Dry Weather Consumptive Loss

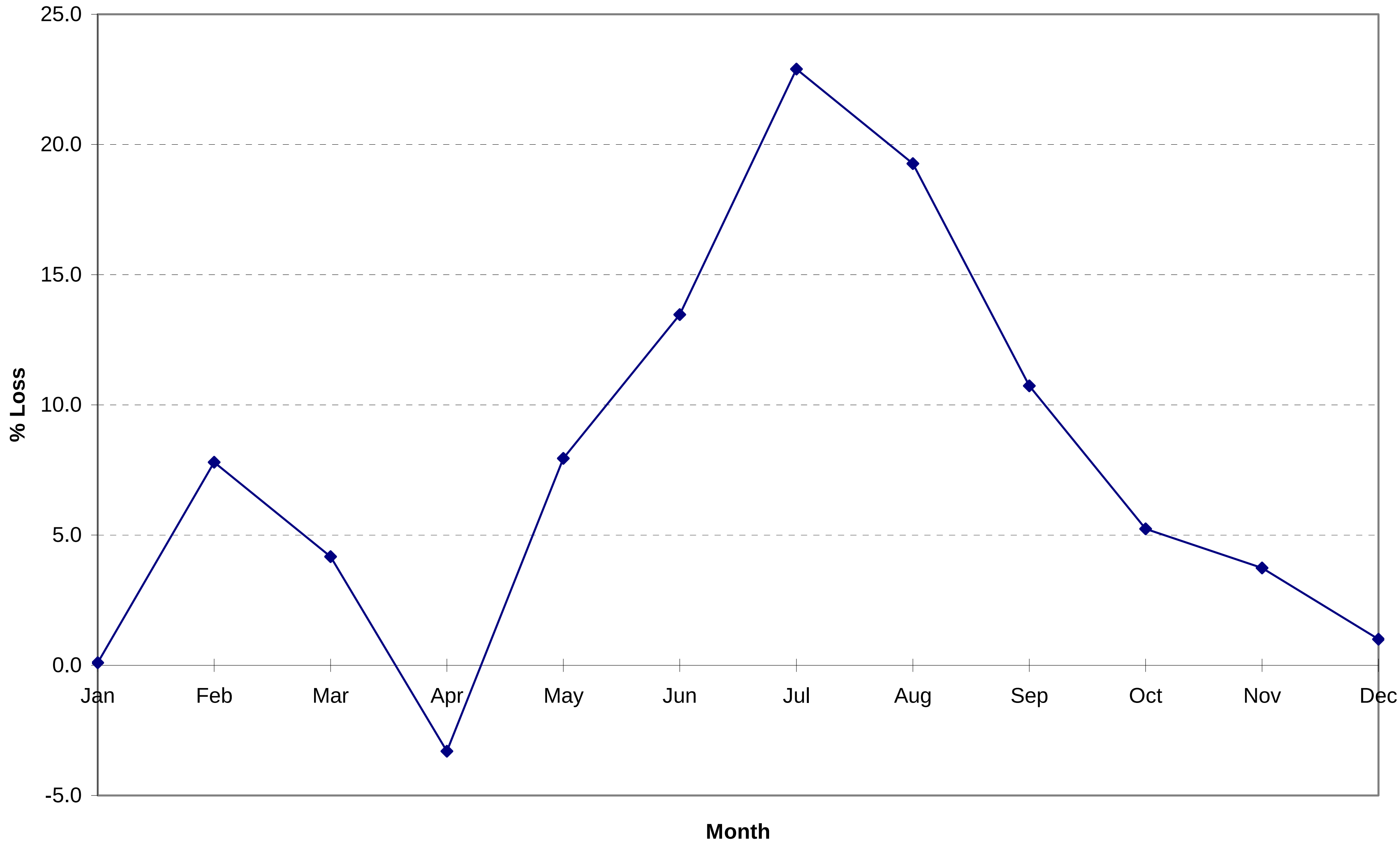
Month	Cont Sanitary	Cont Supply	Dry Sanitary	Dry Supply
May	1589.1	1615.8	1521.9	1653.2
Jun	1665.4	1879.9	1652.2	1909.3
Jul	1618.5	1978.3	1635.4	2121.0
Aug	1630.3	1902.4	1600.5	1982.3
Sep	1558.5	1679.3	1497.3	1677.3
Average	1612.4	1811.1	1581.5	1868.6
Cons Loss %	11.0		15.4	
Ratio	1.4			

Continuous Period Ratio Averages

Month	Influent	I&I	Adj Sanitary	Supply	Consumptive
Jan	1832.4	342.5	1489.5	1491.0	0.1
Feb	1762.6	363.9	1365.7	1481.2	7.8
Mar	2036.9	617.6	1401.8	1462.9	4.2
Apr	2027.3	500.3	1507.8	1459.6	-3.3
Oct	1990.9	506.3	1461.5	1542.3	5.2
Nov	2082.9	639.7	1427.4	1482.9	3.7
Dec	1972.6	507.1	1461.2	1476.1	1.0

Month	Measured WRP Total Influent cfs	Simulated WRP I&I cfs	Simulated WRP San. Sewage cfs	Measured Water Supply cfs	Consumptive Loss %
Jan	1832.4	342.5	1489.5	1491.0	0.1
Feb	1762.6	363.9	1365.7	1481.2	7.8
Mar	2036.9	617.6	1401.8	1462.9	4.2
Apr	2027.3	500.3	1507.8	1459.6	-3.3
May	1571.8	49.8	1521.9	1653.2	7.9
Jun	1672.5	20.3	1652.2	1909.3	13.5
Jul	1641.9	6.5	1635.4	2121.0	22.9
Aug	1607.7	7.2	1600.5	1982.3	19.3
Sep	1508.9	11.6	1497.3	1677.3	10.7
Oct	1990.9	506.3	1461.5	1542.3	5.2
Nov	2082.9	639.7	1427.4	1482.9	3.7
Dec	1972.6	507.1	1461.2	1476.1	1.0
Average	1809.2	297.3	1502.6	1646.2	8.7

Figure C-3.2 Merged-Ratio Period



Attachment C-4
Sensitivity Analysis

Table C-4.1 CONTINUOUS PERIOD (Positive) SENSITIVITY ANALYSIS

Month	Measured WRP Total Influent cfs	Simulated WRP I&I cfs	Simulated WRP San. Sewage Cfs	Measured Water Supply cfs	Consumptive Loss %
Jan	1672.2	409.4	1262.9	1481.1	14.7
Feb	1721.4	479.1	1242.4	1478.5	16.0
Mar	2064.4	890.4	1174.0	1470.1	20.1
Apr	2015.2	708.5	1306.7	1466.3	10.9
May	1663.9	190.4	1473.4	1641.3	10.2
Jun	1700.2	71.3	1628.8	1935.3	15.8
Jul	1718.1	112.6	1605.4	1995.2	19.5
Aug	1709.8	115.1	1594.7	1928.7	17.3
Sep	1711.0	228.9	1482.0	1686.2	12.1
Oct	2099.6	861.6	1238.0	1558.1	20.5
Nov	2197.0	997.9	1199.2	1480.8	19.0
Dec	1845.6	570.1	1275.4	1472.9	13.4
Average	1799.6	377.2	1422.4	1694.5	16.1

Figure C-4.1 Continuous Period (Positive)

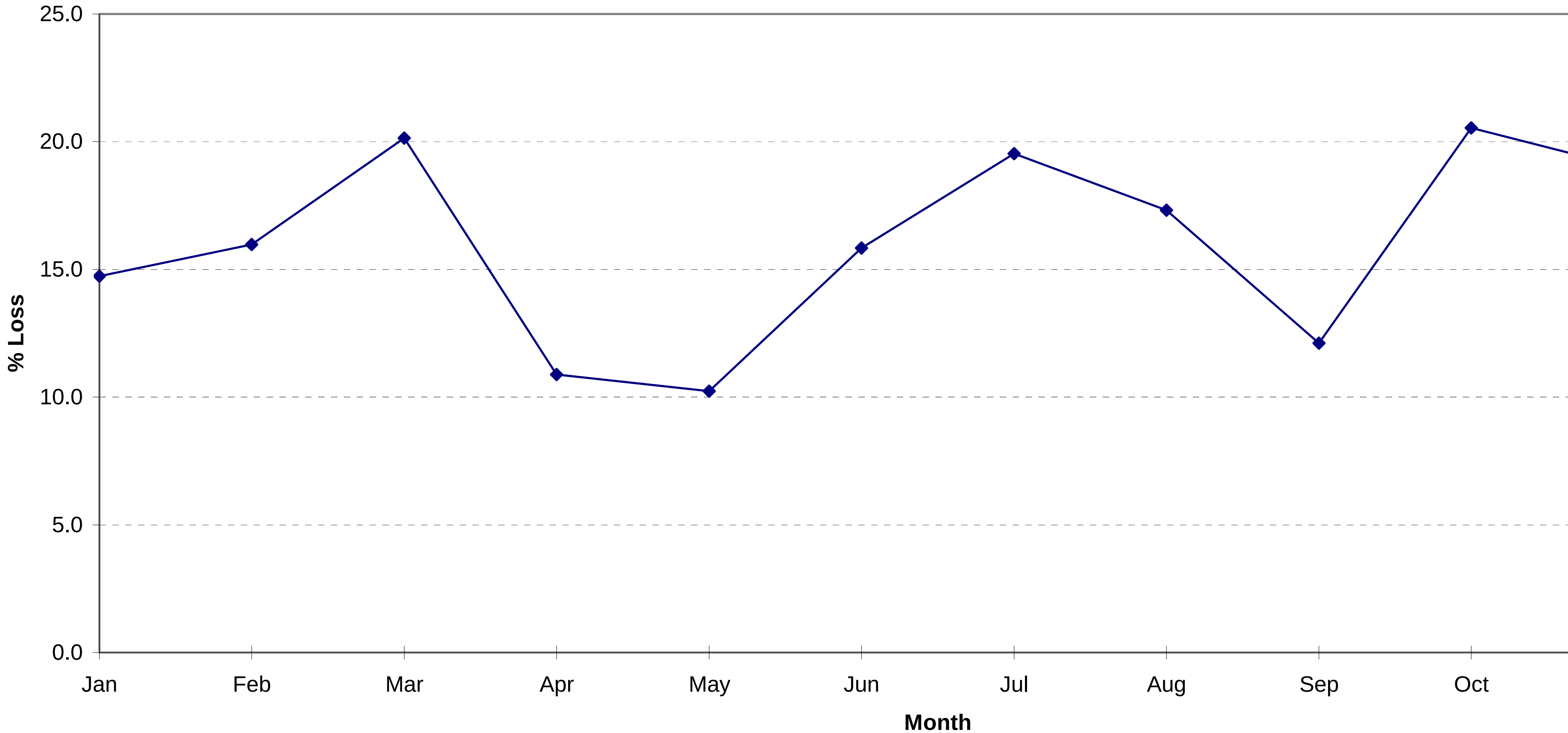


Table C-4.2 CONTINUOUS PERIOD (>0%) SENSITIVITY ANALYSIS

Month	Measured WRP Total Influent cfs	Simulated WRP I&I cfs	Simulated WRP San. Sewage cfs	Measured Water Supply cfs	Consumptive Loss %
Jan	1832.4	342.5	1364.3	1491.0	8.5
Feb	1762.6	363.9	1361.0	1481.2	8.1
Mar	2036.9	617.6	1338.7	1462.9	8.5
Apr	2027.3	500.3	1411.7	1459.6	3.3
May	1813.3	224.2	1523.7	1615.8	5.7
Jun	1738.8	73.3	1639.8	1879.9	12.8
Jul	1737.4	118.9	1613.7	1978.3	18.4
Aug	1745.5	115.2	1617.0	1902.4	15.0
Sep	1794.2	235.6	1519.4	1679.3	9.5
Oct	1990.9	506.3	1408.1	1542.3	8.7
Nov	2082.9	639.7	1365.5	1482.9	7.9
Dec	1972.6	507.1	1386.9	1476.1	6.0
Average	1878.3	353.6	1462.9	1621.9	9.8

Figure C-4.2 Continuous Period (>0%)

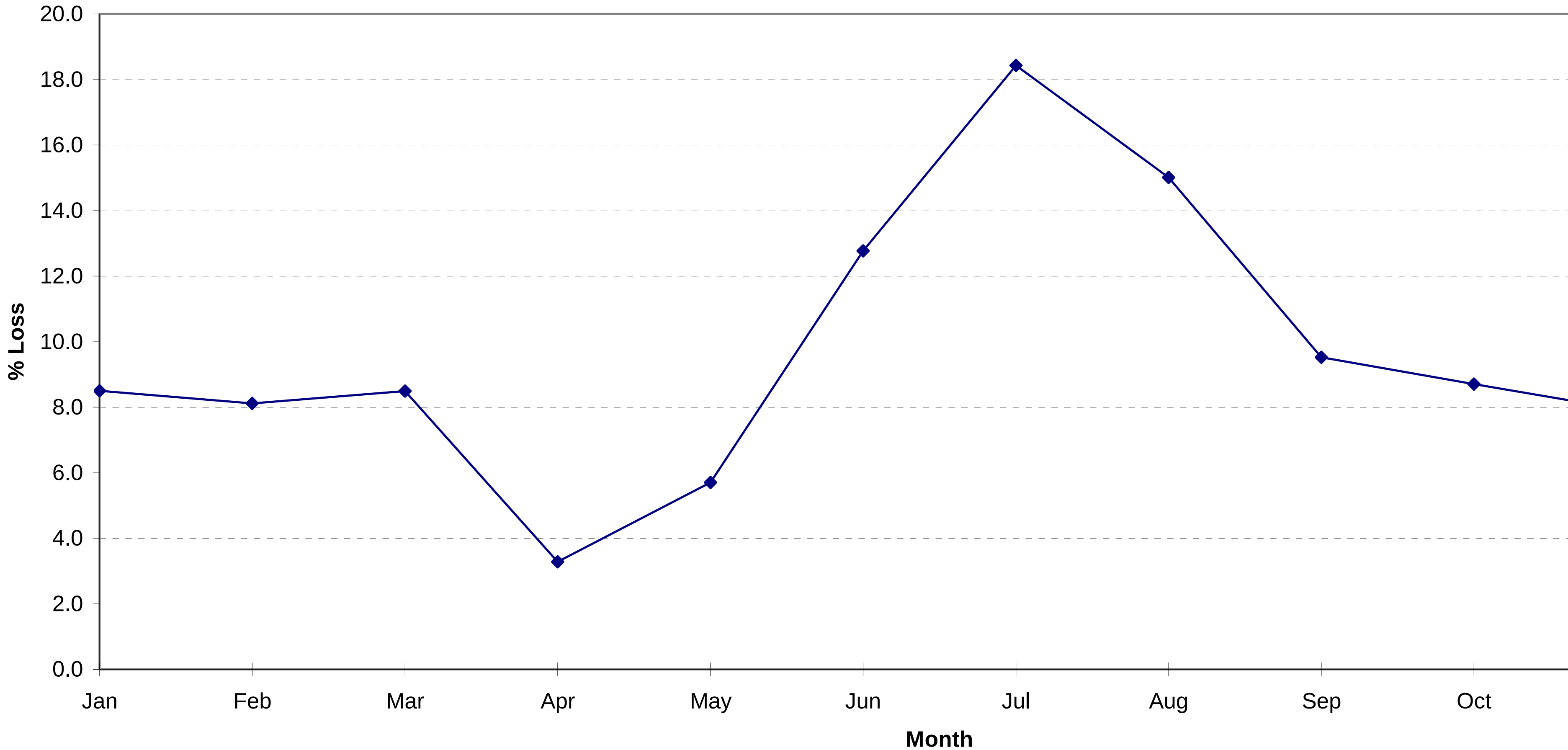


Table C-4.3 CONTINUOUS PERIOD (>3%) SENSITIVITY ANALYSIS

Month	Measured WRP Total Influent cfs	Simulated WRP I&I cfs	Simulated WRP San. Sewage Cfs	Measured Water Supply cfs	Consumptive Loss %
Jan	1832.4	342.5	1342.2	1491.0	10.0
Feb	1762.6	363.9	1336.7	1481.2	9.8
Mar	2036.9	617.6	1311.1	1462.9	10.4
Apr	2027.3	500.3	1378.0	1459.6	5.6
May	1813.3	224.2	1501.4	1615.8	7.1
Jun	1738.8	73.3	1628.9	1879.9	13.4
Jul	1737.4	118.9	1609.1	1978.3	18.7
Aug	1745.5	115.2	1609.3	1902.4	15.4
Sep	1794.2	235.6	1504.7	1679.3	10.4
Oct	1990.9	506.3	1380.1	1542.3	10.5
Nov	2082.9	639.7	1336.5	1482.9	9.9
Dec	1972.6	507.1	1359.8	1476.1	7.9
Average	1878.3	353.6	1442.0	1621.9	11.1

Figure C-4.3 Continuous Period (>3%)

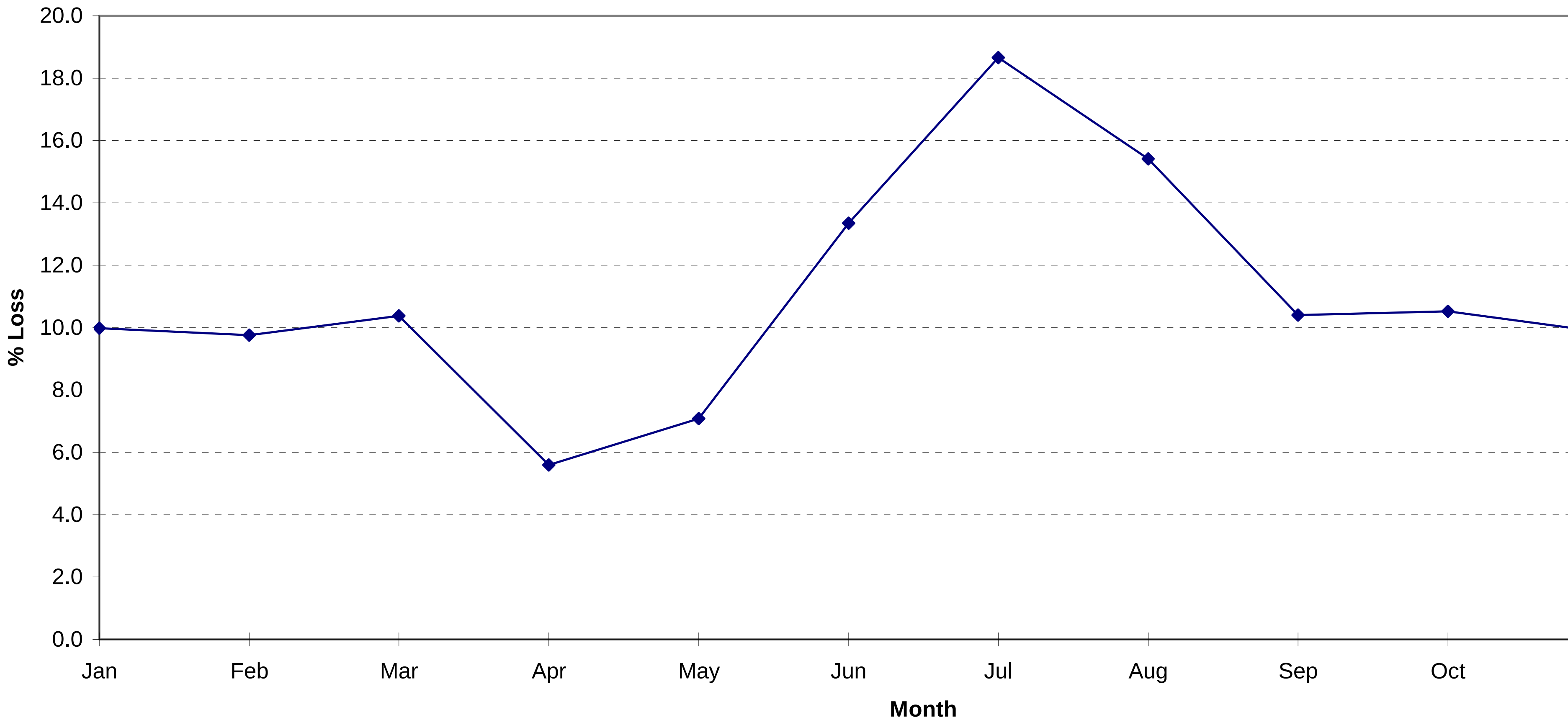
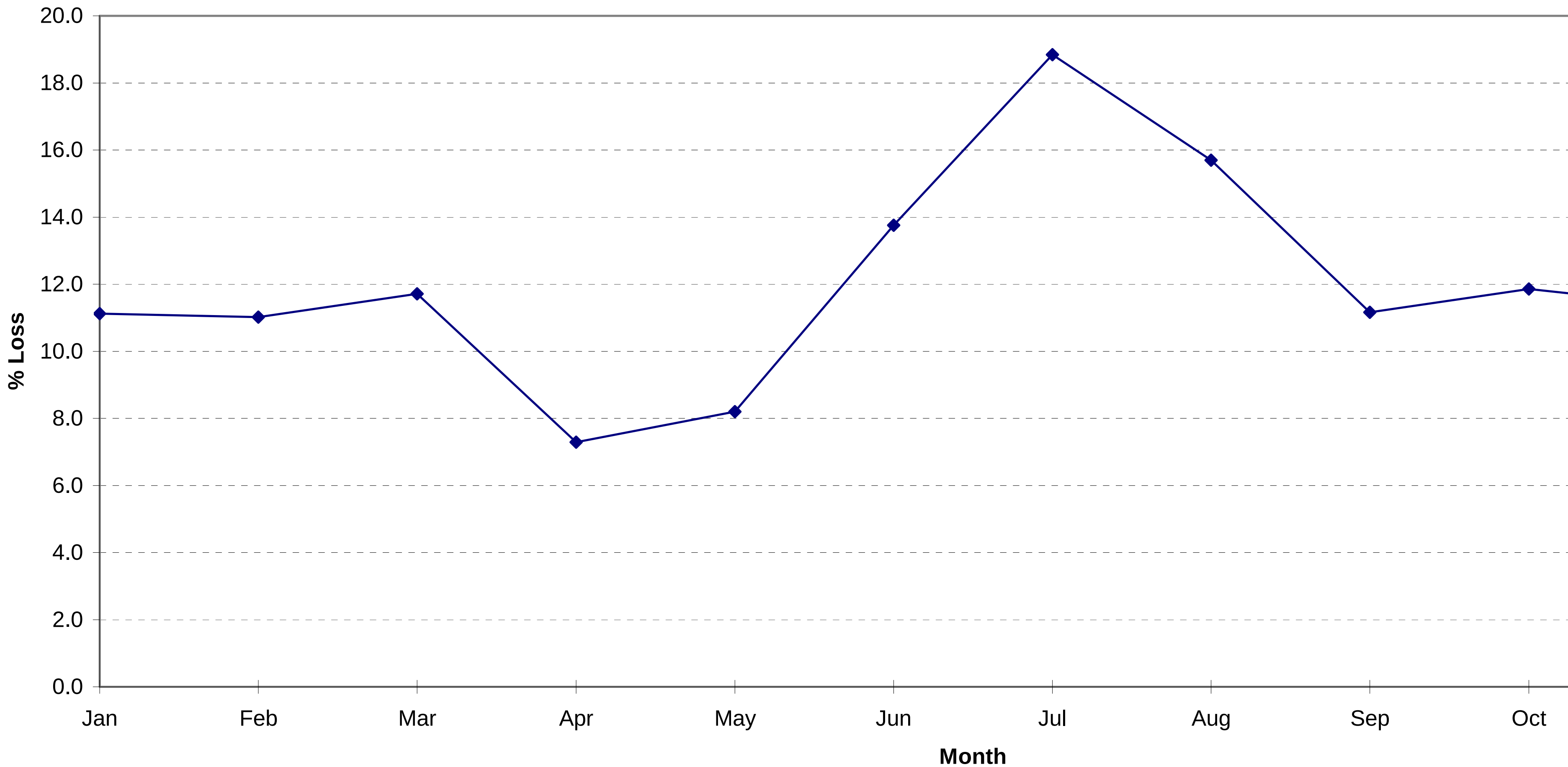


Table C-4.4 CONTINUOUS PERIOD (>5%) SENSITIVITY ANALYSIS

Month	Measured WRP Total Influent cfs	Simulated WRP I&I cfs	Simulated WRP San. Sewage cfs	Measured Water Supply cfs	Consumptive Loss %
Jan	1832.4	342.5	1325.2	1491.0	11.1
Feb	1762.6	363.9	1318.0	1481.2	11.0
Mar	2036.9	617.6	1291.5	1462.9	11.7
Apr	2027.3	500.3	1353.2	1459.6	7.3
May	1813.3	224.2	1483.3	1615.8	8.2
Jun	1738.8	73.3	1621.3	1879.9	13.8
Jul	1737.4	118.9	1605.5	1978.3	18.8
Aug	1745.5	115.2	1603.8	1902.4	15.7
Sep	1794.2	235.6	1491.9	1679.3	11.2
Oct	1990.9	506.3	1359.5	1542.3	11.9
Nov	2082.9	639.7	1314.7	1482.9	11.3
Dec	1972.6	507.1	1340.8	1476.1	9.2
Average	1878.3	353.6	1426.2	1621.9	12.1

Figure C-4.4 Continuous Period (>5%)



Appendix D

Monte Carlo Simulation
Comparison of Lockport vs. Lakefront Measurements

1. The Chicago District (with technical support from the United States Geological Survey - USGS) has undertaken an error analysis of the accounting flows. The intent of the analysis is to provide a comparison of the existing Lockport based Lake Michigan diversion accounting system with potential Lakefront based accounting systems. In performing this analysis three "Monte Carlo" simulations were completed. These simulations can be used to compare the existing accounting system (LOCKPORT) with a direct movement of the accounting system to the lakefront (LAKEFRONT), and with a possible future lakefront accounting system (FUTURE).

2. The Monte Carlo simulations used ten years of certified flows from WY86-95. All errors in the flows were assumed to be distributed either normally or log-normally, with the means equal to the reported values and standard deviations based on the expected accuracy of the specific component of the accounting system. The results are shown in the tables in attachment D-1. It should be noted that the results are "ball-park" in that at least some of the flows may be distributed differently, and also no comprehensive review of expected accuracies was performed. However, for discussion purposes the results do have value.

3. In reference to the LOCKPORT table, a number of considerations should be noted:

- The normal distribution was generally chosen for all flows, except those that are small (e.g. diversion above gage) or those that have large standard deviations (e.g. leakage). For these two cases the log-normal distribution was selected to prevent the possibility of flows below zero.

- In WY 92 the Chicago Flood required a large diversion to keep the canal level low and allow the freight tunnels to be plugged. However, to prevent unnatural biasing the effects of this diversion were removed from the analysis.

- Leakage values normally reported by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) have been replaced by estimates of leakage developed in attachment D-2. The standard deviation of these flows was computed from the square root of the sum of squares of the standard deviations of the USGS measurements.

- The Acoustic Velocity Meter (AVM) and lockage flows have the smallest errors (5% standard deviation), and the leakage flows have the largest errors (35% standard deviation).

- For comparison purposes the LOCKPORT simulation also gives the standard deviations of the component flows and the standard deviations of the imbalances.

5. The LAKEFRONT table is based on the following assumptions:

- The recommended fixed runoff value of 800 cfs with a standard deviation of 30% was used in the error analysis. A log-normal distribution was also selected for this component, because of the possibility that the large errors might generate negative flows.

- All direct diversion will be accounted for via the new lakefront AVM's to be installed this spring. The 15% standard deviation is based on an approximate average of high flow conditions (when the AVM's work well and the standard deviation is approximately 5%) and low flow conditions (when the standard deviation could be similar to the 35% computed for the Acoustic Doppler Current Profiler (ADCP) measurements of leakage).

- A consumptive use value is required for the computations of the imbalance in the sensitivity analysis (appendix B) and the error analysis (appendix D). The recommended fixed consumptive use value of 168 cfs with a standard deviation of 30% (similar to the variance in the watershed runoff) was used in the error analysis.

5. The FUTURE conditions simulation is based on the following assumptions:

- Runoff has been excluded from the sum of components. This exclusion is based on the assumption that a consensus has been reached to either explicitly (by picking a value) or to implicitly (by modifying the Decree target) select an average runoff value. This approach assumes that there is no uncertainty inherent in using a fixed value. However, as noted above, any selection of an average runoff value would have significant error because the variation in runoff is extremely high from year to year (see the Runoff Analysis, appendix A).

- Similar to the Lakefront system, all direct diversion will be accounted for via the new lakefront AVM's. Again, a standard deviation of 15% is expected.

- The leakage through the Chicago River lock and harbor walls has been reduced by one-half, i.e. 90 cfs. Added to the O'Brien (20 cfs) and Wilmette flows (5 cfs), gives a revised total leakage of 115 cfs.

- Using the same approach as for the watershed runoff, the explicit entry of the consumptive use credit has been excluded from sum of components. Again, this approach assumes that there is no uncertainty inherent in using a fixed value. However, as noted above, any selection of an average consumptive use value would have significant error becauseit is extremely difficult to separate effluent from strorm water (see the Consumptive Use Analysis, appendix C)

6. The simulations were completed to compare errors in the existing diversion accounting system with errors from a direct movement to the lakefront and with errors in a possible future accounting system. From Tables D-1.1 through D-1.3 it can be seen that the standard deviation for the existing system is 187 cfs, for the lakefront system is 310 cfs, and for the future system is 188 cfs. Two general conclusions can be reached from this analysis of errors:

- If the two systems that account for all of the flow past Lockport are considered (i.e. LOCKPORT and the LAKEFRONT), then present accounting system is clearly more accurate. This is primarily due to the fact that errors are smaller in computing runoff than in assuming a fixed value. Also, a consumptive use value (and the associated error) doesn’t have to be assumed.

- However, if a complete revision to the accounting can be negotiated, and a new total flow at the lakefront adopted (i.e. replacing the 3,200 cfs), then an accounting system equivalent to the FUTURE system would be in effect. In comparison to the LOCKPORT system, this new possible system would not significantly alter the level of accuracy.

Attachment D-1
Monte Carlo Simulations

Table D-1.1 Lake Michigan Diversion Accounting
Lockport - Monte Carlo Simulation

Description	WY 86	WY 87	WY 88	WY 89	WY 90	WY 91	WY 92	WY 93	WY 94	WY 95	Std Dev	Average
AVM Record	4113	4028	3537	3515	3749	3713	3452	4076	3094	3235	5N%	
Diversion Above Gage	0	2	1	1	1	1	2	2	1	2	10L%	
Total Canal Flow	4113	4030	3538	3516	3750	3714	3454	4078	3095	3237		
Groundwater	128	120	110	82	102	116	110	89	89	92	15N%	
Indiana Pumpage	82	82	31	28	28	29	30	42	40	35	10N%	
Des Plaines Runoff	180	146	106	135	192	200	177	340	153	168	10N%	
Federal Facilities	2	4	23	2	2	2	2	2	2	1	15N%	
Total Deductions	392	352	270	247	324	347	319	473	284	296		
By-Passed Flows	30	96	109	108	106	117	192	237	251	255	10N%	
Accountable Flows	3751	3774	3377	3377	3532	3484	3327	3842	3062	3196		3472
Standard Deviation	201	213	177	182	180	192	179	212	158	171		187
Illinois Pumpage	1724	1805	1906	1792	1755	1819	1785	1799	1887	1828	10N%	
Watershed Runoff	877	812	520	707	873	1041	848	1505	681	798	10N%	
Lockages	179	146	97	84	72	89	83	92	118	97	5N%	
Leakages	311	271	271	271	356	357	342	38	37	35	35L%	
Navigation Makeup	142	157	73	52	46	37	43	59	34	28	15N%	
Discretionary Flow	302	314	352	264	305	315	293	331	308	320	15N%	
Direct Diversions	934	888	793	671	779	798	761	520	497	480		
Component Flows	3535	3505	3219	3170	3406	3658	3394	3824	3065	3106		3388
Standard Deviation	230	217	228	210	237	237	230	246	207	213		226
Imbalance	216	269	158	207	125	-175	-67	18	-3	90		84
Standard Deviation	306	295	287	278	297	309	300	320	251	275		292

%N: Normal - Standard Deviation (%)　　%L: Lognormal - Standard Deviation (%)

Table D-1.2 Lake Michigan Diversion Accounting
Lakefront - Monte Carlo Simulation

Description	WY 86	WY 87	WY 88	WY 89	WY 90	WY 91	WY 92	WY 93	WY 94	WY 95	Std Dev	Average
Illinois Pumpage	1724	1805	1906	1792	1755	1819	1785	1799	1887	1828	10N%	
Watershed Runoff	800	800	800	800	800	800	800	800	800	800	30L%	
Lockages	179	146	97	84	72	89	83	92	118	97	15N%	
Leakages	311	271	271	271	356	357	342	38	37	35	15N%	
Navigation Makeup	142	157	73	52	46	37	43	59	34	28	15N%	
Discretionary Flow	302	314	352	264	305	315	293	331	308	320	15N%	
Direct Diversions	934	888	793	671	779	798	761	520	497	480		
Consumptive Use	168	168	168	168	168	168	168	168	168	168	30L%	
Component Flows	3290	3325	3331	3180	3167	3234	2874	2950	3016	2940		3131
Standard Deviation	311	310	321	297	296	327	307	305	304	323		310

%N: Normal - Standard Deviation (%) %L: Lognormal - Standard Deviation (%)

Table D-1.3 Lake Michigan Diversion Accounting
Future - Monte Carlo Simulation

Description	WY 86	WY 87	WY 88	WY 89	WY 90	WY 91	WY 92	WY 93	WY 94	WY 95	Std Dev	Average
Illinois Pumpage	1724	1805	1906	1792	1755	1819	1785	1799	1887	1828	10N%	
Lockages	179	146	97	84	72	89	83	92	118	97	15N%	
Leakages	75	75	75	75	75	75	75	75	75	75	15N%	
Navigation Makeup	142	157	73	52	46	37	43	59	34	28	15N%	
Discretionary Flow	302	314	352	264	305	315	293	331	308	320	15N%	
Direct Diversions	698	692	597	475	498	516	494	557	535	520		
Component Flows	2422	2497	2503	2267	2253	2335	2279	2356	2422	2348		2368
Standard Deviation	180	182	195	188	183	197	188	193	191	185		188

%N: Normal - Standard Deviation (%)

Attachment D-2
Leakage Estimates

Leakage Estimates

1. As a requirement to compute the imbalances required for the runoff sensitivity analyses (appendix B) as well as the error analysis in this appendix, an estimate of the total leakage over the period of WY86-95 is needed. Table D-2.1 and Figure D-2.1 provide the calculated estimates. The leakage estimates are based on USGS's 1993 leakage measurements at the Chicago River and Controlling Works (see Tables D-2.2, D-2.3 and D-2.4), and on the following assumptions:

- Wall leakage in WY84-86 is the sum of the current leakage through the walls plus the excessive leakage repaired by the State of Illinois. The current leakage is the difference between the USGS leakage measurements on the Chicago River at Lakeshore Drive (see Table D-2.2, 192 cfs) and through the West Gate (see Table D-2.3, 133 cfs), or 59 cfs. Beginning in mid -1986, the levels in Lake Michigan exceeded record levels. To mitigate the flows over and through the inner harbor wall the State of Illinois performed a number of repairs. As part of these repairs, in November 1986, the flow through a number of voids in the wall were reduced (the Illinois Department of Natural Resources - IDNR estimated 30 cfs).

- Wall leakage during the period of WY87-95 was decreased to reflect the State's November 1986 repairs.

- The IDNR also noted that overtopping of the inner harbor wall occurred in 1986. The USACE approximates the leakage for this category at 10 cfs.

- A review of the impacts of the USACE's repairs to the west gates is also provided in the second and third table. A simplified hydraulic analysis was undertaken to evaluate the possible effects of the repairs. In the analysis the effects of changes in Lake and River levels were minimized through the computation of flow through proxy orifices, (reflecting flow through the gates). The analysis showed no changes in openings before and after repairs, and this suggests that any impacts were not significant. However, it should be noted that the standard deviations in the USGS measurements is quite high, and just replacing the gate seals should have had an effect. Therefore the USACE approximates the leakage for reduction through the west gates, after the third quarter of WY93, at 10%.

- Based on the USACE's 93 repairs, the excessive east gate leakage was eliminated. This saved approximately 85 cfs, after the third quarter of WY93 (see the following paragraphs for a detailed explanation of the computations).

- The pump leakage at the Wilmette Pumping Station was reduced, based on MWRDGC's 93 repairs. The leakage through the pumps was reduced from 59 cfs to less than 15 cfs (USGS's estimate). To fix this leakage value the average of the USGS measurement (5 cfs) will be used.

2. Table D-2.4 provides USGS's leakage measurement through the Lake (or east) gate, and as can be seen from the table the leakages into the lock are excessive. The leakage calculations are based on leakage occurring while the river (or west) gates are open. The rate of leakage is the leakage through the east gate (see Table D-2.4, 919 cfs) less the leakage through the west gate (133 cfs) or 786 cfs. The procedure used to compute the east gate leakage is similar to the method employed by the IDNR, and is as follows:

- Obtain the number of lock empties from the diversion accounting data.

- Obtain lock operation data from the USACE's lock performance monitoring system.

- Compute the leakage for each period of the lock operation data.

- Regress the number of lock empties versus the leakage.

- Determine the leakage for the period in which the east gates did not properly close, October 1989 through May 1993.

3. The number of lock empties is provided for the time period October 1989 through April 1996 in Table D-2.5, and plotted on Figure D-2.2. The values for April 1990 and April 1991 were unavailable, and those shown are averages of the values for April 1992-1996.

4. The lock operation information is from the lock performance monitoring system data files. The file contains the direction, date and times of each major vessel and lockage (for each lockage, at least one description of date, times, and vessel information is available). If only one large vessel or only recreational vessels were locked through, one set of times were recorded. For multiple major vessels, each vessel was described. The description includes vessel name (or 'recreation'), date of lockage, direction (up or down) of lockage, and times for start of lockage, entry to lock finished, start of lock exit, and end of lockage. The start of lockage and start of lock exit were considered to be when the lock gates began to open. The end of entry and end of lockage were used as times the gates began to close. A second type of data was the unavailable period. Two types exist in the period examined. The first type is when the west gates are opened to flush accumulated ice from the lock chamber. The duration of the opening is included in the calculations. The second type is a shutdown of the lock for repairs. These periods, in May 1995, were when the lock was closed and were not included in the calculations.

5. Due to differences in data file formats for differing periods, only June-July 1992, August-October 1993, and December 1993-December 1995 were readily available and used to compute the leakage through the lock. The actual calculations of the leakage were performed using a FORTRAN program, and based on the following method:

- For an upbound lockage (river to lake), the west gates were assumed open from 2 minutes before the start of lockage until 2 minutes after the end of entry. The total open time usually lasts around 6 minutes. If the next vessel/lockage is also upbound and starts before the end of the current lockage, this vessel/lockage is included with the current one.

- For downbound lockages (lake to river), the start of exit to end of lockage plus 4 minutes define the gate open time. Again, direction and times for the current lockage are compared with the next to determine concurrent lockages.

- The rate of excessive leakage is the leakage through the east gate (919 cfs) less the leakage through the west gate (133 cfs) or 786 cfs.

- During the two minutes the west gates open or close, the excess leakage was assumed to change linearly.

- At the end of each day, the accumulated open time, at an excessive leakage rate of 786 cfs, is averaged across the day for daily leakage. At the end of the month, the monthly accumulated open time at 786 cfs is averaged across the month. The annual average is based on the total open time for the year. The daily leakages ranged from 2 to 313 cfs. The monthly leakage ranged from 7 to 215 cfs. The total volume averaged across 1994 and 1995 is 96 cfs. The results are shown in the leakage column in Table D-2.6.

6. A regression analysis was performed to compute the leakage for the period in which the locks did not properly close, October 1989 through May 1993. This analysis was based on the regression of the number of lock empties versus the leakage. The results of the regression analysis are provided in Table D-2.6. The leakage computed using the regression equation for October 1989 through May 1993 is given in Table D-2.7 and shown on Figure D-2.3.

Table D-2.1 Estimated Total Leakage

W Year	Walls	Overtop	W - Gate	E - Gate	O'Brien	Wilmette	Total
1984	89		133		20	59	301
1985	89		133		20	59	301
1986	89	10	133		20	59	311
1987	59		133		20	59	271
1988	59		133		20	59	271
1989	59		133		20	59	271
1990	59		133	85	20	59	356
1991	59		133	86	20	59	357
1992	59		133	84	20	59	342
1993	59		130	42	20	46	255
1994	59		120		20	5	204
1995	59		120		20	5	204

Notes:
- Wall leakage 84-86 is difference between Lakeshore and West Gate (59 cfs) plus excess leakage through walls (30 cfs)
- Wall leakage 87-95 has been decreased to reflect State's 87 repairs
- Overtopping of inner harbor wall occurred in 86
- West gate leakage was reduced by 10%, based on COE's 93 repairs
- Excessive east gate leakage was eliminated, based on COE's 93 repairs
- Willmette pump leakage was reduced, based on MWRD's 93 repairs

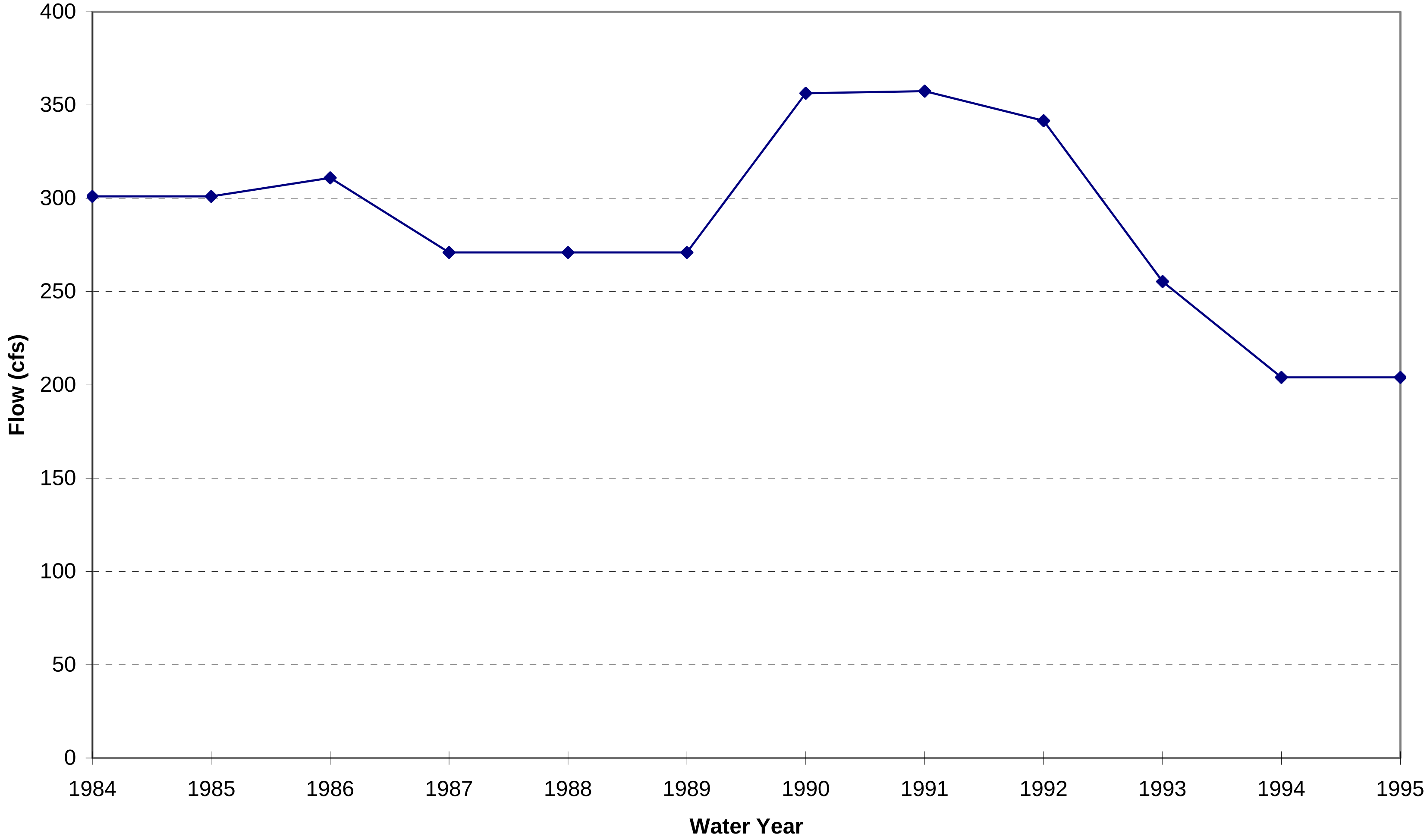


Figure D-2.1 Total Leakage

Table D-2.2 Chicago River Measurements						
Date	Lake Elev	River Elev	Transects	Flow	Head	Area
	(ft ccd)	(ft ccd)	(#)	(cfs)	(ft)	(ft^2)
4/5/93	0.56	-1.53				
4/6/93	0.36	-1.46				
4/7/93	0.39	-1.57	5	172	1.96	26
5/3/95	1.06	-1.72				
5/4/93	1.10	-1.84				
5/5/93	1.02	-1.60				
5/6/93	1.06	-1.72				
5/10/93	1.07	-1.48	4	167	2.55	22
7/12/93	1.54	-1.40				
7/13/93	1.60	-1.46				
7/14/93	1.80	-1.77	5	251	3.57	28
7/15/93	1.74	-1.44	4	182	3.18	21
7/16/93	1.67	-1.52	2	145	3.19	17
9/20/93	1.35	-1.77	6	227	3.12	27
9/21/93	1.52	-1.49	7	208	3.01	25
10/4/93	1.18	-1.54				
10/5/93	0.91	-1.29	2	153	2.20	21
10/6/93	0.59	-1.51	4	145	2.10	21
Average:				192		
Notes: Lake and river elevations for 5/3 and 5/6 are averages of 5/4 and 5/5						
Lake and rvier elevations for 7/16 are averages of 7/12 through 7/15						
Q=CA(2gH)^.5, or A=Q/(C(2gH)^.5), Assume C=.6						
where Q=Flow, H=Head, A=Area						

Table D-2.3 River Gate Measurements						
Date	Lake Elev	River Elev	Transects	Flow	Head	Area
	(ft ccd)	(ft ccd)	(#)	(cfs)	(ft)	(ft^2)
4/5/93	0.56	-1.53	14	132	2.09	19
4/6/93	0.36	-1.46	4	143	1.82	22
4/7/93	0.39	-1.57				
5/3/95	1.06	-1.72	2	237	2.78	30
5/4/93	1.10	-1.84	22	131	2.94	16
5/5/93	1.02	-1.60				
5/6/93	1.06	-1.72	18	123	2.78	15
5/10/93	1.07	-1.48				
7/12/93	1.54	-1.40				
7/13/93	1.60	-1.46	10	102	3.06	12
7/14/93	1.80	-1.77	14	143	3.57	16
7/15/93	1.74	-1.44	11	148	3.18	17
7/16/93	1.67	-1.52				
9/20/93	1.35	-1.77				
9/21/93	1.52	-1.49				
10/4/93	1.18	-1.54				
10/5/93	0.91	-1.29				
10/6/93	0.59	-1.51				
Average:				133		
Notes: Lake and river elevations for 5/3 and 5/6 are averages of 5/4 and 5/5						
Lake and rvier elevations for 7/16 are averages of 7/12 through 7/15						
Q=CA(2gH)^.5, or A=Q/(C(2gH)^.5), Assume C=.6						
where Q=Flow, H=Head, A=Area						

Table D-2.4 Lake Gate Measurements						
Date	Lake Elev	River Elev	Transects	Flow	Head	Area
	(ft ccd)	(ft ccd)	(#)	(cfs)	(ft)	(ft^2)
4/5/93	0.56	-1.53	7	912	2.09	131
4/6/93	0.36	-1.46	10	835	1.82	129
4/7/93	0.39	-1.57				
5/3/95	1.06	-1.72				
5/4/93	1.10	-1.84				
5/5/93	1.02	-1.60				
5/6/93	1.06	-1.72				
5/10/93	1.07	-1.48	27	961	2.55	125
7/12/93	1.54	-1.40	4	862	2.94	104
7/13/93	1.60	-1.46				
7/14/93	1.80	-1.77				
7/15/93	1.74	-1.44				
7/16/93	1.67	-1.52				
9/20/93	1.35	-1.77				
9/21/93	1.52	-1.49				
10/4/93	1.18	-1.54				
10/5/93	0.91	-1.29				
10/6/93	0.59	-1.51				
Average:				919		
Notes: Lake and river elevations for 5/3 and 5/6 are averages of 5/4 and 5/5						
Lake and rvier elevations for 7/16 are averages of 7/12 through 7/15						
Q=CA(2gH)^.5, or A=Q/(C(2gH)^.5), Assume C=.6						
where Q=Flow, H=Head, A=Area						

Table D-2.5 Lock Empties							
Month	Empties		Month	Empties		Avg April:	510.4
Oct-89	1006		Jun-93	907			
Nov-89	281		Jul-93	1334			
Dec-89	83		Aug-93	1346			
Jan-90	111		Sep-93	876			
Feb-90	62		Oct-93	827			
Mar-90	84		Nov-93	295			
Apr-90	510		Dec-93	137			
May-90	1268		Jan-94	91			
Jun-90	1486		Feb-94	89			
Jul-90	1514		Mar-94	126			
Aug-90	1520		Apr-94	694			
Sep-90	1338		May-94	1521			
Oct-90	869		Jun-94	1598			
Nov-90	321		Jul-94	1810			
Dec-90	93		Aug-94	1739			
Jan-91	88		Sep-94	1501			
Feb-91	70		Oct-94	1095			
Mar-91	126		Nov-94	300			
Apr-91	510		Dec-94	161			
May-91	1360		Jan-95	101			
Jun-91	1493		Feb-95	120			
Jul-91	1589		Mar-95	182			
Aug-91	1589		Apr-95	624			
Sep-91	1275		May-95	1376			
Oct-91	984		Jun-95	1613			
Nov-91	292		Jul-95	1644			
Dec-91	90		Aug-95	1447			
Jan-92	105		Sep-95	1293			
Feb-92	92		Oct-95	996			
Mar-92	98		Nov-95	276			
Apr-92	404		Dec-95	99			
May-92	1333		Jan-96	59			
Jun-92	1422		Feb-96	56			
Jul-92	1520		Mar-96	23			
Aug-92	1518		Apr-96	384			
Sep-92	1295						
Oct-92	1059						
Nov-92	252		W Year	Empties			
Dec-92	148		90	9263			
Jan-93	138		91	9383			
Feb-93	148		92	9153			
Mar-93	184		93	3309			
Apr-93	434		94	10428			
May-93	946		95	9956			

Figure D-2.2 Lock Empties versus Time

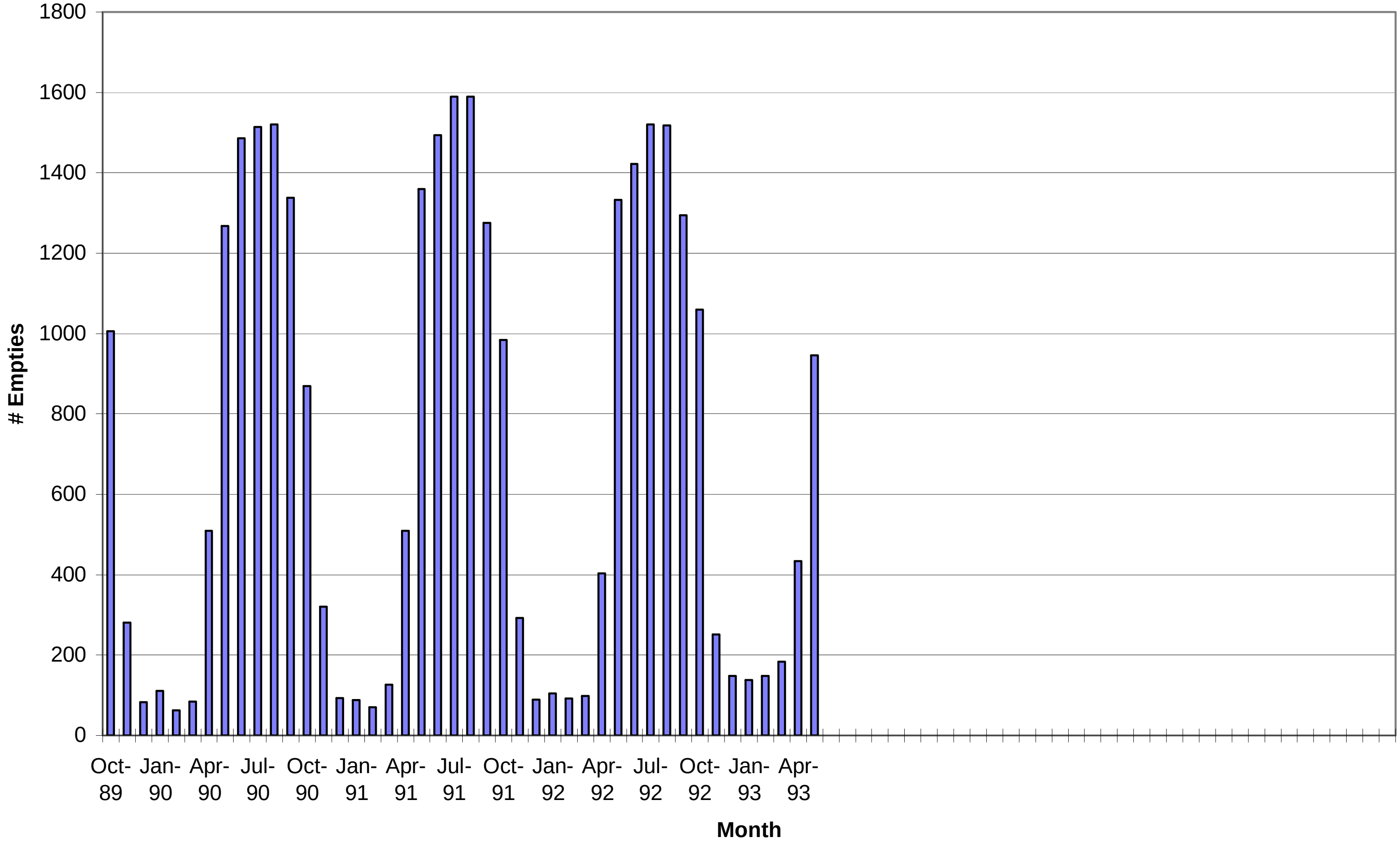


Table D-2.6 East Gate Regression Analysis			
Month	Leakage	Empties	Est Leak
Jun-92	160	1422	164
Jul-92	175	1520	176
Aug-93	168	1346	155
Sep-93	102	876	98
Oct-93	77	827	92
Dec-93	10	137	8
Jan-94	7	91	2
Feb-94	8	89	2
Mar-94	9	126	7
Apr-94	64	694	76
May-94	159	1521	176
Jun-94	189	1598	186
Jul-94	206	1810	212
Aug-94	196	1739	203
Sep-94	173	1501	174
Oct-94	104	1095	125
Nov-94	24	300	28
Dec-94	13	161	11
Jan-95	7	101	4
Feb-95	15	120	6
Mar-95	14	182	13
Apr-95	56	624	67
May-95	145	1376	159
Jun-95	194	1613	188
Jul-95	215	1644	191
Aug-95	203	1447	167
Sep-95	162	1293	149
Oct-95	96	996	113
Nov-95	20	276	25
Dec-95	9	99	3
Average	99		99
Intercept	-8.75		
Slope	0.122		
R Square	0.98		
Std Err	12.32		

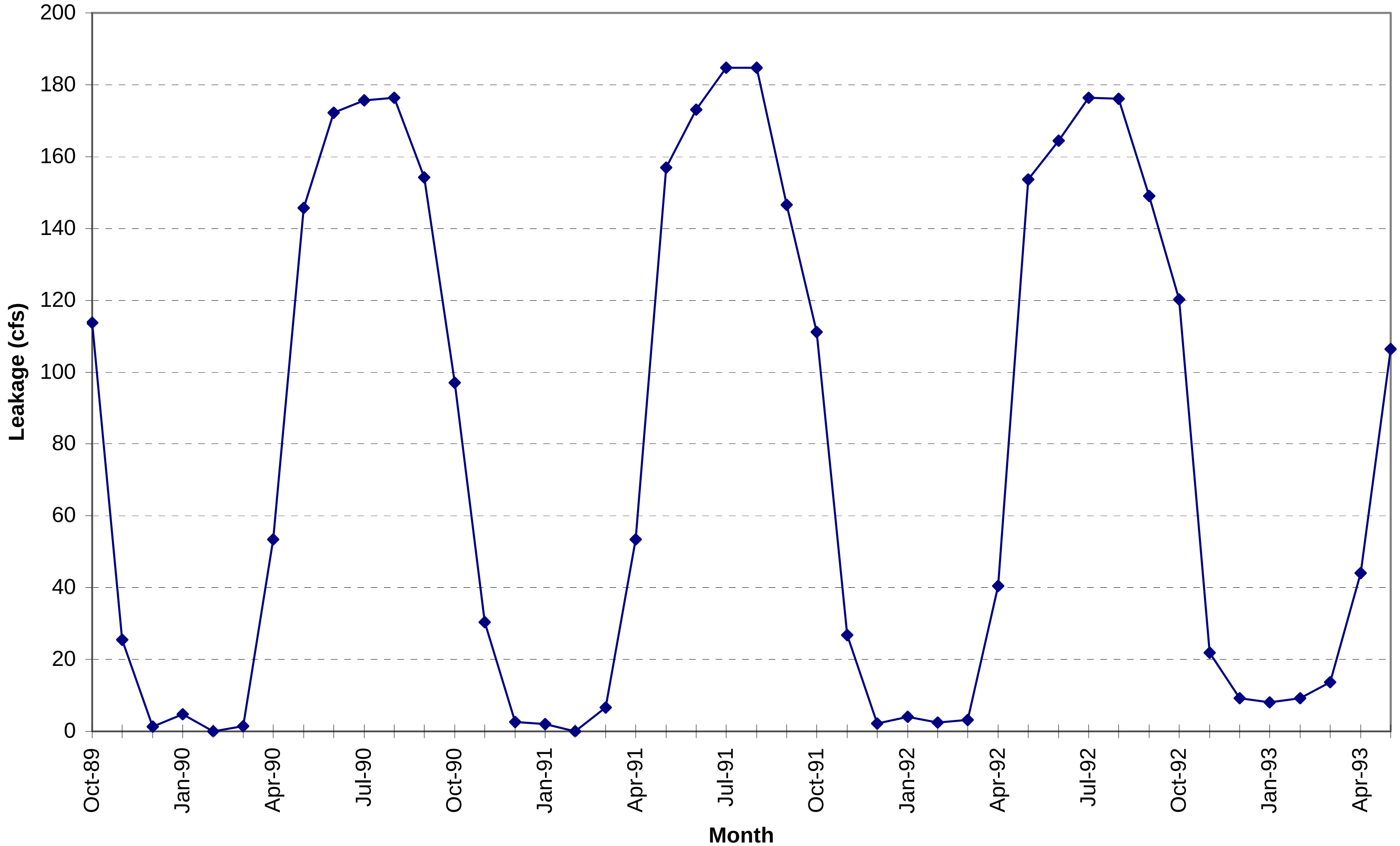
Table D-2.7 Excessive East Gate Leakage

Month	Empties	Leakage	
Oct-89	1006	114	
Nov-89	281	25	
Dec-89	83	1	
Jan-90	111	5	
Feb-90	62	0	
Mar-90	84	1	
Apr-90	510	53	
May-90	1268	146	
Jun-90	1486	172	
Jul-90	1514	176	
Aug-90	1520	176	
Sep-90	1338	154	
Oct-90	869	97	
Nov-90	321	30	
Dec-90	93	3	
Jan-91	88	2	
Feb-91	70	0	
Mar-91	126	7	
Apr-91	510	53	
May-91	1360	157	
Jun-91	1493	173	
Jul-91	1589	185	
Aug-91	1589	185	
Sep-91	1275	147	
Oct-91	984	111	
Nov-91	292	27	
Dec-91	90	2	
Jan-92	105	4	
Feb-92	92	2	
Mar-92	98	3	
Apr-92	404	40	
May-92	1333	154	
Jun-92	1422	164	
Jul-92	1520	176	
Aug-92	1518	176	
Sep-92	1295	149	
Oct-92	1059	120	
Nov-92	252	22	
Dec-92	148	9	
Jan-93	138	8	
Feb-93	148	9	
Mar-93	184	14	
Apr-93	434	44	
May-93	946	106	

Prior to Repairs

W Year	Empties	Leakage	
90	9263	85	
91	9383	86	
92	9153	84	
93	3309	42	

Fogire D-2.3 Excess East Gate Leakage



Appendix E

Responses to Comments on Draft Report

1. The draft report of this document, dated 31 January 1996, was submitted to the Corps of Engineers' Hydrologic Engineering Center (HEC), and the parties of the Great Lakes mediation process for review. Comments were received from HEC, the State of Illinois and the State of New York. This appendix provides copies of the comments and the District's responses.

2. Included in the attachments to this appendix are the written comments and the District's responses:

- Attachment E-1 - Comments from the Hydrologic Engineering Center
- Attachment E-2 - Comments from the State of Illinois
- Attachment E-3 - Comments from the State of New York

Attachment E-1
Comments from the
Hydrologic Engineering Center

Comments from the Hydrologic Engineering Center

<u>HEC #1</u>

1. The subject report was reviewed for the quality of its analysis to estimate an average annual runoff for the Chicago River Watershed. Opinion is withheld regarding the results produced by the analysis as the Districts efforts are ongoing to refine and substantiate the results. Additional such efforts will likely result from the comments that follow. The comments below describe weaknesses in the analysis along with suggested further analysis to mitigate, where possible. Comments are also offered on adjustments to the architecture of the report to make certain points clearer to the reader. References are made to paragraph numbers in the subject report.

Response #1

Noted.

<u>HEC #2</u>

2. The order of the comments below carries no significance. A table at the end of this review gives a qualitative sense of the importance of each item.

Response #2

Noted.

<u>HEC #3</u>

3. The rain gage network used for the period of record analysis is somewhat sparse with only three gages. This would normally be a big point for scrutiny and therefore needs to be addressed. In particular, reasons why this is acceptable should be presented. These might include: for long periods of record analysis, the random nature of rainfall should allow the three gages to accurately pick up the true rainfall, on a long-term average.
The most important point to make supporting the adequacy of the three gages is that they all lie near the area for which flow is computed using simulation. The northern portion of the watershed is poorly gaged, but flow from this region is computed using flow separation on the gaged streamflow. The southern portion is also poorly gaged and is analyzed using both flow separation and simulation. The southern portion is also the area where rainfall amounts are most suspect.

One check that could be performed on the adequacy of rain gages is to run the same model for the period of 1990-92 , twice - once using the 25 precipitation polygons (from the 1990 model) and once using the 3 precipitation polygons.

Response #3

This effort has been completed and is provided in the runoff sensitivity analyses, appendix B.

<u>HEC #4</u>

4. The HSPF and SCALP models have evolved and been tested and proven to produce expected results over several years. Aside from items mentioned in other paragraphs herein, it is likely that these models are quite mature and stable, free of significant errors in both hydrologic characterization and input setup. Due to the inherent large data sets required, and the fact that the data were retrieved from diverse sources and subsequently manipulated, it is, unfortunately, possible that errors do exist in the data. Visual means are recommended for detecting any such errors. Since all data is in DSS format prior to model execution (see table 2), DSPLAY is a candidate tool for simply plotting time series data to look for anomalies. A more thorough approach would be to use Water Control Software to screen the data. Datcheck and Datvue criteria files may already exist for nearby areas.

An additional reason to double check the data is that credibility of the current effort is almost more important than accuracy. If, when the models and data become public, an error is found in the basic data, credibility of the models could be damaged disproportionately to the inaccuracy caused by the bad data.

Response #4

This District recognizes the requirement for data credibility. Reasonable efforts have been undertaken to insure that the base data is accurate. The only significant error in draft report was the use of erroneous rainfall data for WY 83. This error was in the National Weather Service database, and was corrected based on the District's concern.

<u>HEC #5</u>

5. We recommend that simplified techniques are attempted to adjust historical streamflow records to reflect urbanization (see paragraph 19). Use of event rainfall runoff models for this purpose, as noted in paragraph 33, would be unjustifiably time consuming. There are a few points to consider regarding a proposed simplified approach whereby the percent increase in the two year flow for the North Branch is applied to the other streamflow records in a linear fashion between the beginning and ending of the period of record. First, since large flows are less affected by urbanization, two trends should be extracted from the North Branch analysis, that for the two-year event and also for the 50-year event. Then the adjustment to the historical streamflow records should proceed such that each value is adjusted based on its magnitude as well as where it lies between the beginning and ending of the period of record. The second issue is that the results from the North Branch should only be used if the urbanization parameters are adequately physically based, and if the results of the North Branch behave as expected.

In particular, the adjusted and original curves should diverge at lower flows and converge at larger flows.

Response #5

For convenience, paragraphs 19 and 33 are listed below:

19. The historic streamflow records do not account for increases in urbanization. Equivalent precipitation amounts would produce larger streamflows today than were measured in 1950 due to increased urbanization. This would result in higher runoffs from the streamflow separation techniques since streamflows would be higher. This study does not quantify the effect of changes in urbanization where runoffs are derived from streamflow records.

33. The significant point to be made in the evaluation of the rainfall records, is that for each station, or each category of watershed, there was a small, but consistent increase in the average annual rainfall each year. Again, this small, but consistent increase also occurs in the watershed runoff records. The initial conclusion from this is that the rainfall is increasing over time. This statement is in consistent with studies performed by the Illinois State Water Survey. Whether or not the long term average runoff should take this factor into account is dependent on if the long term weather patterns are cyclic, and if the period analyzed here is a representative portion of a complete cycle. A second conclusion is that the models are consistent, in that station rainfall, watershed rainfall, and watershed runoff all increase slightly, on average, from year to year. Finally, the increase in runoff from gaged watersheds is slightly less than the increase in runoff from sewered (i.e. simulated) watersheds. This is likely to be true because the sewered watersheds were modeled using a single land use condition (WY89 models), while the gaged runoff data is a function of the land use in the basin during the time periods in which the measurements were recorded. To correct this potential problem, an update of the gage record to existing conditions (using existing single event rainfall-runoff models) should be undertaken.

This effort has been completed and is documented in appendix A in the section on "Gaged Watersheds."

<u>HEC #6</u>

6. Adjusting the historical streamflow data to current urbanization conditions has an extra benefit. The current streamflow separations being carried out use inaccurate historical sanitary flows. Adjusting the historical streamflow data to current urbanization conditions would allow the more accurate current sanitary flows to be used in the streamflow separation scheme.

Response #6

Comment noted. Because of the shift (through the weighting process) to existing condition gage records, only the WY 1990 sanitary flows were used in the streamflow separation process.

<u>HEC #7</u>

7. Estimates of historical sanitary discharges are suspect but efforts to refine them are contingent upon whether or not the streamflow records will be adjusted to reflect current conditions. If the streamflow records are adjusted, current sanitary conditions can be used in the flow separation scheme.

Response #7

Comment noted. Because of the shift (through the weighting process) to existing condition gage records, only the WY 1990 sanitary flows were used in the streamflow separation process.

HEC #8

8. Although the subject report only proposes using the 1990 model to perform the period of record analysis, subsequent conversations indicate that efforts are underway to reevaluate using that model. We concur that the 1990 model should be used because of the updated land use conditions in the 1990 model and endorse that the results from this effort supersede those of the previous model.

Response #8

Comment noted. Appendix A provides the documentation for the District's use of the WY 90 diversion accounting model.

HEC #9

9. We propose that overestimation of flows at treatment plants with the 1990 model could be due to recent corrections in the models to more accurately characterize the watershed.   Since previous calibration occurred with the errors in existence, it is possible that calibration parameters were adjusted to compensate for the errors. When the error is corrected but the compensating parameter is left unchanged, new errors can result.

Response #9

It is the District's position that the calibrations of the HSPF/SCALP models are adequate. Appendix B, covering runoff sensitivity analyses, points out that the average error in matching treatment plants flows over the period of WY 90-92 is 1.3% (28 cfs). Therefore, any compensation difficulties in the recalibration process ought to result in only negligible errors.

HEC #10

10. An issue raised in the report is the difference between the NIPC results (see memo, 8 Dec 1987) and the results of the subject effort. We concur with the explanations for these differences given in paragraphs 35, 36, and 37. The difference in precipitation data used between the two models (paragraph 38) needs further investigation. The use of the single Midway gage in the NIPC effort is inferior to the use of the three gages in the subject effort. Since the annual precipitation value for Midway is higher than those for the other two gages (35.3" versus 32.9" and 34.8") there remains uncertainty as to how the NIPC analysis could produce significantly lower flow, ignoring the more important differences in paragraphs 35 - 37. To check this, we propose running the same model using only the Midway gage, as well as with all three gages to quantify the impact of basin transformation on the rainfall record. This proposal is intended as an option to be considered if paragraph 34 becomes a point of contention based on paragraph 38. Otherwise, the effort is not warranted because of the dominating impact of the model differences described in paragraphs 35-37.

For convenience, paragraphs 34-38 are listed below:

34. The runoff study by NIPC resulted in an annual runoff of 636 cfs while the Chicago District's study resulted in an annual average of 745 cfs. A comparison of methodologies used provides an explanation of the difference in results. Four primary differences in methodology will be discussed. The four differences are with respect to differences in:

- *Period of record*
  NIPC: WY49-WY79
  Chicago District: WY51-WY94
- *Model parameters*
- *Determination of runoff from streamflow areas*
- *Precipitation data employed in the models*

35. The period of the record explains some of the difference of 109 cfs between the two studies. NIPC used a 31 year period of record (WY49-WY79) while the District used a 44 year period of record (WY51-WY94). The District's study shows an average annual runoff of 696 cfs over the 29 year period of record portion common to both studies (WY51-WY79) while the average annual runoff for the 15 year period beyond the NIPC study (WY80-WY94) was 840 cfs. Weighted Lake Michigan watershed precipitation from the three gages used in the District's study was 34.56 inches over the common period (WY51-WY79) and 38.02 inches over the period beyond the NIPC study (WY80-WY94).

36. The second contributor to the differences in results between the District and NIPC is the use of different model parameters. NIPC used models that were in existence for the WY83 accounting. Some of the model parameters were questioned by the Second Technical Committee and subsequently revised by Christopher Burke Engineering, Ltd. under contract to the District. The model revisions incorporated in the WY84 accounting resulted in increasing the runoff component of flow in the sewers. The revised models used in this runoff study showed a large improvement in simulated to recorded ratios at the MWRDGC WRPs when compared to model results from WY83. Refer to the 1989 Annual Report containing the WY84 and WY85 Accounting Reports for additional details. The revised models simulated approximately 43 cfs and 49 cfs more sewer runoff for WY84 and WY85 than the models used by NIPC.

37. The third major difference between the District's and NIPC's studies is the method of determining runoff at stream gage sites outside the MWRDGC WRP service areas. The method employed by NIPC was to fully simulate those areas. The District employed streamflow separation techniques. The streamflow *separation is superior to modeling in these areas since it helps to account for the complex hydraulics of the rivers in the southeast portion of the diverted watershed. Additionally, streamgages help to capture changes in the isohyetal precipitation distribution since they actually measures flows resulting from localized storms. Because NIPC used Midway Airport as the sole precipitation gage, variances in storm distributions go unnoticed. NIPC's use of modeling the streamflow areas does however account for more current urbanization effects on runoff whereas the District's streamflow records would require adjustment to better capture changes in runoff due to urbanization.*

38. The final primary difference between the two studies is in the precipitation data incorporated into the modeling. NIPC used only the Midway Airport gage while the District incorporated the O'Hare Airport and University of Chicago along with the Midway Airport gage. At first glance it would appear that the NIPC study would result in slightly higher runoffs due to precipitation since the Midway gage measured an annual precipitation of 35.3 inches over the period WY51-WY79 while the O'Hare and University of Chicago precipitation gages measured 32.9 and 34.8 inches respectively. However, the Midway gage tends to measure low during extreme events which may tend to negate some of the runoff. It is typical for more extreme events to produce a larger proportion of runoff, as compared to infiltration and evapotranspiration loses.

Response #10

This effort has been completed and is provided in the runoff sensitivity analyses, appendix B.

HEC #11

11. We understand that your efforts are ongoing to increase the accuracy of estimates of flow for the Grand Calumet River, the most uncertain flow component. The efforts include the use of the HSPF and UNET models (developed for the area as part of an unrelated feasibility study) to check the regression equations. Additional comment on this part of the study will be postponed until these efforts are completed. Although the sensitivity analysis proposed elsewhere will show a corresponding wide range in the current estimates of flows for the Grand Calumet, it is still anticipated that the impact of this range on the total flow from the Chicago River Watershed will be small.

Response #11

This effort has been completed and is provided in the period of record runoff analysis, appendix A.

HEC #12

12. The % imperviousness used for the 80 square mile ungaged Calumet watershed should be checked. The pre 1990 model used 10 % (estimated in the report to represent land use of the early 80's) and the post 1990 model uses 40 %. It is questionable that there was a 30% increase during the period - which could cast doubt on the accuracy of the 1990 values. However, the new assessment was carefully performed using areal photographs. It is likely a good estimate. The report should document the confidence in the value estimated in the 80's so the reader can cast appropriate doubt on those values instead of on the 90 values (presumably, the earlier estimate was performed in a less

rigorous fashion than the 90 estimate). Because the % imperviousness is such an important element of the runoff computation, it should be anticipated that it will receive a proportionate amount of scrutiny. Since there is high confidence in the values used, confidence is bolstered for the resulting estimated runoff and the report should support this confidence by allowing appropriate confidence in the 80's value. (Based on conversations with District, earlier value may be based on early 70's. This should be verified and made clear in the report as it is more believable that the 30% increase occurred over a period of twenty instead of ten years.

Response #12

The 40% imperviousness does appear to be excessive. It has been adjusted to 25%, see the runoff appendix A.

HEC #13

13. We recommend that in the instances where regression equations are used to fill in streamflow records, a simple visual comparison is made with the rainfall hyetographs from nearby gages to ensure reasonableness of filled in streamflow records.

Response #13

Regression equations were used to fill in streamflow records at the Munster gage on the Little Calumet River, and for the Grand Calumet River flow analysis, see appendix A. Due to time constraints comparisons were not made with rainfall hyetographs at local gages. Additionally, the District also believes that these comparisons would be relatively unimportant and would probably not provide any additional verification. In the Little Calumet River, the Munster and South Holland gages (the source of data for the Munster regression equations) are so close that any precipitation effects would be drowned out by the complex hydraulics and flow reversals generated by the storage effects in the east reach of the basin. Further, the regression equations used to fill in data on the Grand Calumet River are far more dependent on treatment plant flow splits than on rainfall.

HEC #14

14. We do not recommend modifying the precipitation data during the period of record (see paragraphs 30 -33). The regression should not be the sole basis for the conclusions regarding the increase. If the Illinois State Water Survey has found similar increases, then their analysis should be included in an addendum. Otherwise, the sample size is too small for the inferences made, or at least, the sample size required for such inferences is a controversial point. We concur that further analysis should be undertaken before the models are changed to address increased rainfall amounts.
For convenience, paragraphs 30-33 are listed below:

30. The second phase of the analysis of the period of record flows consists of a review of a series of trend analyses that compare increases over time in station rainfalls, modeled rainfalls, and modeled runoffs. Attachment 2 lists the rainfall for each of the Midway Airport, O'Hare Airport and University of Chicago *stations. The attachment also provides the basic statistics (minimums, maximums, averages and standard deviations) for each of the stations, as well as an analysis of a 5-year running average for each station. However, what are most useful from the attachment are the results of the linear regression done for each station. In the analyses the year (independent variable) was regressed with the station precipitation (dependent variable). Although the coefficients of determination for the three regressions are low, the results all suggest that the precipitation is increasing over time (from 0.14 inches/year at Midway Airport and the University of Chicago, to 0.19 inches/year at O'Hare Airport).*

31. Attachment 3 provides annual rainfall for each water year used for the gaged areas, the sewered areas (i.e. simulated) and total areas. In a manner similar to attachment 2, this attachment also provides the basic statistics (minimums, maximums, averages and standard deviations) for each of the watersheds, as well as an analysis of a 5-year running average for each type of watershed. Likewise, what are most useful from the attachment are the results of the linear regression done for each category of watershed. In the analyses the year (independent variable) was regressed with watershed precipitation (dependent variable). Again the coefficients of determination for the three regressions are low, but the results all suggest that the annual precipitation is increasing over time (approximately 0.16 inches/year).

32. Listings of the watershed runoff for gaged areas, sewered areas and the total areas is provided in attachment 4. The basic statistics and an analysis of 5-year running averages are also included. As with the rainfall, the most important information provided in the attachment is the results of the three regression analyses of year versus runoff. These regressions showed that runoff was increasing 3.7 cfs/year for the gaged watersheds, 4.7 cfs/year for the sewered watersheds, for a total increase of 8.3 cfs/year.

33. The significant point to made in an evaluation of the rainfall records, is that for each station, or each category of watershed, there was a small, but consistent increase in the average annual rainfall per year. Again, this small, but consistent increase also occurs in the watershed runoff records. The initial conclusion from this is that the rainfall is increasing over time. This statement is in consistent with studies performed by the Illinois State Water Survey. Whether or not the long term average runoff should take this factor into account is dependent on if the long term weather patterns are cyclic, and if the period analyzed here is a representative portion of a complete cycle. A second conclusion is that the models are consistent, in that station rainfall, watershed rainfall, and watershed runoff all increase slightly, on average, from year to year. Finally, the increase in runoff from gaged watersheds is slightly less than the increase in runoff from sewered (i.e. simulated) watersheds. This is likely to be true because the sewered watersheds were modeled using a single land use condition (WY89 models), while the gaged runoff data is a function of the land use in the basin during the time periods in which the measurements were recorded. To correct this potential problem, an update of the gage record to existing conditions (using existing single event rainfall-runoff models) should be undertaken.

Response #14

The District concurs with conclusion that it is impossible to evaluate or compute climatological changes based on the information presented in this report. Kenneth Potter, Ph.D. of the University of Wisconsin, a member of the mediation's technical team, has also stated that it his opinion that without very long term information it would be unwarranted to make changes in rainfall records. The District has provided a qualitative sensitivity assessment of this effect in the main report. However, any further efforts are considered to be out of the scope of this analysis.

HEC #15

15. In several instances, the flows resulting from period-of-record analysis are compared to flows generated during past efforts to estimate annual diversions. The latter flows are labeled "certified flows" in the context of the comparisons. The use of the term "certified" is somewhat misleading in that the computation of runoff from the Chicago watershed was concomitant to the determination of annual diversions. A slightly longer and more accurate description should be used for the Chicago watershed runoff computed ancillary to previous annual diversions. Comparing the results of the current effort to the poorly titled "certified" results lends unearned credibility to the results of the current effort.

Response #15

Comment noted, and the label "certified flows" has been changed to "diverted flows" or "diversion accounting flows." However, it should be noted, that although the runoff analysis provided in the accounting reports is not completely consistent with the procedures used here, the results are directly used in the diversion accounting process and are based on state-of-the-art procedures. Therefore these flows are indeed "certified."

<u>HEC #16</u>

16. Conclusions/recommendations:

a. A summary of our findings is presented below in the form of a table. Each item has with it a qualitative sense of our concern, as well as the impact that item may have on the total model results.

Issue	Concern	Impact of item on total model result
Sparse rain gages	Moderate	Low
Data errors	High	High
Adjusted Historic Flow	Moderate	Low
Sanitary flow estimates	High	Moderate
1990 model use	Null	Moderate
Error correction without re-calibration	High	Low
NIPC model vs. NCC model	High	High
Grand Calumet River	High	Low
% impervious for Calumet	Low	Low

Issue	Concern	Impact of item on total model result
Regression equations for data fill-in	Low	Low
Modifying rainfall for trend	Low	Low

Using “certified” as label	Low	Null

b. We recommend that sensitivity analysis be performed on the final Chicago watershed models. The purpose is to quantify the potential impacts of suspected model inaccuracies/inadequacies. After all model changes have been finished and the data has been double checked, the following approach is proposed. Identify a select few aspects of the Chicago runoff calculation which have one or more of the qualities of impacting the results greatly, or being particularly suspect, or controversial. An obvious candidate for the first category is the % imperviousness. This has a large impact on simulated flow. Estimate two values, higher and lower than the current best estimate based on an intuitive possible range in this value. Since the studies to estimate the current values seem to be thorough, the range will be small - but there is a range - say plus or minus 5 % of the value.

c. The regression equation for flow at the Grand Calumet River is typical for the second category in that it is suspect. Although the range estimated for this will be great, the total impact on runoff from the entire Chicago watershed is small. Consider other such aspects of the model and create a table with low and high estimates. Here, low and high imply that the estimates will impact the results by lowering them and raising them.

d. Obviously, this effort can run away and quickly produce a large range of flow estimates. However, through judicial selection of model elements to vary, a convincing argument can be made for a comment along the lines of “it is improbable that the true average annual runoff is below ____ or above ___.” This approach is really a way to put into numbers the general feeling that the components of the runoff computation which are uncertain are fortunately minor relative to the total flow, and that the major components carry with them a high level of confidence. Also, a range of possible average annual runoff values might be more useful in negotiating a single adopted value.

e. The modeling effort described in the subject report involved extensive analysis. Several items described here need to be addressed. Also, the subject report should be reorganized to make it clearer to the reader where uncertainties remain, and the impact they have on the final results. Some suggestions have been provided for doing this.

Response #16

The District has resolved all of the issues having a high (data errors and NIPC model versus NCC model) or moderate (sanitary flow estimates and 1990 model use) impact on the results. Additionally the District has completed technical analyses for a number of issues that may have only a low impact (sparse rain gages, adjustment of historic flows *and Grand Calumet River). For the remaining low impact issues the District believes that no further analysis is warranted (error correction without recalibration, regression equations for data fill in and modifying rainfall for trend), or that further investigation is required (% imperviousness for Calumet).*

Due to time constraints, the sensitivity analyses undertaken in this report have been limited to the information provided in the qualitative assessment of sensitivities provided in the main report, the sensitivity analysis for the period of record runoff, provided in appendix B and the consumptive use sensitivity analysis provided in appendix C.

It is hoped that the qualitative assessment of sensitivities in main report serves to clarify where uncertainties remain in the analysis and what impacts they may have on the final results.

Attachment E-2
Comments from the
State of Illinois

Comments from the State of Illinois

SoI #1

In general, I support your list of eleven items which you distributed at the mediation session. Some of the items, such as checking the outliers, recalibrating at the West/Southwest Plant and checking for data entry errors are extremely important. Some items, such as generating runoff using existing conditions (1990), need careful explanation so that the reader will understand that your results are based on historic rainfall and current land use conditions. This combination will yield runoff values that will be higher that what actually occurred in the past.

Response #1

The eleven items are listed below (I-1 through I-11), along with comments regarding the incorporation of the update (C-1 through C-11).

Basic Assumptions

I-1. Base data will be verified using a variety of approaches (redundancy checks, plotting and limit checks, evaluation of high and low outliers).

C-1. Base data has been reviewed, and we have found no further discrepancies.

I-2. Drainage areas will be screened to minimize any double counting of flows.

C-2. Drainage areas have been screened, see appendix A.

I-3. The watershed runoff will be generated for existing conditions (approximately 1990).

C-3. The runoff from gaged and simulated areas has been computed for 1990 conditions, see appendix A.

Gaged Areas

I-4. Flows from gaged areas will be adjusted to reflect existing conditions.

C-4. Gaged area runoff has been modified for 1990 conditions, see appendix A.

I-5. Flows from the Grand Calumet River will be evaluated, and possibly updated, using existing simulation models.

C-5. The flows from the Grand Calumet River have been updated using existing simulation models and revised regression equations, see appendix A.

Sewered Areas

I-6. The Water Year 1990 diversion accounting model will be used to simulate the flows in the sewered areas.

C-6. The Water Year 1990 diversion accounting model have been used to simulate the flows for 1990 conditions, see appendix A.

I-7. The Water Year 1990 model will be "re-calibrated" to reproduced a truer simulation of the volume at the West Southwest Water Reclamation Plant.

C-7. The Water Year 1990 diversion accounting model was <u>not</u> re-calibrated." Appendix B, covering runoff sensitivity analyses, points out that the average error in matching treatment plants flows over the period of WY 90-92 is 1.3% (28 cfs). It was felt that the accounting models were reasonably calibrated for this period, and the extensive work required for a finer calibration was not warranted at this point. The potential impact of this possible "high calibration" is covered in the qualitative sensitivity analysis in the main report.

I-8. Historic precipitation values from Midway Airport, the University of Chicago and O'Hare Airport will be used to compute yearly runoff values.

C-8. The precipitation values from the three gages were used in computing the runoff values. A sensitivity analysis of this procedure is described in appendix B.

I-9. No adjustments will be made to the precipitation records to reflect any trends or cycles.

C-9. It has been suggested by both the Hydrologic Engineering Center, and by Kenneth Potter, Ph.D. of the University of Wisconsin that it is impossible to evaluate or compute climatological changes based on the information utilized in this report. The District concurs with this conclusion, and outside of the qualitative sensitivity assessment presented in the main report, any further efforts with respect to this are considered to be outside the scope of this analysis.

<u>Evaluation of Results</u>

I-10. A sensitivity analysis will be conducted to give a range of watershed runoff values.

C-10. The sensitivity analyses provided in this report include the qualitative assessment of sensitivities provided in the main report, sensitivity analyses of the period of record runoff provided in appendix B and the consumptive use sensitivity analysis provided in appendix C.

I-11. Review comments provided by the Hydrologic Engineering Center (HEC) and the Task Group, subject to funding and scheduling constraints, will be evaluated and incorporated into the analysis.

C-11. This appendix provides responses to all review comments.

<u>SoI #2</u>

Since it appears that this report will be the primary source of information that will be used to establish a fixed runoff value, your discussion of the results of your simulation will be carefully scrutinized, especially as it relates to an estimation of the 'current' average stormwater runoff.

Response #2

Noted.

<u>SoI #3</u>

There was discussion at the mediation session of periodically updating the stormwater runoff number, say every 10 years. You may want to address this issue since I assume that you will lose your capability to do this kind of modeling if we move diversion accounting to the lakefront.

Response #3

In accordance with the Supreme Court Decree, the accounting models are state-of-the-art and reflect the current existing conditions. Concur with your comment. Unless the Diversion Accounting program is appropriately budgeted, it would be impossible to maintain current state-of-the-art models for periodically updating the stormwater runoff number.

<u>The following comments have been made in the margins of the draft runoff report:</u>

<u>SoI #4</u>

With regards to paragraph 12's discussion of inflow/infiltration and imperviousness - "Does this include consideration of I/I control, i.e. disconnected downspouts, etc.?"

For convenience, paragraph 12 is listed below:

12. The HSPF unit runoffs were used as input to the hydraulic sewer routing model, SCALP, which models the sewers contained in the service areas of the four MWRDGC water reclamation plants: West Southwest (Stickney), North Side, Calumet, and Lemont. Flows are generated for 214 sewer sub-basins known as Special Contributing Areas (SCAs). Both combined and separately sewered areas are modeled. Sanitary flow estimates are based on population equivalents within each SCA. Sewer inflow and infiltration for each SCA are based on impervious and pervious areas within each SCA. The impervious and pervious areas are determined through a thorough study of land uses within each SCA. Each land use has associated with it assumed percentages of impervious and pervious areas. Population equivalents and impervious and pervious areas for each SCA are based on the WY89 models.

Response #4

In general, the percent imperviousness should be equal to the percent of directly connected area. This would tend to take into account, at least from the selection of a percentage and from the calibration, reduced I&I and disconnected downspouts. Based on a review of the % imperviousness, the District has made modifications to the ungaged Calumet flows and to the overflows (see the runoff analysis, appendix A)

<u>SoI #5</u>

With regards to paragraph 13's development of sanitary flows - "What did you use? Sources? Accuracy?"

For convenience, paragraph 13 is listed below:

13. Sanitary flows are computed by multiplying population equivalents by per capita sanitary estimates. Monthly, daily, and hourly multipliers are used to simulate changes in sanitary flow generation from month to month, day to day, and hour to hour. Sewer inflows are computed by multiplying impervious surface unit runoffs (IMPRO) and grassland surface unit runoffs (OLFRO) by the impervious and pervious areas falling within the polygon of the corresponding precipitation for which the unit hydrographs were computed. Some SCAs fall within more than one of the thirteen precipitation polygons. Sewer infiltration is computed by multiplying the grassland subsurface unit runoffs (SUBRO) by the pervious areas falling within the polygon of the corresponding precipitation gage. SCALP outputs both interceptor flows and overflows and keeps track of the three constituent flows: sanitary, inflow, and infiltration.

Response #5

The population equivalents used is a carryover from NIPCs construction of the accounting models. However, the sanitary estimates from each SCA have been updated based on domestic water supply pumpages. This was accomplished by revising the per capita sanitary usage values for each SCA by dividing the water supply pumpage by the existing population equivalents. Although changes in sanitary discharge are more likely a result of population changes rather than changes in per capita usage, the population changes since 1980 could only be reflected through a revision of the per capita sanitary usage values. Revised population equivalent values are not readily available, and the District is currently attempting to determine the source of the original values.

<u>SoI #6</u>

With regards to paragraph 15's discussion of the impervious values for the Calumet Watershed - "Does this translate into more runoff? Impact of stormwater detention program?"

For convenience, paragraph 15 is listed below:

15. The first two items above are solely from HSPF and SCALP simulations based on the WY89 models. The two models used are calibrated against the four MWRDGC WRP influent pumpage records. Statistical analyses at the four MWRDGC WRPs, for each Water Year, show a good correlation, both with respect to the correlation coefficient and the simulated to recorded ratios. However, it must be pointed out *that prior to WY90 the total simulated flows were somewhat less than the total recorded inflows at the four water reclamation plants. The revised models used for WY90 and thereafter show total simulated flows that are slightly higher than the total recorded inflows at the water reclamation plants. Specific values can be found within the individual accounting reports published within the Lake Michigan Diversion Accounting Annual Reports. Additionally, the simulated ungaged Calumet Watershed has been found, during WY90 modeling revisions, to contain significantly more impervious area than was modeled for WY89. The increased runoff due to higher percentages of imperviousness is not reflected in this study.*

Response #6

This increase in impervious area does result in increased runoff from this area and it is reflected in the revised runoff study. The effects of a stormwater detention program were not directly modeled. However, the adjustment of calibration parameters should ultimately reflect the impacts of such a program.

SoI #7

With regards to paragraph 19's comments on the effects of urbanization on streamflow records - "They most certainly do! Confusing, since you're using 1990 land use isn't runoff over estimated?"

For convenience, paragraph 19 is listed below:

19. The historic streamflow records do not account for increases in urbanization. Equivalent precipitation amounts would produce larger streamflows today than were measured in 1950 due to increased urbanization. This would result in higher runoffs from the streamflow separation techniques since streamflows would be higher. This study does not quantify the effect of changes in urbanization where runoffs are derived from streamflow records.

Response #7

The streamflow records, not the simulation models, initially did not account for urbanization in that the historical records reflect prevalent urbanization conditions at the time of the measurement. The revised study has adjusted the streamflow records such that they reflect 1990 urbanization conditions, see the revised section on "Gaged Watersheds" and the revised attachment A-1.

SoI #8

With regards to paragraph 25's comment on the initial reports underestimation of the Grand Calumet River flow - "How do you know if pumpage has remained relatively constant? If runoff would not be under estimated."

For convenience, paragraph 25 is listed below:

25. The computed Grand Calumet runoff is also in question due to the unavailability of water supply pumpage data. The Lake Michigan runoff in the Grand Calumet flow was calculated by subtracting the water supply pumpage for Whiting, East Chicago, and Hammond, IN from the simulated river flow. The modeled period of record assumed the fixed WY89 pumpage of 74.7 cfs since historic records were not *readily available. This results in an underestimated runoff from the Grand Calumet River. All Grand Calumet runoff flows below zero were set to zero. Using these regressions and the runoff computation, the runoff was zero for 82.4% of the period of record.*

Response #8

It is highly unlikely that the water supply pumpage in the early portions of the period of record were the same as that during WY89. However, this comment is moot since the Grand Calumet River runoff computation has been revised. Refer to appendix A.

SoI #9

With regards to table 1's simulated runoff for 1983 and 1993 - "1301.5? 1377.1?"

For convenience, table 1 is listed below:

Table 1 Simulated Runoff

W Year	*Runoff*	*W Year*	*Runoff*	*W Year*	*Runoff*	*W Year*	*Runoff*
1951	*762.3*	*1962*	*546.9*	*1973*	*1122.8*	*1984*	*921.2*
1952	*748.9*	*1963*	*302.8*	*1974*	*1032.2*	*1985*	*825.3*
1953	*428.1*	*1964*	*368.7*	*1975*	*974.6*	*1986*	*916.7*
1954	*544.7*	*1965*	*808.7*	*1976*	*750.2*	*1987*	*839.4*
1955	*718.3*	*1966*	*797.0*	*1977*	*454.2*	*1988*	*537.6*
1956	*441.3*	*1967*	*775.3*	*1978*	*720.5*	*1989*	*724.4*
1957	*714.6*	*1968*	*656.8*	*1979*	*926.6*	*1990*	*781.5*
1958	*505.4*	*1969*	*800.4*	*1980*	*611.2*	*1991*	*849.9*
1959	*609.0*	*1970*	*902.7*	*1981*	*804.3*	*1992*	*685.3*
1960	*736.6*	*1971*	*597.9*	*1982*	*825.6*	*1993*	*1301.5*
1961	*557.5*	*1972*	*881.7*	*1983*	*1377.1*	*1994*	*603.9*

Notes: 1. Minimum: 302.8 cfs 2. Maximum: 1377.1 cfs
3. Average: 745.3 cfs 4. Stan Dev: 220.7 cfs

Response #9

The values were correct for the initial draft level of analysis. However, they have since been revised in this updated runoff study.

SoI #10

With regards to table 2's statement concerning the changes to the simulation modeling 1986 and 1987, and the effect of TARP on runoff - "Is this true?"

For convenience, a portion of table 2 (for 1986 and 1987) is listed below:

Table 2 Changes Effecting Runoff Simulations

Water Year	Modification
1986	The Mainstream System TARP was incorporated into the model. TARP has no effect on the runoff.
1987	The Calumet System TARP was incorporated into the model. TARP has no effect on the runoff.

Response #10

Yes, it is true that TARP has had no effect on the computed runoff. The portion of runoff from the Lake Michigan watershed that enters TARP does so only through CSOs. Those CSOs that were routed to TARP previously discharged to adjoining rivers. In either case, they are accounted for.

SoI #11

With regards to paragraph 32's analysis of increasing runoff - "How much is due to precipitation increases? To urbanization?"

For convenience, paragraph 32 is listed below:

32. Listings of the watershed runoff for gaged areas, sewered areas and the total areas is provided in attachment 4. The basic statistics and an analysis of 5-year running averages are also included. As with the rainfall, the most important information provided in the attachment is the results of the three regression analyses of year versus runoff. This regression showed that runoff was increasing 3.7 cfs/year for the gaged watersheds, 4.7 cfs/year for the sewered watersheds, for a total increase of 8.3 cfs/year.

Response #11

In the draft report the linear fits are merely trend lines, and not statistically significant regression curves. As such the information provided in the draft report (e.g. an increase of 8.3 cfs/year) is an overstatement of the significance of the available data. Therefore, this assessment has been deleted from the current version of the sensitivity analyses.

SoI #12

With regards to paragraph 33's comments on the increasing trend in rainfall - "Lake Michigan also receives this increased rainfall. Why not discuss and estimate the increased volume from this?"

For convenience, paragraph 33 is listed below:

33. The significant point to made in an evaluation of the rainfall records, is that for each station, or each category of watershed, there was a small, but consistent increase in the average annual rainfall per year. Again, this small, but consistent increase also occurs in the watershed runoff records. The initial conclusion from this is that the rainfall is increasing over time. This statement is in consistent with studies performed by the Illinois State Water Survey. Whether or not the long term average runoff should take this *factor into account is dependent on if the long term weather patterns are cyclic, and if the period analyzed here is a representative portion of a complete cycle. A second conclusion is that the models are consistent, in that station rainfall, watershed rainfall, and watershed runoff all increase slightly, on average, from year to year. Finally, the increase in runoff from gaged watersheds is slightly less than the increase in runoff from sewered (i.e. simulated) watersheds. This is likely to be true because the sewered watersheds were modeled using a single land use condition (WY89 models), while the gaged runoff data is a function of the land use in the basin during the time periods in which the measurements were recorded. To correct this potential problem, an update of the gage record to existing conditions (using existing single event rainfall-runoff models) should be undertaken.*

Response #12

It has been suggested by both the Hydrologic Engineering Center, and by Kenneth Potter, Ph.D. of the University of Wisconsin, that it is impossible to evaluate or compute climatological changes based on the information utilized in this report. The District concurs with this conclusion, and outside of the qualitative sensitivity assessment presented in the main report, any further efforts with respect to this are considered to be outside the scope of this analysis.

SoI #13

With regards to paragraph 40 - "This seems like a very big increase. 40% of this watershed is paved over, buildings?"

For convenience, paragraph 40 is listed below:

40. The WY89 models were initially used for this study so as to utilize as many of the actual precipitation records as possible from the HSPF models (i.e., polygon areas). However, only three of the original thirteen gages were used. As previously discussed, precipitation for the other 10 gages was synthesized based on overlapping Thiessen polygons. This same method can be used in synthesizing precipitation over the period of record for the 25-gage precipitation network that has been incorporated in the WY90 diversion accounting. By doing so, the modeling will reflect 1990 perviousness and imperviousness values, derived from a detailed land use study of 1990 aerial photographs. The WY90 models show an increase in the overall imperviousness of the diverted watershed, especially in the 80.2 square mile ungaged Calumet watershed (which increased from ten percent imperviousness to forty percent imperviousness). It must be noted that the Pre-WY90 models generally simulated total sewer flows that were slightly less than total measured inflows at the four MWRDGC WRPs. Post-WY90 models exhibited responses that tended to simulate sewer flows that were greater than the total measured sewer flow at the MWRDGC WRPs. See table 3 for the actual simulated to recorded ratios from WY84 through WY92. If the measured flows are accurate, then the actual sewered flows should fall somewhere between those simulated by the pre-WY90 and post-WY90 models. Additionally, quality assurance/quality control (QA/QC) reconnaissance missions must be conducted in order to validate the accuracy of the measuring components used at the MWRDGC WRPs before drawing concrete conclusions concerning the accuracy of simulated sewer flows.

Response #13

The 40% imperviousness does appear to be excessive. It was revised to 25%; see the runoff analysis, appendix A.

Attachment E-3
Comments from the

State of New York

Comments from the State of New York

NY #1

1. Calibration of some of the Corps' diversion accounting models is founded in part on wastewater plant discharge flow values submitted by operators of such facilities in the Chicagoland area. To the extent that these flow values are inaccurate, your model may also provide inaccurate results for average runoff values. Since exercise of the models for diversion accounting still fails to fully account for all the flow measured at Romeoville each year, inaccurate wastewater flow values may be partly to blame.

The models we are talking about are the foundation for the retrospective analyses that you have been doing to arrive at an average value for runoff from the watershed. To have confidence in your final estimates for average runoff, the calibrations of the models deserve some additional consideration. We recommend that your staff assess the condition of flow measurement equipment and procedures at the wastewater plants. If your site-specific research indicates that the situation at any of these plants warrants modification of past data, we expect that you will endeavor to recalibrate the appropriate models where necessary. Adoption of any final average runoff value in any new version of the Decree should await completion of this work. We trust that your next report will address this issue in some detail.

Response #1

Concur with the concept of quality assurance / quality control (QA/QC) of reclamation plant discharges. The District will work with MWRDGC, the USGS and/or an independent expert to review the discharge measurements at the three major reclamation plants (West-Southwest, Northside and Calumet). However, any re-calibration, or significant effort to update the average runoff is not within the District's capabilities for FY 96.

NY #2

2. Your February 2nd report concludes that rainfall on and resulting runoff from the Chicagoland watershed appear to be increasing. In fact, paragraph 32 on page 13 concludes that average runoff has been increasing at the rate of 8.3 cfs every year over the period of record being used in your studies. At the mediation plenary session, you modified that conclusion somewhat, however. Instead of relying on a linear fit to the rainfall and runoff data that can be found on figures in Attachments 2, 3 and 4, your slides at the meeting used a quadratic fit which suggested that these increases had leveled off during the last decade or so. We understand that staff of the Illinois Water Survey have concluded that rainfall on this watershed has been increasing -- a conclusion somewhat at variance with the conclusion suggested by your slides using the quadratic fit. Some parties to the Decree may believe that rainfall patterns should be addressed in the design of and numerical values for any revision of the Decree. We request that you consider this issue in detail in your next report, including discussion of observations made by others on this subject.

For convenience, paragraph 32 is listed below:

32. Listings of the watershed runoff for gaged areas, sewered areas and the total areas is provided in attachment 4. The basic statistics and an analysis of 5-year running averages are also included. As with the rainfall, the most important information provided in the attachment is the results of the three regression analyses of year versus runoff. This regression showed that runoff was increasing 3.7 cfs/year for the gaged watersheds, 4.7 cfs/year for the sewered watersheds, for a total increase of 8.3 cfs/year.

Response #2

It should be noted that for both the draft report and for the charts provided at the mediation meeting, linear or quadratic fits are merely trend lines, and not statistically significant regression curves. As such the information provided in the draft report (e.g. an increase of 8.3 cfs/year) is an overstatement of the significance of the available *data. However, the use of a quadratic fit, and the suggested trends, is within the bounds of a qualitative assessment of the information.*

NY #3

3. We would appreciate a more detailed explanation and an example of your double Theissen procedure used to balance rainfall from three gages across the entire Chicagoland watershed. We look forward to more details about the statistical limitations of this procedure.

Response #3

The double Theissen procedure, as described in the report, is a procedure for waiting precipitation data that does not require the re-creation of the polygons for each of the 200+ special contributing areas (SCAs). In performing the diversion accounting analysis, Theissen polygons have been created for the 25 rainfall gage network. To use a different series of rainfalls gages with these polygons (e.g. the network consisting of Midway, O'Hare and the University of Chicago), the procedure is to translate the new gages to the 25 gage network locations with a second series of polygons. The double weighting produces the same result as if the Theissen polygons were re-created using the three gage network for each of the SCAs. The procedure is further discussed in the sensitivity analysis of the effects of the number of precipitation stations. For further clarification, or an example of this procedure, please contact the Chicago District.

NY #4

4. We would appreciate a more detailed explanation and example of your streamflow separation technique used in several sub-watershed areas as you discuss in paragraph 6 on page 2 and elsewhere.
For convenience, paragraph 6 is listed below:

6. Based on the availability of precipitation, meteorological, and streamflow data, it was decided that the period of record would be Water Year 1951 (WY51) through Water Year 1994 (WY94). The majority of the Lake Michigan Watershed runoff was simulated using the Hydrologic Simulation Program - FORTRAN (HSPF) as well as a hydraulic sewer routing model Special Contributing Area Loading Program (SCALP). Areas that were simulated include the service areas of four Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) water reclamation facilities: West Southwest (Stickney), North Side, Calumet, and Lemont, as well as one 80.2 square mile "ungaged" Calumet watershed. The runoff from the remaining areas of the Lake Michigan watershed is based on streamgage records. This area included the runoff from the northern and southeastern extents of the watershed. In these areas, a streamflow separation technique was used, in which estimated sanitary discharges upstream of a stream gage are subtracted from a stream gage record to determine the portion of streamflow that is runoff. Total simulated area is approximately 491 square miles, while the total area using streamflow separation techniques is approximately 182 square miles. Some areas overlap in that they fall within both the simulated area and the stream gaged area. These areas are separately sewered where the sanitary sewers convey flow to the water reclamation plants while the storm sewers discharge into streams to be measured by the gages. Overlapping areas were classified as simulated areas. See map on page 3 for location of simulated, gaged and ungaged areas.

Response #4

The streamflow separation technique is described in more detail in the revised section on "Gaged Watersheds" and in the revised attachment A-1 . For further clarification, or an example of this procedure, please contact the Chicago District.

NY #5

5. We would appreciate more thorough conceptual explanation of the mass balance associated with management, pumpage and disposal of water in the Chicagoland area. A cross-section illustration of what happens to water withdrawn from Lake Michigan on the way to discharge at Romeoville would be very useful. The negotiation process might benefit from a detailed explanation of what happens to water leaking from the distribution system (in short and long term horizons) and used or consumed by the populace and commercial establishments of the region. Illustration of where flow measurements or estimates are made for the runoff models on this cross-section would also help us understand your procedures better. A description of precipitation routing in the region with respect to this conceptual mass balance would also be instructive.

Response #5

A mass balance of the rainfall-runoff process is provided in the runoff sensitivity analysis (appendix B).

NY #6

6. The role of urbanization changes on runoff and possibly climate changes (including rainfall and evaporation rates) deserve more consideration in your next report. We are not fully aware of the details of the characteristics associated various land use bases that your models could or have relied on for parametric values. The sensitivity of these aspects of the models deserves careful consideration.

Response #6

It has been suggested by both the Hydrologic Engineering Center, and by Kenneth Potter, Ph.D. of the University of Wisconsin, that it is impossible to evaluate or compute climatological changes based on the information utilized in this report. The District concurs with this conclusion, and outside of the qualitative sensitivity assessment presented in the main report, any further efforts with respect to this are considered to be outside the scope of this analysis.

NY #7

7. Like the possible inaccuracy associated with wastewater treatment plant flows and land use parameters, there may be other fundamental parameters on which your models or calibration thereof rely which we do not fully appreciate. We request that your next report list the fundamental measured or estimated parameters which influence the validity of the final values for runoff and some estimates about the sensitivity of the results to changes in these parameters. Finally, we would appreciate some explanation of how accuracy could be improved for those parameters which are have substantial impact via the numerical values for runoff.

Response #7

Due to time constraints, the sensitivity analyses undertaken in this report have been limited to the information provided in the qualitative assessment of sensitivities provided in the main report, the sensitivity analysis of the period of record runoff provided in appendix B and the consumptive use sensitivity analysis provided in appendix C.

NY #8

8. Paragraph 5, page 2, talks about estimating flows at Wilmette as a part of the proposed Lakefront accounting system. We would appreciate more detail on how this estimation might be accomplished, given the low flows and channel characteristics there.

For convenience, paragraph 5 is listed below:

5. Through the use of lakefront AVMs, the revised procedure for computing the diversion would consist of the additions of direct diversions, water supply, negotiated "constant runoff," and the subtraction of a negotiated value of consumptive use. Direct diversions would be measured at CRCW and O'Brien Lock and Dam and estimated at Wilmette Controlling Works. Lake Michigan water supply pumpages from primary (first order) users would be summed and federal pumpages subtracted along with an agreed upon percentage consumptive use. Runoff diverted from Lake Michigan watershed would be an agreed upon constant value based on an average runoff determined through a period of record simulation. The consumptive use credit would be negotiated and could be either a fixed value of a fixed percentage of the water supply. This report outlines the methodology used in the period of record runoff study.

Response #8

In performing the leakage measurements in WY 93, the USGS determined that the leakage through the Wilmette Controlling Works was less than 15 cfs. It is proposed to use the average of the USGS measurements (5 cfs) as the annual leakage, with QA/QC provided by periodic measurements.

NY #9

9. You were kind enough to explain the foundation for and interpretations of paragraph 27 on page 10 in some detail to me over the telephone. Other parties might benefit from a similar effort in text form in your next report.

For convenience, paragraph 27 is listed below:

27. The comparison of the period of record flows with those previously certified in the diversion accounting reports is given in attachment 1. The table in the attachment provides a series of period of record flows and certified flows for WY83-WY92. The initial statistics for the period show that the simulation flows have an average of 845.8 cfs, with a standard deviation of 219.4 cfs. The corresponding certified runoffs for this period have an average of 823.4 cfs, with a standard deviation of 139.3 cfs. A two-tailed paired t-test was completed for WY83-WY92, comparing the period of record versus certified flows. The results of the t-test showed that there was a 69% chance that the flows were from the same population. Additionally, the correlation coefficient (61%) for the period showed that the flows do tend to move together. Finally, a regression analysis of the period of record flows (independent variable) versus the certified runoff's (dependent variable) is presented in the first chart (Period of Record versus Certified Runoff) given in the attachment. The low coefficient of determination suggests that there could be significant errors if the regression analysis were used with period of record flows to predicate certified flows.

Response #9

A discussion of the procedures for performing statistical testing and analysis are outside of the scope of this report. However, clarifications to any statement or concept provided in any District report will readily be provided by telephone or email.

NY #10

10. In paragraph 41 on page 15, you indicate that "WY93 certified results were excluded from this analysis..." Do you mean WY83 certified results?

For convenience, paragraph 6 is listed below:

41. An estimate of the increases in the simulated runoff due to the WY90 models can be estimated from noting the differences in runoff between the period of record flows and the certified flows. From WY84-WY89 the average period of record flow is 39 cfs higher than the certified runoff. However, after the precipitation station upgrades and the recalibration efforts in WY90, the average certified runoff is 149 cfs higher, with increases of 91 cfs in WY90 and 191 cfs in WY91. This would suggest that the increase due to the WY90 models is in the range of 120 cfs to 220 cfs. It should be noted that the WY93 certified results were excluded from this analysis because of the questionable hydraulic parameters used in the computations.

Response #10

Concur with comment.

<u>NY #11</u>

11. We further recommend that your average annual runoff analyses include rainfall and other related values from WY93. It is our understanding that your office possesses most of the information needed to undertake a full accounting of that water year and possibly even 1994. We should use all the data reasonably available for estimating average runoff. At least WY93 should qualify as such.

Response #11

Preliminary diversion accounting results for WYs 93-95 are provided in the runoff sensitivity analysis, appendix B. However, because of the procedures used, the computation of the average annual runoff is not a direct function of the accounting results (see appendix A).

Addendum 1

Period-of-Record Runoff Analysis (WY51 – WY99)

Background and Summary

1. To move the Lake Michigan Diversion Accounting from Romeoville to Lakefront, a representative average annual runoff from the Lake Michigan watershed is needed. By fixing the annual runoff and consumptive use, the State of Illinois can effectively plan the long-term Lake Michigan water diversion needs and fully utilize the short-term annual diversion budget. In addition, the diversion accounting computations can be simplified significantly and the pertinent information can be disseminated to the interested parties in a more timely fashion. The original runoff analysis was done in 1996. In that effort, the period-of-record runoff analysis covered WY51 through WY94. The analysis is documented in the main portion of this report that has been officially reviewed by the USACE and the parties on the Great Lakes Mediation. This addendum documents the runoff analysis that has been augmented by extending the simulation period to WY99.

2. Using methods consistent with the initial analysis, the Lake Michigan watershed runoff analysis was extended from WY94 to WY99, and the average annual runoff decreased slightly from 785.2 cfs to 783.5 cfs. Additionally, a sensitivity analysis of the impact of the precipitation distribution on the runoff was performed using the precipitation data from 20 of 25 ISWS gages. These well-maintained gages have been available since WY90. Based on data for WY90-99, the estimated average runoff for this ten year period would increase from 818.9 cfs to 861.5 cfs.

Hydro-Meteorological Data

3. The runoff analysis in this addendum extended the previous period-of-record simulation to WY99. Although the approach of the analysis was unchanged, some modification to the data processing was necessary due to the availability of the meteorological and precipitation data. For the runoff analysis, the hourly observed data for air temperature, wind speed and solar radiation, as well as the daily observed data for dew point and cloud cover, are used in the HSPF hydrologic simulation. However, NOAA discontinued operating the meteorological station at University of Chicago in November 1994, and both air temperature and precipitation data became unavailable since then. Air temperature data at Midway is used in place of the data at University of Chicago in the extended period of the simulation.

4. The period of record was based on the three NOAA precipitation gages at O'Hare, Midway and University of Chicago. As mentioned in the previous paragraph, the precipitation data at University of Chicago is only available through October 1994. However, during WY90 through WY94 precipitation data was available at University of

Chicago and three nearby ISWS precipitation gage stations: Chinatown in the northwest, Englewood in the southwest, and South Water Plant in the southeast. The hourly precipitation data at these three gage stations was compared to the data at University of Chicago to determine which gage station would be used to substitute for the discontinued data at University of Chicago. Two statistics were examined to determine the best replacement station: the total amount of precipitation and the correlation of hourly rainfall. The total precipitation over the 5-year period (WY90-WY94) was 186.94", 203.21", 203.96" and 195.03" at University of Chicago, Chinatown, Englewood, and South Water Plant, respectively. The correlation coefficients of precipitation at the three ISWS gage stations to that at University of Chicago were .6259, .7634, and .7457 for Chinatown, Englewood and South Water Plant, respectively. Based on the above statistics, South Water Plant was selected to replace University of Chicago in the modeling based on the 3-gage precipitation network during WY95-WY99. Additionally, the precipitation data at Midway showed some brief periods of missing records, and for days and hours of missing records, data from the ISWS gage at Bedford Park was used.

5. The simulation model used for this analysis is based on the 1990 Lake Michigan Diversion Accounting (LMDA) model in which a total of 25 watershed subareas (each centered with a precipitation gage) are modeled. To fit the runoff model to the LMDA model framework, the effective precipitation in each of the 25 subareas was derived from overlaying the 3-gage Thiessen's polygons to the 25-gage Thiessen's polygons. Table 1-1 lists the weighting factors for this area-based mapping. The Thiessen's polygons were not adjusted for the replacement data from South Water Plant and "Bedford Park. This was done to simplify the modeling effort, as there is no significant loss in accuracy.

6. The sensitivity analysis was completed to evaluate the affects of the precipitation pattern on the runoff. The precipitation data used was from 20 of the 25 ISWS gage stations that are listed in Table 1-1. The following five subareas are outside the modeled Lake Michigan watershed and were not used: Westchester, La Grange, Lemont, Matteson and Chicago Heights. For this analysis the measured WY90-99 hourly precipitation data matches the simulation model's Thiessen polygon layout and was entered without any adjustments.

Runoff Analysis Results

7. Table 1-2 shows the summary of the Lake Michigan watershed runoff based on the 3-gage precipitation network.

8. Table 1-3 shows the comparison of total runoff between the two precipitation networks for WY90 through WY99. It shows that the computed runoff based on the 25-gage precipitation network is consistently higher (3-8%), with the exception of WY96, than the computed value based on the 3-gage precipitation network. The 25-gage precipitation network results in an average runoff of 861.5 cfs, which is 42.5 cfs or 5.2% higher than the runoff of 818.9 cfs resulting from the 3-gage precipitation network.

Table 1-3 also shows that the average runoff generated from the 3-gage precipitation network during WY90 through WY99 is about 36 cfs (4.6%) higher than the mean for the period of WY51 through WY99.

9. Figures 1-1 through 1-5 exhibit the precipitation distribution in the Lake Michigan watershed in WY90, WY91, WY92, WY93 and WY94, respectively. These figures show that the precipitation based on the 25-gage network is constantly higher than its counterpart based on the 3-gage network. Although annual precipitation is not the sole factor in determining the annual runoff[1], it is the most important contributing factor.

10. Figure 1-6 shows the streamgage runoff on North Branch Chicago River at Touhy Avenue in Niles, Little Calumet River at South Holland and Grand Calumet River for the period of WY51 through WY99. WY 91 and WY93 are the two wettest years in the last 10 years for the simulation. The high streamgage runoff in WY93 is confirmed by the precipitation data (see Figure 1-4) within the modeled watershed. A relatively high runoff in the Little Calumet River in WY91 can also be explained from the precipitation data (see Figure 1-2) of the gages located in the southern part of the modeled watershed. The NOAA precipitation gage at Shelby River (Lake County, Indiana) does not have data beyond WY90 to confirm the runoff trend in the Grand Calumet and Little Calumet Rivers across the Indiana/Illinois state boundary.

[1] The intensity and duration of the rainfall will influence the amount of resulting runoff. In addition, the soil condition (e.g., moisture content) and climatic condition prior to and during the rainfall also affect the runoff.

Table 1-1 – Overlay of Precipitation Polygons

Gage Number	Name	O'Hare	Midway	U. of Chicago
1	Northbrook	1		
2	Winnetka	1		
3	Des Plaines	1		
4	Skokie	1		
5	Franklin Park	1		
6	Bricktown	0.8206	0.0680	0.1114
7	Diversey Harbor	0.1210		0.8790
8	Westchester	0.4040	0.5960	
9	Cicero	0.0240	0.9253	0.0507
10	China Town		0.0606	0.9394
11	La Grange		1	
12	Bedford Park		1	
13	Englewood		0.3137	0.6863
14	South Water Plant			1
15	Lemont		1	
16	Palos Park		1	
17	Alsip		1	
18	West Pullman		0.7851	0.2149
19	Wolf Lake			1
20	Orland Park		1	
21	Tinley Park		1	
22	Harvey		0.9764	0.0236
23	Lansing		0.0883	0.9117
24	Matteson		1	
25	Chicago Heights		0.6667	0.3333

Table 1-2 – Lake Michigan Watershed Runoff (WY51-99)
Based on 3-Gage Precipitation Network

Water Year	Streamgage (cfs)	Simulated (cfs)	Baseflow (cfs)	Total (cfs)
1951	282.0	551.4	4.0	837.4
1952	356.5	460.5	4.0	821.0
1953	174.8	316.5	4.0	495.4
1954	187.1	416.6	4.0	607.8
1955	289.0	508.2	4.0	801.1
1956	158.6	290.7	4.0	453.2
1957	204.4	517.8	4.0	726.2
1958	162.5	339.0	4.0	505.5
1959	233.1	412.4	4.0	649.5
1960	305.2	454.7	4.0	764.0
1961	164.4	441.2	4.0	609.6
1962	226.4	391.4	4.0	621.8
1963	73.8	243.1	4.0	320.8
1964	92.7	275.1	4.0	371.8
1965	250.5	572.5	4.0	827.0
1966	273.9	534.7	4.0	812.5
1967	233.5	584.8	4.0	822.2
1968	224.0	449.1	4.0	677.2
1969	298.8	558.7	4.0	861.5
1970	289.2	658.8	4.0	952.0
1971	228.4	435.5	4.0	667.9
1972	345.5	557.5	4.0	907.0
1973	492.0	677.0	4.0	1173.0
1974	472.6	687.9	4.0	1164.5
1975	370.4	658.3	4.0	1032.6
1976	330.8	462.1	4.0	796.9
1977	137.9	387.5	4.0	529.5
1978	293.3	454.3	4.0	751.6
1979	340.6	601.1	4.0	945.7
1980	297.0	409.8	4.0	710.8
1981	346.4	488.5	4.0	838.8
1982	324.7	535.2	4.0	863.9
1983	422.2	876.2	4.0	1302.4
1984	353.0	606.2	4.0	963.2
1985	309.9	571.5	4.0	885.4
1986	376.5	595.6	4.0	976.1
1987	343.9	495.9	4.0	843.9
1988	215.9	351.3	4.0	571.2
1989	241.4	493.5	4.0	738.9
1990	277.4	524.1	4.0	805.5

Table 1-2 cont' – Lake Michigan Watershed Runoff (WY51-99)
Based on 3-Gage Precipitation Network

Water Year	Streamgage (cfs)	Simulated (cfs)	Baseflow (cfs)	Total (cfs)
1991	341.1	524.9	4.0	870.0
1992	235.9	475.8	4.0	715.7
1993	482.2	856.6	4.0	1342.8
1994	239.1	372.9	4.0	616.0
1995	259.9	406.7	4.0	670.6
1996	307.6	487.0	4.0	798.5
1997	272.3	479.7	4.0	755.9
1998	345.8	452.6	4.0	802.4
1999	291.1	516.6	4.0	811.7
Average	281.1	498.4	4.0	783.5
Min	73.8	243.1	4.0	320.8
Max	492.0	876.2	4.0	1342.8
Median	289.0	488.5	4.0	801.1
Standard Deviation	90.5	127.7	0	208.1

Table 1-3 – WY90-WY99 Runoff Comparison
(25-gage vs. 3-gage precipitation Network)

Water Year	25-gage	3-gage	Ratio
1990	865.60	805.52	1.07
1991	938.40	870.01	1.08
1992	762.60	715.68	1.07
1993	1405.40	1342.85	1.05
1994	636.90	616.05	1.03
1995	732.10	670.63	1.09
1996	793.30	798.54	0.99
1997	801.80	755.94	1.06
1998	827.00	802.40	1.03
1999	851.60	811.71	1.05
Average	861.47	818.93	1.05

Figure 1-1 - Precipitation for WY90

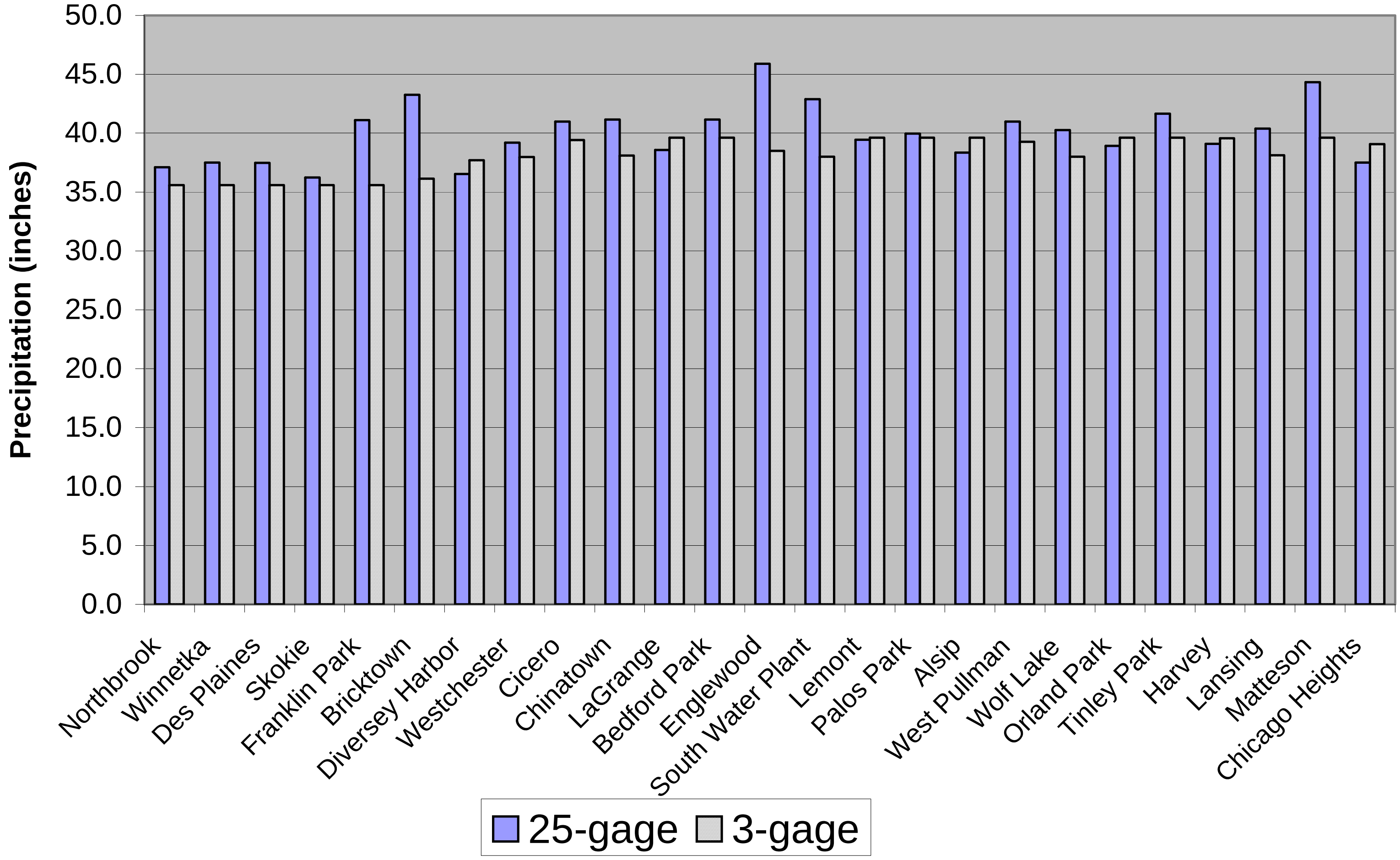


Figure 1-2 - Precipitation for WY91

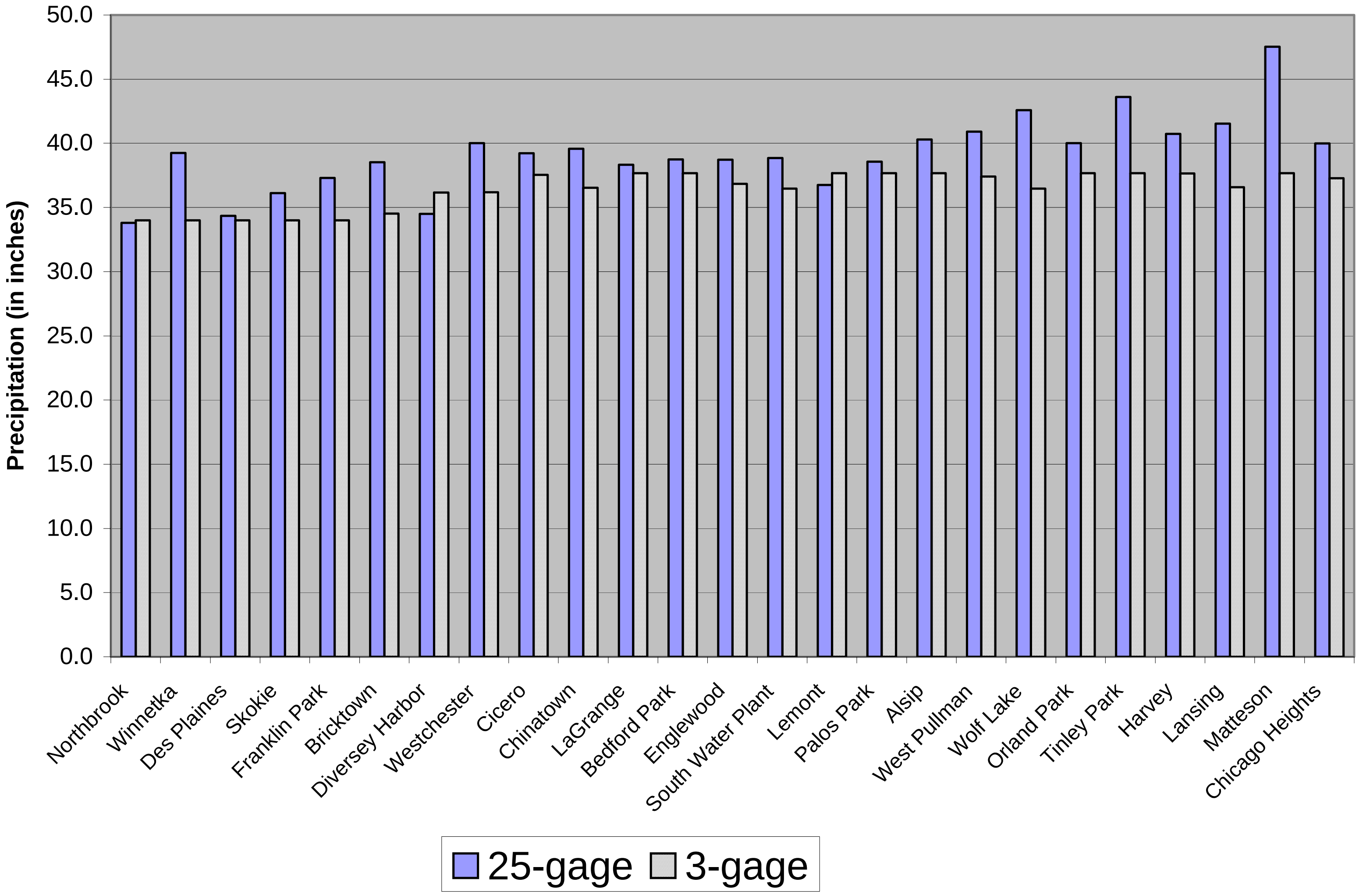


Figure 1-3 - Precipitation for WY92

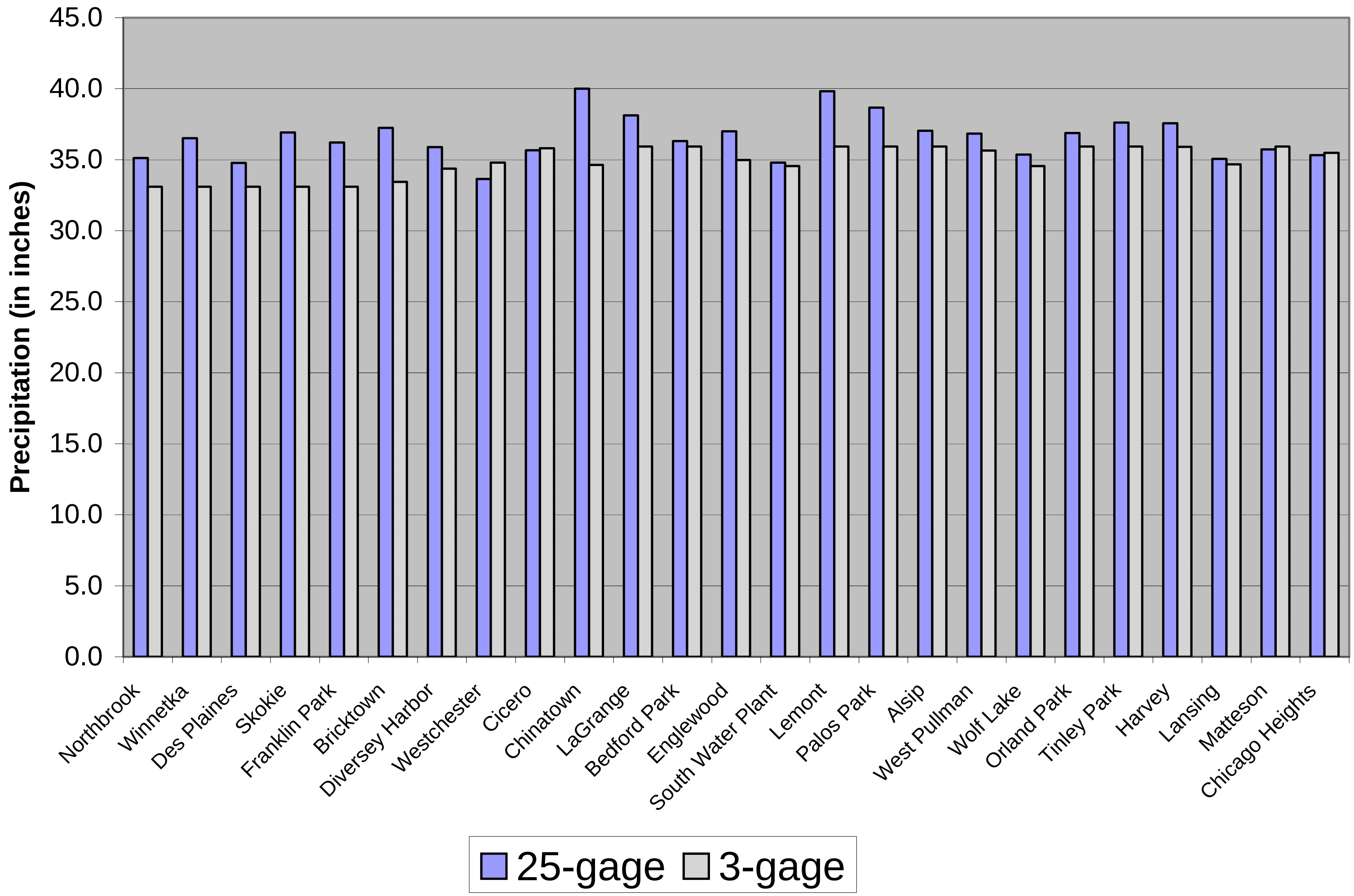


Figure 1-4 - Precipitation for WY93

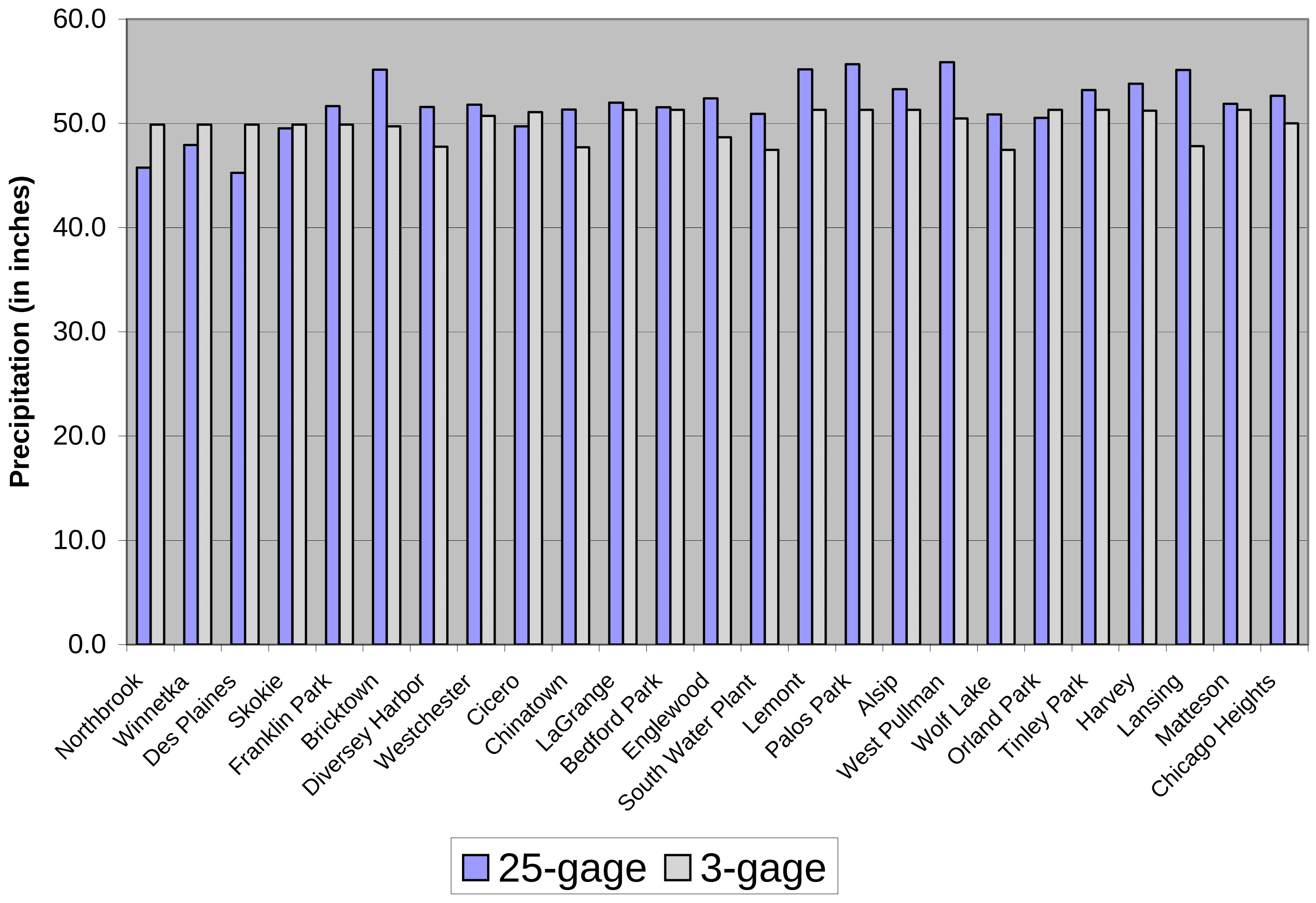


Figure 1-5 - Precipitation for WY94

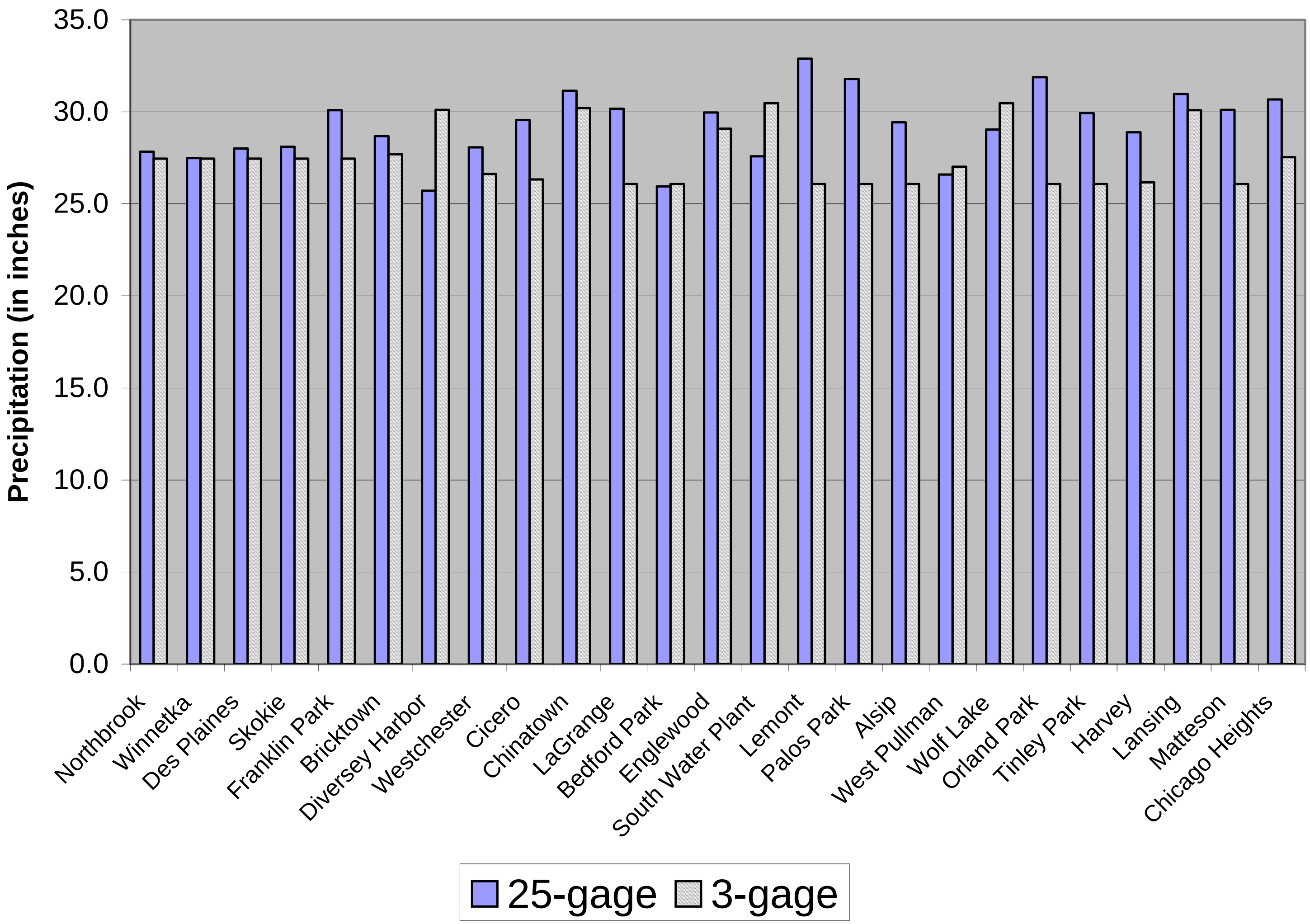
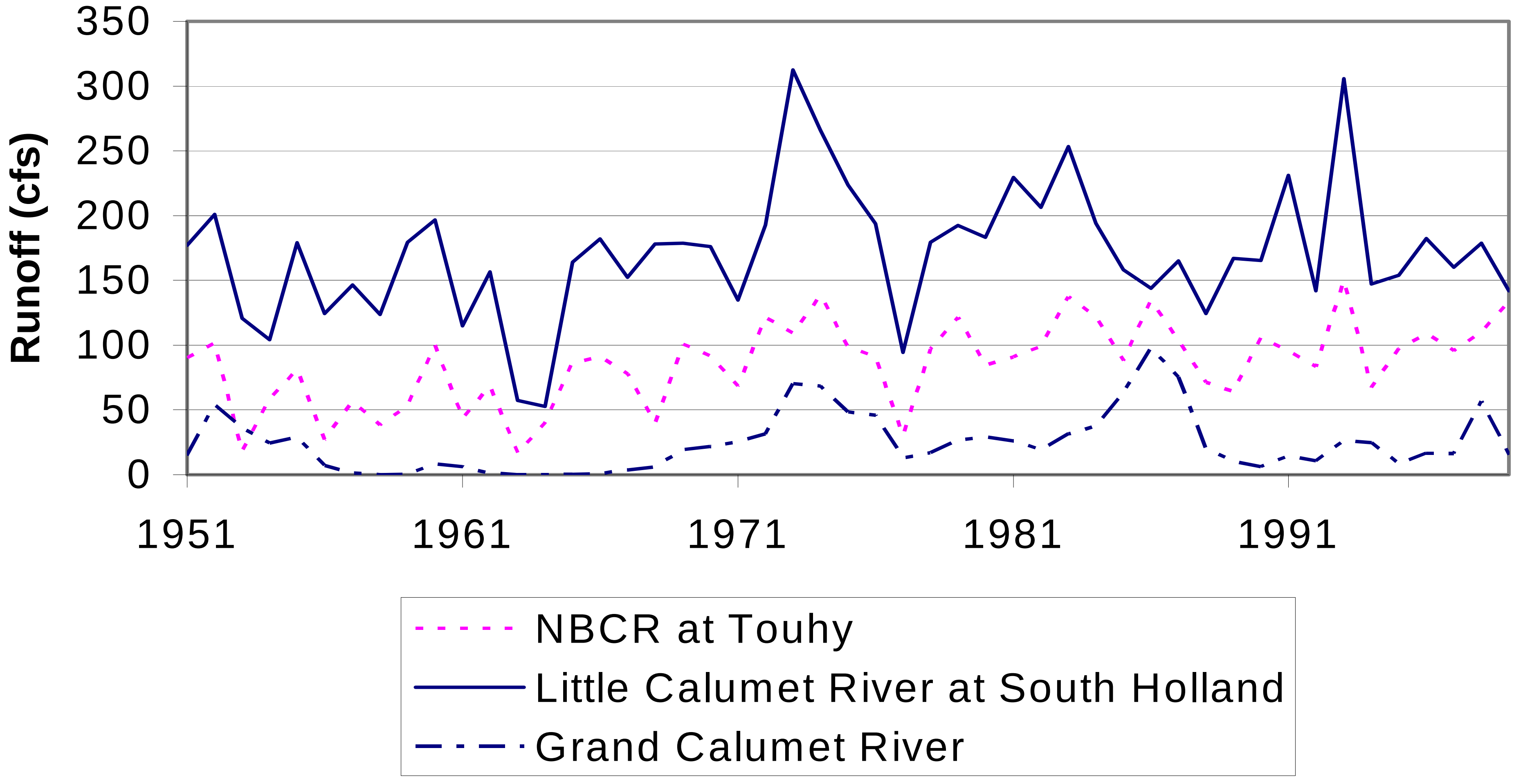


Figure 1-6 - Stream Gage Runoff



Addendum 2

Uncertainty Comparison
Lockport versus Lakefront Measurements

Background

1. The Chicago District has undertaken an uncertainty comparison of accounting methods that can be used to determine the diversion of Lake Michigan waters by the State of Illinois. Extensive technical support was provided and documented in the following references:

a. "Lake Michigan Diversion, Findings of the Fifth Technical Committee for the Review of Diversion Flow Measurements and Accounting Procedures," July 2004.

b. United States Geological Survey (USGS), "Computation of Error Analysis of Discharge for the Lake Michigan Discharge Project in Illinois: 1997-99 Water Years," provisional 2006.

c. Mead & Hunt, "Technical Review of Lake Michigan Withdrawals, Summary of Findings," September 2003.

2. The intent of this evaluation is to provide comparisons of uncertainties in the existing Lockport based Lake Michigan diversion accounting system with uncertainties in potential Lakefront based accounting systems. The accounting methods evaluated include:

- LOCKPORT – The existing accounting system in which flows are measured at the Romeoville AVM, with non-accountable flows deducted and by-passed flows credited to the measured flows.

- LAKEFRONT - A lakefront accounting system that is inclusive of direct diversions and water supply. Runoff and the consumptive use of water supply have been excluded from the sum of components. This system is based on the assumption that a consensus has been reached at some future time to redefine what is accountable.

- FIXED - A lakefront accounting system that includes direct diversions, water supply, and fixed values of runoff and consumptive use. Note, this is the system proposed by the Mediation Committee for computing lakefront accounting.

- ANNUAL - A lakefront accounting system that includes direct diversions, water supply, consumptive use (at 10% of water supply), and the computed annual value of runoff. This analysis is provided as a sensitivity assessment of uncertainties in the FIXED system.

Procedures

3. This uncertainty comparison used seven years of certified flows from WY97 through WY03. All errors in the flows were assumed to be distributed normally, with the means equal to the reported values and standard deviations based on the expected accuracy of the specific component of the accounting system. The method used to aggregate the flows is the first order variance procedure as utilized by the Technical Committee (reference 1a), USGS (reference 1b), and Mead & Hunt (reference 1c). The results are shown in the tables provided with this addendum. Tables 2-1a through 2-7a give results for Lockport accounting (LOCKPORT) for WY97-WY03, tables 2-1b through 2-7b give results for lakefront accounting (LAKEFRONT, FIXED, ANNUAL) and table 2-8 summarizes LOCKPORT and FIXED lakefront flows and uncertainties.

4. With respect to the computation of LOCKPORT uncertainties, a number of considerations should be noted:

- The coefficient of variation (2.1%) for the Romeoville AVM is the flow weighted average of the values given by the USGS.

- The coefficient of variation (1.1%) for all water supply flows is the combined uncertainty computed from first order variances of the water supply uncertainties presented in the Mead & Hunt report. This procedure is acceptable because the flows are relatively independent (with the exception of seasonal effects) and the errors are uncorrelated. Note, the Fifth Technical Committee aggregated the errors using flow weighted averages.

- The coefficient of variation (10%) for all simulated flows is consistent with the methods adopted by the Technical Committee for satisfactorily calibrated models.

5. The LAKEFRONT uncertainties are based on the following assumptions:

- The coefficient of variation (14.0%) for the sum of the Chicago River Controlling Works (CRCW) and O'Brien AVM's is based on the error analysis provided in the USGS report. The flows were summed because they are both strongly dependent on lake level and direct diversions and therefore not independent. The combined uncertainty computed from first order variances is 14.5%, and it was rounded down to account for the dependency of the flows.

- The coefficient of variation (1.1%) for all water supply flows is a conservative estimate based on the water supply reports prepared by Mead & Hunt (see above).

- The coefficient of variation (38.2%) for the Wilmette AVM is the flow weighted average of the values presented in the USGS report.

6. The FIXED values uncertainties are based on the following assumptions:

    - The coefficient of variation (14.0%) for the sum of the Chicago River Controlling Works (CRCW) and O'Brien AVM's, and the coefficient of variation (38.2%) for the Wilmette AVM, are the same as for the LAKEFRONT procedure given above.

    - The recommended fixed runoff value of 800 cfs with a standard deviation of 208 cfs (or 26%) was used in the uncertainty analysis. The standard deviation is from the extended period of record analysis, see Addendum 1.

    - The coefficient of variation (1.1%) for all water supply flows is a conservative estimate based on the water supply reports prepared by Mead & Hunt (see above).

    - The coefficient of variation (30%) for consumptive use is based on a lack of confidence in the value - see the Technical Committee report.

7. The ANNUAL values uncertainties are based on the following assumptions:

    - The aggregate of the uncertainties from the direct diversions through the lakefront structures (as measured by USGS), the water supply less 10% for consumptive use, and the computed runoff.

    - The coefficient of variation (14.0%) for the sum of the Chicago River Controlling Works (CRCW) and O'Brien AVM's, and the coefficient of variation (38.2%) for the Wilmette AVM are the same as for the LAKEFRONT and FIXED procedures given above.

    - The annual computed runoff value with a standard deviation of 10% was used in the uncertainty analysis. This standard deviation is consistent with other simulated values obtained from calibrated models.

    - The coefficient of variation (1.1%) for all water supply flows is a conservative estimate based on the water supply reports prepared by Mead & Hunt (see above).

    - The coefficient of variation (30%) for consumptive use is based on a lack of confidence in the value - see the Technical Committee report.

Conclusions

8. The uncertainty analyses were completed to compare errors in the existing diversion accounting system with errors from a proposed lakefront accounting system. From Tables 2-1 through 2-7 it can be seen that the standard deviations for the existing LOCKPORT system range from 52 cfs to 73 cfs, for the LAKEFRONT system from 39 cfs to 95 cfs, for the lakefront system with FIXED values from 218cfs to 234cfs, and for the lakefront system with ANNUAL values from 88 cfs to 134 cfs.

9. The results from the analyses are furthered summarized in Table 2-8. This table shows that the average annual diversion for WY97-WY03 for the existing system is 2,812 cfs, with an uncertainty of 2.3%; while the average annual diversion for the same period from a lakefront accounting system with fixed values of runoff and consumptive use would be 2,765 cfs, with an uncertainty of 7.9%.

10. With respect to the affect the adoption of a lakefront accounting system would have on the uncertainties in the computation of the diversion accountable to the State of Illinois, the following conclusions can be reached from this analysis of errors:

- If the two systems that account for all of the flow past Lockport are considered (i.e. LOCKPORT and FIXED or ANNUAL), then the present accounting system is clearly more accurate. This is due to three factors:
    - The errors are smaller in computing runoff than in assuming a fixed value.
    - A consumptive use value (and the associated error) doesn't have to be assumed.
    - The Lockport AVM is more accurate than the lakefront AVMs.
- However, if a complete revision to the accounting can be negotiated, and a new total flow at the lakefront adopted (i.e. replacing the 3,200 cfs), then an accounting system equivalent to the LAKEFRONT system would be in effect. In comparison to the LOCKPORT system, this possible system would not significantly affect the level of accuracy.

Table 2-1a Lockport Accounting for Water Year 1997

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Variance from this Component (percent)
Romeoville AVM	3,230.9	2.1	69	4,807.0	91.6
Additions:					
Pumpage not discharged to the CSSC	*234.4*				
Water supply to communities that do not discharged to the CSSC	233.7	1.1	2.6	6.8	0.1
CS overflows from domestic water	0.7	10	0.1	0.0	0.0
Diversions above the gage	2.5	10	0.3	0.1	0.0
Subtractions:					
GW pumpage discharged in the CSSC	*91.9*				
Water supply pumpage	33.2	1.1	0.4	0.1	0.0
Seepage into TARP	58.7	10	5.9	34.5	0.7
Water Supply from Indiana	65.6	10	6.6	43.0	0.8
Runoff from the Des Plaines watershed discharged into the CSSC	189.3	10	18.9	358.3	6.8
Pumpage by Federal facilities	6.8	1.1	0.1	0.0	0.0
Romeoville Total	3,114.2	2.3	72.5	5,249.8	100.0

Table 2-1b Lakefront Accounting for Water Year 1997

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Lakefront Variance from this Component (percent)	Fixed Values Variance from this Component (percent)	Annual Values Variance from this Component (percent)
Direct Diversions							
Columbus AVM	*462.8*						
O'Brien AVM	*191.0*						
Columbus+O'Brien AVMs	653.8	14.0	92	8,377.3	92.1	15.2	46.6
Wilmette AVM	47.7	38.2	18	331.7	3.6	0.6	1.8
Total				*8,708.9*			
Lakefront Diversion							
Domestic Pumpage	1,774	1.1	19.7	389.9	4.3		
Fixed Values with Natural Variability							
Runoff	800	26.0	208.1	43,310.5		78.8	
Domestic Pumpage	1,774	1.1	19.7	389.9		0.7	
Consumptive Use	168	30	50.4	2,540.2		4.6	
Total				*46,240.5*			
Annual Values with Natural Variability							
Runoff	776.6	10	77.7	6,031.1			33.6
Domestic Pumpage	1,774	1.1	19.7	389.9			2.2
Consumptive Use (10%)	177.4	30	53.2	2,832.4			15.8
Total				*9,253.4*			
Total - Lakefront Diversion	2,475	3.9	95.4	9,098.9	100.0		
Total - Fixed Values	3,107	7.5	234.4	54,949.5		100.0	

Total - Annual Values	3,075	4.4	134.0	17,962.3	100.0

Table 2-2a Lockport Accounting for Water Year 1998

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Variance from this Component (percent)
Romeoville AVM	3,119.6	2.1	67	4,481.3	92.8
Additions:					
Pumpage not discharged to the CSSC	*255.0*				
Water supply to communities that do not discharged to the CSSC	254.3	1.1	2.8	8.0	0.2
CS overflows from domestic water	0.7	10	0.1	0.0	0.0
Diversions above the gage	2.4	10	0.2	0.1	0.0
Subtractions:					
GW pumpage discharged in the CSSC	*98.7*				
Water supply pumpage	27.3	1.1	0.3	0.1	0.0
Seepage into TARP	71.4	10	7.1	51.0	1.1
Water Supply from Indiana	59.1	10	5.9	34.9	0.7
Runoff from the Des Plaines watershed discharged into the CSSC	158.7	10	15.9	251.9	5.2
Pumpage by Federal facilities	1.1	1.1	0.0	0.0	0.0
Romeoville Total	3,059.4	2.3	69.5	4,827.3	100.0

Table 2-2b Lakefront Accounting for Water Year 1998

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Lakefront Variance from this Component (percent)	Fixed Values Variance from this Component (percent)	Annual Values Variance from this Component (percent)
Direct Diversions							
Columbus AVM	*359.4*						
O'Brien AVM	*190.8*						
Columbus+O'Brien AVMs	550.1	14.0	77	5,931.7	88.5	11.3	38.0
Wilmette AVM	50.1	38.2	19	366.5	5.5	0.7	2.3
Total				*6,298.2*			
Lakefront Diversion							
Domestic Pumpage	1,801	1.1	20.0	401.7	6.0		
Fixed Values with Natural Variability							
Runoff	800	26.0	208.1	43,310.5		82.4	
Domestic Pumpage	1,801	1.1	20.0	401.7		0.8	
Consumptive Use	168	30	50.4	2,540.2		4.8	
Total				*46,252.4*			
Annual Values with Natural Variability							
Runoff	773.6	10	77.4	5,984.6			38.4
Domestic Pumpage	1,801	1.1	20.0	401.7			2.6
Consumptive Use (10%)	180.1	30	54.0	2,918.2			18.7
Total				*9,304.4*			

Total - Lakefront Diversion	2,401	3.4	81.9	6,699.9	100.0		
Total - Fixed Values	3,033	7.6	229.2	52,550.6		100.0	
Total - Annual Values	2,994	4.2	124.9	15,602.6			100.0

Table 2-3a Lockport Accounting for Water Year 1999

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviatio n (cfs)	Varianc e (cfs^2)	Variance from this Component (percent)
Romeoville AVM	2,944.5	2.1	63	3,992.5	91.0
Additions:					
Pumpage not discharged to the CSSC	*261.7*				
Water supply to communities that do not discharged to the CSSC	260.8	3	7.8	61.2	1.4
CS overflows from domestic water	0.9	10	0.1	0.0	0.0
Diversions above the gage	2.5	10	0.3	0.1	0.0
Subtractions:					
GW pumpage discharged in the CSSC	*117.8*				
Water supply pumpage	26.5	3	0.8	0.6	0.0
Seepage into TARP	91.3	10	9.1	83.4	1.9
Water Supply from Indiana	23.3	10	2.3	5.4	0.1
Runoff from the Des Plaines watershed discharged into the CSSC	156.9	10	15.7	246.2	5.6
Pumpage by Federal facilities	1.2	3	0.0	0.0	0.0
Romeoville Total	2,909.5	2.3	66.3	4,389.4	100.0

Table 2-3b Lakefront Accounting for Water Year 1999

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviatio n (cfs)	Varianc e (cfs^2)	Lakefront Variance from this Component (percent)	Fixed Values Variance from this Component (percent)	Annual Values Variance from this Component (percent)
Direct Diversions							
Columbus AVM	*202.9*						
O'Brien AVM	*169.2*						
Columbus+O'Brien AVMs	372.1	14.0	52	2,714.4	81.8	5.5	22.7
Wilmette AVM	38.0	38.2	15	210.4	6.3	0.4	1.8
Total				*2,924.8*			
Lakefront Diversion							
Domestic Pumpage	1,784	1.1	19.9	394.2	11.9		
Fixed Values with Natural Variability							
Runoff	800	26.0	208.1	43,310.5		88.1	
Domestic Pumpage	1,784	1.1	19.9	394.2		0.8	
Consumptive Use	168	30	50.4	2,540.2		5.2	
Total				*46,244.8*			
Annual Values with Natural Variability							
Runoff	759.3	10	75.9	5,765.4			48.3
Domestic Pumpage	1,784	1.1	19.9	394.2			3.3
Consumptive Use (10%)	178.4	30	53.5	2,863.3			24.0
Total				*9,022.9*			

Total - Lakefront Diversion	2,194	2.6	57.6	3,319.0	100.0		
Total - Fixed Values	2,826	7.8	221.7	49,169.6		100.0	
Total - Annual Values	2,775	3.9	109.3	11,947.7			100.0

Table 2-4a Lockport Accounting for Water Year 2000

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Variance from this Component (percent)
Romeoville AVM	2,562.5	2.1	55	3,023.9	91.8
Additions:					
Pumpage not discharged to the CSSC	*262.8*				
Water supply to communities that do not discharged to the CSSC	262.2	1.1	2.9	8.5	0.3
CS overflows from domestic water	0.6	10	0.1	0.0	0.0
Diversions above the gage	2.5	10	0.3	0.1	0.0
Subtractions:					
GW pumpage discharged in the CSSC	*68.8*				
Water supply pumpage	27.4	1.1	0.3	0.1	0.0
Seepage into TARP	41.4	10	4.1	17.1	0.5
Water Supply from Indiana	18.8	10	1.9	3.5	0.1
Runoff from the Des Plaines watershed discharged into the CSSC	154.9	10	15.5	239.9	7.3
Pumpage by Federal facilities	0.9	1.1	0.0	0.0	0.0
Romeoville Total	2,584.4	2.2	57.4	3,293.2	100.0

Table 2-4b Lakefront Accounting for Water Year 2000

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Lakefront Variance from this Component (percent)	Fixed Values Variance from this Component (percent)	Annual Values Variance from this Component (percent)
Direct Diversions							
Columbus AVM	*152.7*						
O'Brien AVM	*139.7*						
Columbus+O'Brien AVMs	292.4	14.0	41	1,675.4	79.3	3.5	16.8
Wilmette AVM	21.8	38.2	8	69.5	3.3	0.1	0.7
Total				*1,744.9*			
Lakefront Diversion							
Domestic Pumpage	1,724	1.1	19.2	368.1	17.4		
Fixed Values with Natural Variability							
Runoff	800	26.0	208.1	43,310.5		90.3	
Domestic Pumpage	1,724	1.1	19.2	368.1		0.8	
Consumptive Use	168	30	50.4	2,540.2		5.3	
Total				*46,218.7*			
Annual Values with Natural Variability							
Runoff	718.2	10	71.8	5,158.1			51.9
Domestic Pumpage	1,724	1.1	19.2	368.1			3.7
Consumptive Use (10%)	172.4	30	51.7	2,673.9			26.9

Total				*8,200.1*			
Total - Lakefront Diversion	2,038	2.3	46.0	2,113.0	100.0		
Total - Fixed Values	2,670	8.2	219.0	47,963.7		100.0	
Total - Annual Values	2,584	3.9	99.7	9,945.1			100.0

Table 2-5a Lockport Accounting for Water Year 2001

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Variance from this Component (percent)
Romeoville AVM	2,710.4	2.1	58	3,383.0	90.0
Additions:					
Pumpage not discharged to the CSSC	*263.1*				
Water supply to communities that do not discharged to the CSSC	262.8	1.1	2.9	8.6	0.2
CS overflows from domestic water	0.3	10	0.0	0.0	0.0
Diversions above the gage	2.6	10	0.3	0.1	0.0
Subtractions:					
GW pumpage discharged in the CSSC	*75.5*				
Water supply pumpage	24.9	1.1	0.3	0.1	0.0
Seepage into TARP	50.6	10	5.1	25.6	0.7
Water Supply from Indiana	18.2	10	1.8	3.3	0.1
Runoff from the Des Plaines watershed discharged into the CSSC	184.0	10	18.4	338.6	9.0
Pumpage by Federal facilities	0.9	1.1	0.0	0.0	0.0
Romeoville Total	2,697.5	2.3	61.3	3,759.2	100.0

Table 2-5b Lakefront Accounting for Water Year 2001

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Lakefront Variance from this Component (percent)	Fixed Values Variance from this Component (percent)	Annual Values Variance from this Component (percent)
Direct Diversions							
Columbus AVM	*119.1*						
O'Brien AVM	*116.9*						
Columbus+O'Brien AVMs	236.0	14.0	33	1,092.1	71.4	2.3	9.3
Wilmette AVM	22.1	38.2	8	71.0	4.6	0.1	0.6
Total				*1,163.1*			
Lakefront Diversion							
Domestic Pumpage	1,717	1.1	19.1	365.4	23.9		
Fixed Values with Natural Variability							
Runoff	800	26.0	208.1	43,310.5		91.4	
Domestic Pumpage	1,717	1.1	19.1	365.4		0.8	
Consumptive Use	168	30	50.4	2,540.2		5.4	
Total				*46,216.0*			
Annual Values with Natural Variability							
Runoff	871.5	10	87.2	7,595.1			64.5

Domestic Pumpage	1,717	1.1	19.1	365.4			3.1
Consumptive Use (10%)	171.7	30	51.5	2,654.3			22.5
Total				*10,614.8*			
Total - Lakefront Diversion	1,975	2.0	39.1	1,528.5	100.0		
Total - Fixed Values	2,607	8.3	217.7	47,379.1		100.0	
Total - Annual Values	2,675	4.1	108.5	11,777.9			100.0

Table 2-6a Lockport Accounting for Water Year 2002

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Variance from this Component (percent)
Romeoville AVM	2,919.4	2.1	63	3,924.9	90.3
Additions:					
Pumpage not discharged to the CSSC	*277.3*				
Water supply to communities that do not discharged to the CSSC	265.5	1.1	3.0	8.7	0.2
CS overflows from domestic water	11.8	10	1.2	1.4	0.0
Diversions above the gage	1.9	10	0.2	0.0	0.0
Subtractions:					
GW pumpage discharged in the CSSC	*65.2*				
Water supply pumpage	26.8	1.1	0.3	0.1	0.0
Seepage into TARP	38.4	10	3.8	14.7	0.3
Water Supply from Indiana	14.8	10	1.5	2.2	0.1
Runoff from the Des Plaines watershed discharged into the CSSC	198.6	10	19.9	394.4	9.1
Pumpage by Federal facilities	1.0	1.1	0.0	0.0	0.0

Romeoville Total	2,919.0	2.3	65.9	4,346.5	100.0

Table 2-6b Lakefront Accounting for Water Year 2002

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviatio n (cfs)	Varianc e (cfs^2)	Lakefront Variance from this Component (percent)	Fixed Values Variance from this Component (percent)	Annual Values Variance from this Component (percent)
Direct Diversions							
Columbus AVM	*144.5*						
O'Brien AVM	*88.8*						
Columbus+O'Brien AVMs	233.3	14.0	33	1,066.7	65.8	2.2	7.8
Wilmette AVM	37.4	38.2	14	203.8	12.6	0.4	1.5
Total				*1,270.5*			
Lakefront Diversion							
Domestic Pumpage	1,683	1.1	18.7	350.8	21.6		
Fixed Values with Natural Variability							
Runoff	800	26.0	208.1	43,310.5		91.2	
Domestic Pumpage	1,683	1.1	18.7	350.8		0.7	
Consumptive Use	168	30	50.4	2,540.2		5.4	
Total				*46,201.4*			

Annual Values with Natural Variability							
Runoff	970.6	10	97.1	9,420.6			69.3
Domestic Pumpage	1,683	1.1	18.7	350.8			2.6
Consumptive Use (10%)	168.3	30	50.5	2,548.2			18.8
Total				*12,319.7*			
Total - Lakefront Diversion	1,953	2.1	40.3	1,621.3	100.0		
Total - Fixed Values	2,585	8.4	217.9	47,471.9		100.0	
Total - Annual Values	2,756	4.2	116.6	13,590.1			100.0

Table 2-7a Lockport Accounting for Water Year 2003

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Variance from this Component (percent)
Romeoville AVM	2,342.2	2.1	50	2,526.2	93.2
Additions:					
Pumpage not discharged to the CSSC	*262.5*				
Water supply to communities that do not discharged to the CSSC	261.9	1.1	2.9	8.5	0.3
CS overflows from domestic water	0.6	10	0.1	0.0	0.0
Diversions above the gage	1.8	10	0.2	0.0	0.0
Subtractions:					
GW pumpage discharged in the CSSC	*66.6*				
Water supply pumpage	27.3	1.1	0.3	0.1	0.0
Seepage into TARP	39.3	10	3.9	15.4	0.6
Water Supply from Indiana	14.9	10	1.5	2.2	0.1
Runoff from the Des Plaines watershed discharged into the CSSC	126.0	10	12.6	158.8	5.9

Pumpage by Federal facilities	0.7	1.1	0.0	0.0	0.0
Romeoville Total	2,398.3	2.2	52.1	2,711.3	100.0

Table 2-7b Lakefront Accounting for Water Year 2003

Component	Value (cfs)	Coefficient of Variation (percent)	Standard Deviation (cfs)	Variance (cfs^2)	Lakefront Variance from this Component (percent)	Fixed Values Variance from this Component (percent)	Annual Values Variance from this Component (percent)
Direct Diversions							
Columbus AVM	*138.6*						
O'Brien AVM	*95.4*						
Columbus+O'Brien AVMs	234.0	14.0	33	1,073.2	60.4	2.3	13.7
Wilmette AVM	51.3	38.2	20	384.4	21.6	0.8	4.9
Total				*1,457.6*			
Lakefront Diversion							
Domestic Pumpage	1,607	1.1	17.9	319.9	18.0		
Fixed Values with Natural Variability							
Runoff	800	26.0	208.1	43,310.5		90.9	
Domestic Pumpage	1,607	1.1	17.9	319.9		0.7	
Consumptive Use	168	30	50.4	2,540.2		5.3	

Total				*46,170.5*			
Annual Values with Natural Variability							
Runoff	608.7	10	60.9	3,705.2			47.5
Domestic Pumpage	1,607	1.1	17.9	319.9			4.1
Consumptive Use (10%)	160.7	30	48.2	2,323.9			29.8
Total				*6,348.9*			
Total - Lakefront Diversion	1,892	2.2	42.2	1,777.5	100.0		
Total - Fixed Values	2,524	8.6	218.2	47,628.1		100.0	
Total - Annual Values	2,340	3.8	88.4	7,806.5			100.0

Table 2-8 Summary Flows/Uncertainties

	Lockport		**Fixed Lakefront**	
Year	**Flow (cfs)**	**CoV (%)**	**Flow (cfs)**	**CoV (%)**
1997	3,114	2.3	3,107	7.5
1998	3,059	2.3	3,033	7.6
1999	2,909	2.3	2,826	7.8
2000	2,584	2.2	2,670	8.2
2001	2,698	2.3	2,607	8.3
2002	2,919	2.3	2,585	8.4
2003	2,398	2.3	2,524	8.6
Average	**2,812**		**2,765**	
Wt Average		**2.3**		**7.9**