ENGINEERS • SURVEYORS • PLANNERS

Higgins Lake Level Control Structure
2010 Engineering Report

State Identification No.: 2011
NW Quarter of Section 34, T24N, R03W
Gerrish Township, Roscommon County, Michigan
Located on the Cut River
Per Part 307, Act 451 of 1994

Prepared for:

Roscommon County Board of Commissioners
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Roscommon, MI 48653

Prepared By:

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Date of Report: December 14, 2010

Project I.D. Number 118475SG2010

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I. SUMMARY

The Roscommon County Board of Commissioners has commissioned Spicer Group to complete an engineering analysis of the Higgins Lake Level Control Structure (LCS). Spicer Group has prepared this report to summarize the conclusions and recommendations of the engineering analysis. This report should be adopted as a guideline for the County related to needed improvements, maintenance and operational changes for the Higgins Lake LCS.

The scope of services, as requested by the County, that were completed by Spicer Group and summarized in this report include:

- Inspection of the existing LCS as it pertains to water control and development of recommendations to address deficiencies observed.
- Calculation of hydraulic capacity of the LCS and development of recommendations that address deficiencies determined.
- Analyze historical lake level data and, based on the data, develop recommendations regarding operation of the LCS.
- Assess impact of wave action at the LCS and estimate water loss due to wave action.
- Assess impact of flow through the unregulated section of the LCS and estimate water loss through the section.
- Assess water loss from the lake due to evaporation.
- Prepare recommendations related to the operation of the LCS.
- This study does not include an assessment of the suitability of the court established lake level as it relates to lake uses and erosion rates along the lake.

The general conclusion is that the Higgins Lake LCS has adequate hydraulic capacity during large runoff events. However, discharge from the lake is limited by the capacity of the Cut River. Additionally, this study has found that the level of Higgins Lake has averaged below the court established legal lake level during the summer months in typical years. Factors such as water loss due to evaporation, wave action and flow through an unregulated low flow channel contribute to the low summer levels. Losses due to evaporation have been calculated to be the most substantial factor followed by flow through the unregulated span and then by losses due to wave action.

In 2007, the structure was altered to include two additional tilting weir gates (also referred to as "flop gates") totaling 33 feet in length and an unregulated low-flow channel measuring roughly 4.75 feet in width. Through comparison of historical data, the average lake level was found to have been lower in the period following these alterations than the period prior. This does not appear to be attributable to a drought as precipitation in the years immediately following the structure's alteration has been well above average. Therefore, if legal levels are to be maintained annually, water levels must exceed the legal level in the early summer months to conserve an adequate volume to maintain the legal level through the later summer months.

The Higgins Lake structure is in need of minor repairs and modifications, but, overall, the LCS is in good condition. Alterations are needed to improve LCS operation to enable lake levels to be maintained closer to the legal level. Specifically, a restrictor should be placed in the low flow channel to reduce the amount of flow to the Cut River in late summer. Also, scour protection should be added to the low flow channel, improvements should be made to the sheet piling portions and improvements should be made to the stop logs. Also, the staff gage should be replaced.

II. BACKGROUND

This section outlines Spicer Group's understanding of the background and history of the LCS. The following information is based on records and data that were provided by Roscommon County.

The legal lake level in Higgins Lake was set by an order issued in 1982 by a Roscommon Circuit Court, in accordance with Part 307 of Public Act 451 of 1994. This order set the legal level at 1154.11 feet above mean sea level for summer and 1153.61 feet for winter months. In 2009, the legal winter level was temporarily amended (effective through 2013/2014) to be 1153.36 beginning between September 15 and November 1. These orders did not specify the elevation datum. Therefore, Spicer Group has assumed the datum to be NGVD '29. This assumption is corroborated by the 1969 and 1995 reports by Ayres, Lewis, Norris and May Consulting Engineers which refer to the "USGS datum." The USGS datum at Higgins Lake is based on NGVD '29 elevations. Furthermore, it is assumed that the intent of 2009 order was to lower the lake level relative to the NGVD '29 and conversions to the NAVD '88 were not completed.

In accordance with Part 307, Roscommon County is responsible for the operation, maintenance and improvement of the LCS. The purpose of this analysis and report is to provide the County with conclusions and recommendation consistent with their responsibilities pursuant to Part 307.

The Higgins Lake Level Control Structure (LCS) regulates flow leaving Higgins Lake to the Cut River. The structure was originally constructed in 1950 however an original engineering plan set of the structure has not been provided. Significant hydraulic modifications to the structure were made in 2007. Improvements to the LCS which included the addition of two (2) 17-foot tilting weir gates and the creation of a 4.75 foot low-flow channel in the center of the structure. These additions in conjunction with the existing three stop log bays, sheet pile weir, and tilting weir gate provide a total length of approximately 90 feet. An overview drawing of the existing structure is shown in Appendix A and a photograph of the upstream face of the structure is shown in the Inspection section (see Figure 1).

Several previous studies have been done on Higgins Lake including two reports by Ayres, Lewis, Norris, and May, Inc. in 1969 and 1995. These studies assess the hydraulics of the LCS and the capacity of the Cut River. Both studies concluded that under high flow conditions, the capacity of the LCS to dewater the lake is limited by the capacity of the downstream river. The 1995 report found that flow out of Higgins Lake is limited by the capacity of the Cut River when flows exceed 110-120 cfs. Therefore, improvements to the LCS beyond the capacity of the downstream river would not be useful in operating lake levels. With the improvements made to the structure in 2007, the Higgins Lake LCS is capable of conveying more flow than the Cut River can accept. This finding is corroborated by testimony from property owners that during large storm, there is no visible head loss across the structure. Therefore, under these conditions, the Cut River capacity limits the flow from Higgins Lake.

Recently, the Board of Commissioners has received complaints of the lake level being too low. At other times, complaints have been received that the lake level is too high. A committee regarding Higgins Lake was formed. The committee includes participation from the Board of Commissioners. Based on input from the committee and the public, the Board of Commissioners directed to have this evaluation of the Higgins Lake LCS completed.

III. INSPECTION

A surface visual inspection of the Higgins Lake LCS was performed by Spicer Group on July 26, 2010. This inspection focused primarily on those aspects of the structure affecting its capacity and hydraulics and secondarily on structural components of the LCS. The following sections detail the findings of this inspection. For reference, a drawing of the existing LCS is in Appendix A. Specific features of the structure are labeled below in Figure 1. Additional pictures of individual components of the LCS are included in Appendix B.

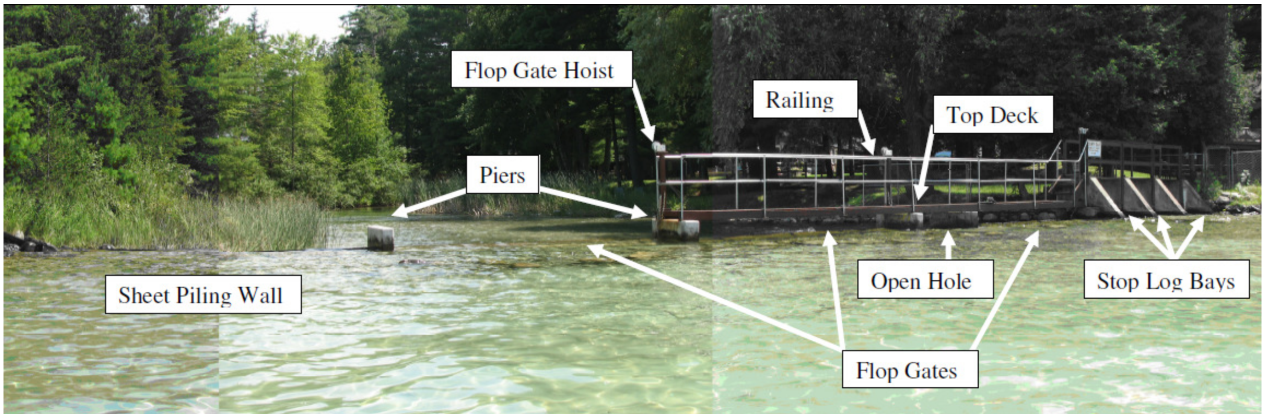


Figure 1: Structural features of the Higgins Lake LCS from the upstream face.

A. Top Deck and Railing

The top deck and railing were found to be in generally good condition with some rust. There is presently no step at the south end of the structure from which to step onto the top deck. The addition of a step here would make access easier.

B. Center Piers

Concrete comprising the center piers appears to be in good condition with only minor areas of spalling.

C. Sheet Piling Walls

The sheet piling cap at north end of structure is in poor condition and uneven. Improvements should be made to this portion of the structure and the cap elevation should be raised slightly.

D. Gates and Operational Features

Gates and gate hoists appear to be in good working order. The stop logs in the three southernmost bays are in poor condition and allow some water to flow between them. Improvements to the stop logs should be made. Also, the staff gage is worn and hard to read. A new staff gage should be installed.

E. Apron Concrete

The apron concrete is in generally good condition. However, no apron exists below the unregulated low flow channel. Scour has begun to occur in this concentrated flow area. The concrete apron should be extended across the open span to resist further erosion.

IV. LAKE LEVEL

In 1982, a court order established the legal level of Higgins Lake at 1154.11 feet for summer months and 1153.61 feet between November 1 and April 15, or ice-out, whichever occurs first. This order was amended in November of 2009 (effective until 2013/2014) to establish the legal winter level at 1153.36 feet with lowering of the lake level beginning each year between September 15 and November 1. Lake level data were obtained from USGS gage #442805084411001. For a period of record from 1986 to 2009, the lake level has averaged 0.1 feet above the legal level to 0.3 feet below the legal level during summer months and 0.15 to 0.4 feet above the legal level during winter months relative to the legal level effective prior to 2009. This comparison is shown below in Figure 2. Note that the legal winter level was amended in 2009 and therefore, the winter lake level trends shown in Figure 2 do not reflect operating procedures currently employed at the Higgins Lake LCS. However, the trends for summer months should be indicative of current procedures as the summer level has not been altered.

Average monthly precipitation data shown in the below graph was collected by the Michigan State University Climatologist's Office using gages located near Houghton and Higgins Lakes for the years of 1971 through 2000 and 1951 through 1978 respectively.

Note that Figure 2 shows the average monthly lake level for the period prior to 2007 and the average level after 2007. As stated in the Hydraulics section, the LCS was modified in 2007 and a low-flow channel was added. It appears from Figure 2 that the average lake level has decreased by 0.1 to 0.4 feet relative with the periods prior to the modification.

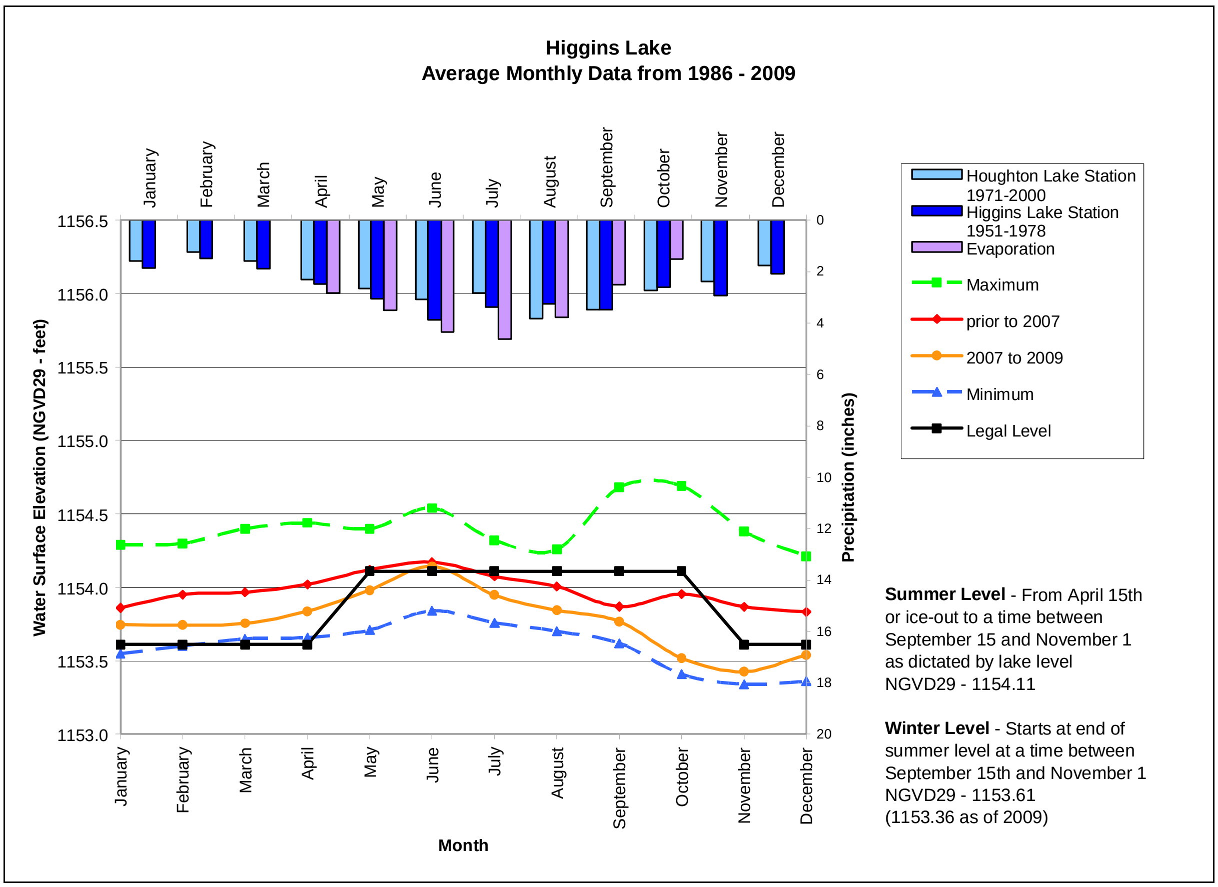


Figure 2: USGS gage data for Higgins Lake related to precipitation and evaporation.

The following sections outline various sources of water loss from Higgins Lake that impact summer water levels.

A. Evaporation

The Michigan State University "Enviro-weather" website provided potential evapo-transpiration (PET) rates for July and August of 2010. The weather station used to obtain these data is located in Arlene, approximately 20-25 miles west of Higgins Lake. Rates of PET were typically between 0.1 and 0.3 inches per day. However, these data included transpiration, which does not occur on open water bodies.

To assess evaporation alone, pan evaporation measurements were used. Monthly averages for pan evaporation were taken from the NOAA Nation Climatic Data Center (NCDC) at Lake City for the years 1967-2008. This site is also approximately 20-25 miles west of Higgins Lake. To convert these pan evaporation rates to lake evaporation rates, pan rates were multiplied by 0.7 as suggested by the "General Guidelines for Calculating a Water Budget" from the Land and Water Management Division of the Michigan DNRE (March 2010). This yielded an average summer evaporation rate of 0.11 inches/day with the highest monthly rate occurring in July (0.15 inches/day). This rate closely matched the summer rate shown in Figure 2 of the aforementioned DNRE document which was 0.11 inches/day (20 inches total evaporation for May-October). The DNRE report is included in Appendix D along with evaporation rates calculated from NCDC data in Appendix E. Monthly evaporation rates are shown on Figure 2 as a hyetograph along the top horizontal axis.

B. Precipitation

Precipitation data were collected from the Michigan State University Climatology website. These data were broken down on a monthly basis for gauging stations at both Higgins Lake and

Houghton Lake as shown in Figure 2. When compared to lake levels in Higgins Lake, months of historically high precipitation have allowed lake levels to rise during summer months.

Data were also collected from the NOAA NCDC on Houghton Lake and at the Roscommon Airport. Though these gages were not specifically on Higgins Lake, they provided a detailed view of changes in precipitation over various time periods. Of particular interest was that in the period of 2007-2010, average rainfall has been over two inches higher than the average of prior years. This appears to indicate that a lack of rainfall has not contributed to lower lake levels observed for the period after the LCS was modified in 2007.

C. Wave Losses

An estimate was created for wave action occurring over the Higgins Lake LCS. This analysis used field observations gathered on 8/31/2010 to estimate wave velocity and frequency. Based on these observations, a design wave speed of 5.0 feet/second was assumed with a frequency of 1.0 waves/second. To obtain an estimate of water loss from wave action, it was assumed that the mean water surface (midpoint of waves) was at the top of the LCS and therefore, the volume of water contained in each wave above this height left the lake. Table 1 gives average daily water loss for waves of varying heights sustained for 24 hour periods. The wave height shown is the distance from crest to trough of each wave.

Table 1: Water lost due to wave action for waves of varying height. Height is given as the total height from crest to trough.

Height (inches)	24-hr Loss (in/day)
4	0.03
6	0.05
9	0.08
12	0.10
18	0.16
24	0.21

D. Water Loss Through Low-Flow Channel

The low-flow channel cut in the dam is approximately 4.75 feet wide with a 4-inch rubber restrictor on one side. From the top of the concrete sill to the top of the pier, the opening is three feet in height however, the distance from the top of the concrete sill to summer legal lake level it is only two feet. Assuming Higgins Lake is at its normal summer level and backwater effects from the Cut River are negligible, it is calculated that 33 cfs flows through the low-flow channel. This flow rate is equivalent to 0.08 in/day draining from the lake assuming there is no inflow to the lake. If 1.0 foot of tailwater is assumed, 28 cfs is allowed to pass through the low-flow channel which equates to a loss of 0.07 in/day, again assuming no inflow to the lake.

E. Summary of Findings

Prior to 2007, the LCS on Higgins Lake did not have a center low-flow channel allowing constant flow to the Cut River and therefore water exited the lake by either evaporation, wave loss over the LCS, or operation of the LCS. By cutting a hole in the center of the structure, the amount of water leaving the LCS during summer months was calculated to increase by roughly 30 percent over the losses due to evaporation and wave action alone. This finding was corroborated through comparison of lake level trends before and after the lake level control structure was modified. Average lake levels in Higgins Lake have decreased by an average of about 0.20 feet in the past three years. Table 2 shows a summary of losses from Higgins Lake based on what are thought to be typical summer conditions.

Table 2: Summary of normal water losses from Higgins Lake.

Water Loss Type	Depth Loss (in/day)
Evaporation	0.10-0.15
Wave Action	0.05
Low-Flow Channel	0.07

Note that this observed decrease in lake level is based on only three years of available data. Upon further data collection, these findings can be reassessed. Although, since precipitation has been above average for the past three years, it seems unlikely that a lack of precipitation has lead to this decrease in water surface.

V. HYDRAULICS

A review and basic assessment of hydraulic calculations for the Higgins Lake LCS was compiled. In performing this review, the first step was to review recently completed studies. A report by Ayres, Lewis, Norris and May, Inc. in May 1995 indicated that the LCS capacity was 55 cfs without a rise in the lake above its legal summer level. The overall capacity of the Cut River was determined to be 110-120 cfs. In general accordance with the 1995 report, modifications were made to the LCS in 2007. The modifications included the addition of two tilting weir (flop) gates totaling 33 feet in length and a low-flow channel roughly 4.75 feet in width. This altered the hydraulic characteristics of the structure such that the hydraulic capacity of the LCS now exceeds the capacity of the Cut River.

A discharge request filed with the Michigan Department of Natural Resources and Environment (MDNRE) on June 9, 2010, reported a 100-year peak flow at the Higgins Lake LCS of 330 cfs. Weir calculations for flow over the structure indicate that with all stop logs removed, gates down, and flow in the Cut River one foot above the invert of the low-flow channel (one foot below legal summer level), 330 cfs can pass through the structure with Higgins Lake at its summer level using the weir and submerged weir equations shown in Appendix C. The center span was modeled as a culvert using Culvertmaster computer software. Despite these calculations, information from the 1995 report coupled with testimony from local residents, indicates that during high flows, there is no noticeable head loss across the LCS. Therefore, flow over the Higgins Lake LCS is ultimately controlled by the downstream Cut River and the LCS provides adequate capacity.

VI. RECOMMENDATIONS

Lake level data on Higgins Lake have shown that the lake has historically been maintained below its legal summer level, notably later in the summer. The following sections outline physical and operational changes that are recommended to help maintain the lake near its legal level and to improve the structural condition of the structure.

A. Structural Improvements

The open span in the center of the Higgins Lake LCS creates high velocities which have caused scour to occur along the downstream toe of the structure therefore, the concrete apron should be extended across the open span to resist further erosion. Also, the sheet pile weir on the north end of the structure has deteriorated and should be improved. Such improvements may include the addition of riprap reinforcement, new sheet piling, and/or a concrete cap on the existing sheeting. When performing such improvements, the sheet piling should be set to an elevation roughly 0.2 feet above the legal summer level. This will assist in attempting to conserve water by holding the elevation at desired times, in excess of the legal level.

Stop logs in the three southernmost bays are in poor condition. However, since the present structure has sufficient capacity to regulate flow using primarily the gates, these logs are seldom needed for lake level regulation. Therefore, rather than replacing the stop logs with new stop logs, a fabricated insert with a top elevation slightly above the legal summer level may be used instead. This would only rarely, if ever, need to be operated.

B. Operational Features

The low-flow channel in the center of the LCS allows constant flow from the structure. Since levels have historically been lower than the legal level, this flow should be reduced during the summer months of July, August and into September. It is recommended that a removable insert be fabricated to enable greater retention of water in the lake. An example of such an insert is shown in Appendix F. If necessary, further control of water leaving the lake could be achieved by mitigating the effects of wave action. This could be done through the installation of a concrete, riprap, or steel break wall. However, the option of controlling wave loss would likely be far more costly than the installation of a restrictor plate and produce less results as more flow discharges via the center span than via wave action.

Operational features which provide accurate and reliable lake level data can facilitate more precise control of lake levels. Spicer Group recommends that the existing staff gage be replaced with one calibrated to the current lake datum. Since Higgins Lake has not been shown to be prone to large, frequent fluctuations in lake level (see Figure 2), the structure does not typically require that the LCS gates be operated regularly to adjust level. Therefore, a remotely transmitting lake level sensor would not be cost effective. Furthermore, the wave action near the Higgins Lake LCS would likely cause any digital sensor to be inaccurate.

C. Operation Guidelines

Note that these guidelines will depend largely on the capacity of the Cut River downstream of the LCS. The 1995 report indicated that the capacity of the Cut River was between 110 and 120 cfs. This is substantially less than the 330 cfs 100-year peak flow rate identified by the MDNRE.

1. Summer Level (NGVD 1154.11)

As stated previously, the level of Higgins Lake has historically been maintained below the court established summer level. Therefore, additional water must be retained in the months of May and June. Recent flow data suggests that roughly 0.4 feet of water is lost between July and September. Therefore, to maintain an average level near the summer legal level, approximately 0.2 feet of water above the legal lake level should be achieved in June.

Around April 15 or ice-out each year, the LCS should be closed to limit flow from the lake. Maintain the LCS to limit flow unless levels rise to more than 0.2 feet above the legal level. If this should occur, operate the LCS to allow the lake to return to a level of 0.2 feet above the legal level.

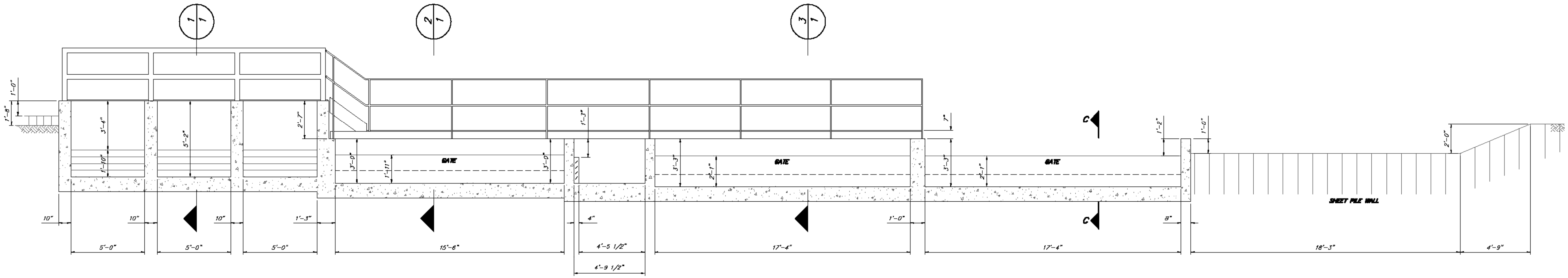
In the months of July, August, and September, the lake level will naturally decrease. Therefore, the LCS should remain closed in an effort to maintain the lake near the court established legal level. Also, the flow restriction device should be installed in the unregulated section of the LCS. Under the court order effective for winter seasons of 2009/2010 through 2013/2014, the LCS should be opened beginning between September 15 and November 1 to draw the lake down to its legal winter level. Due to the limitation of the Cut River to accept flow, it should not be necessary to remove stop logs during this drawdown period.

2. Winter Level (NGVD 1153.36 through winter 2013/2014)

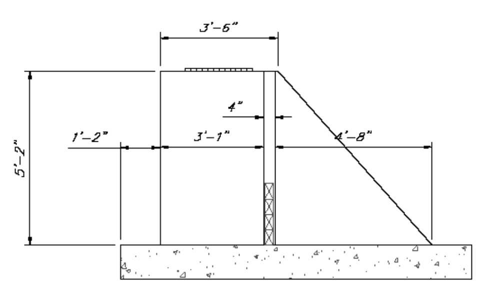
Maintain the LCS in its open position during the winter months. In the event that the lake level drops more than 0.2 feet below the legal level, operate the LCS. On April 15 or ice-out, the LCS gate should be incrementally raised.

APPENDIX A

Plan and Profile of Existing Structure



ELEVATION VIEW
SCALE: NO SCALE
VIEW IS FROM DOWNSTREAM SIDE OF DAM LOOKING UPSTREAM



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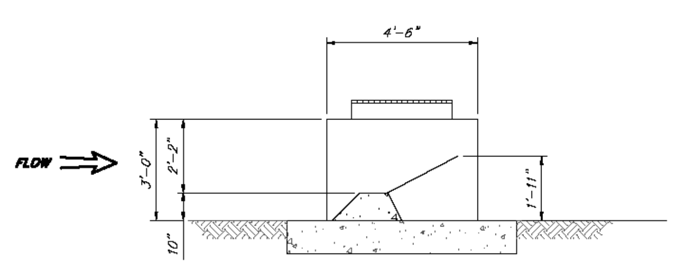

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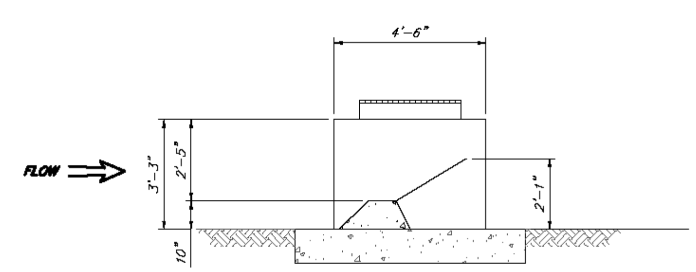

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BY	MARK	REVISIONS	DATE

ROSCOMMON COUNTY
HIGGINS LAKE MICHIGAN

HIGGINS LAKE EXISTING LAKE LEVEL CONTROL STRUCTURE

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APPENDIX B

Inspection Photographs

Upstream face of three stop log bays.

Downstream face of three stop log bays.

Upstream face of southernmost flop gate.

Downstream face of southernmost flop gate.

Unregulated span viewed from upstream face.

Unregulated span viewed from downstream face. Note the rubber restrictor on the left side of the opening.

Upstream face of center flop gate.

Downstream face of center flop gate.

Upstream face of northernmost flop gate.

Downstream face of center (left) and northern flop (right) gates.

Upstream face of the sheet pile weir at the northern end of the Higgins Lake LCS.

Downstream face of the sheet pile weir at the northern end of the Higgins Lake LCS.

North end of decking/railing.

4-inch rubber restrictor in center open span.

Flop gate hoists on downstream railing of Higgins Lake LCS.

Staff gage on southernmost pier of southern stop log bay.

APPENDIX C

Calculation of Hydraulic Capacity

Flow Over Level Control Structure

Weir Equations

Weir equation for free discharge

$$Q_f = C \cdot L \cdot H^{1.5}$$

Where:
Q_f = Discharge (cfs)
C = Discharge coefficient
L = Effective weir length (ft)
H = Head over weir crest (ft)

Calculation of effective weir length (L)

$$L = L' - (2 \cdot K_a \cdot H)$$

Where:
L' = Measured weir length (ft)
K_a = Abutment coefficient

Submerged weir discharge (when tailwater > weir crest elevation)

$$Q_s = Q_f \cdot \left(1 - \left(\frac{H_{ds}}{H_{us}}\right)^{1.5}\right)^{0.385}$$

Where:
Q_s = Discharge over the submerged weir (cfs)
H_{ds} = Head of downstream water surface over weir crest (ft)
H_{us} = Head of upstream water surface over weir crest (ft)

Note: These equations were used for all stop log bays, flop gates, and the sheet pile weir. The center unregulated span was modeled using Culvertmaster software as a culvert.

APPENDIX D

MDNRE Report
"General Guidelines for Calculating a Water Budget"
March 2010

General Guidelines for Calculating a Water Budget

Land and Water Management Division (LWMD)
March 2010

ISSUES:

A water budget is an accounting of all the water that flows into and out of a project area. This area can be a wetland, a lake, or any other point of interest. Development can alter the natural supply of water and severely impact an area, especially if there are nearby ponds or wetlands. A water budget is needed to determine the magnitude of these impacts and to evaluate possible mitigation actions.

DISCUSSION:

A water budget describes the various components of the hydrologic cycle. These components are shown in Figure 1. The water budget typically includes:

- Precipitation (P)
- Evaporation (E)
- Evapotranspiration (ET)
- Surface runoff (SRO)
- Groundwater flow (GF)

The water budget is expressed as an equation relating these components:

$$\Delta S = P - E - ET \pm SRO \pm GF \qquad (1)$$

where ΔS is the change in storage. For example, if the expression on the right-hand side of the equation is positive, storage will increase and the water level in the area of interest will rise. A positive change in storage is often termed a surplus, while a decrease in storage is termed a deficit. The change in storage is usually described with units of inches or feet.

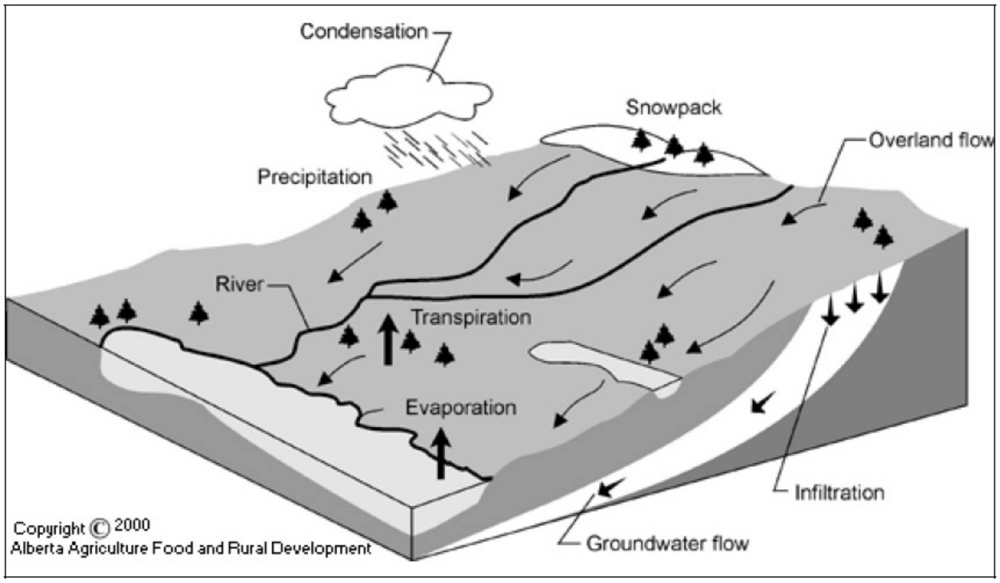


Figure 1 - Components of the hydrologic cycle

In urban areas, the water budget equation may have an additional term that accounts for known point inflows or outflows. These point sources could be withdrawals for industrial uses, outflows from wastewater treatment plants, etc. The amount of water withdrawn or discharged by these point sources can usually be identified from their operating records.

The first three terms of the water budget equation, precipitation, evaporation, and evapotranspiration, are natural processes that are largely unaffected by development. However, changes in land use can significantly affect surface runoff and groundwater flow. For example, commercial development may intercept surface runoff that ran into a wetland and redirect it to a stormwater control basin. This stormwater basin may hold the water until it evaporates or release it to an outlet stream. In either case, the wetland is deprived of the surface runoff that was available before the development. Similarly, water supply wells can permanently lower groundwater levels and change flow directions.

A water budget is calculated for a specified period of time. Permanent projects may be evaluated using daily or monthly data, with the resulting net surplus or deficit is expressed as a seasonal or annual value. Short-term projects, such as lowering a reservoir for maintenance, may be evaluated using hourly or weekly data and express the results on a monthly or seasonal basis.

A water budget should be calculated for a range of conditions. Data from a year with an average amount of precipitation is used to describe long-term effects, but it may be necessary to evaluate 'wet' and 'dry' years for projects with sensitive, natural resources.

The most difficult part of computing the water budget is locating data that allows you to accurately estimate the net surplus or deficit. If the project depends primarily on surface runoff, you can identify years with normal, below normal, and above normal rainfall and use that information to determine the surface runoff under those three climate conditions. Rainfall data are readily available from the National Oceanic and Atmospheric Administration (NOAA) and other agencies. However, if the project area depends on groundwater flow, then you should ideally use groundwater flow data for a range of conditions. But groundwater flow data, if they exist at all, are usually only available for the time period when a permit application is being reviewed.

GUIDANCE/ACTION:

This guidance describes procedures to calculate the components of the water budget equation. Each component is discussed in detail and methods for determining that variable are listed.

This discussion also refers to the permit applicant. When referring to the applicant, we will mean that to also include the applicant's consulting engineers or geologists.

Examples illustrating various situations are also included. Additional discussion and guidance is included in each example.

Precipitation

Precipitation is the primary water input to the hydrologic cycle and is evaluated for all water budget calculations. Precipitation data for a normal year should be used to evaluate the long-term impacts of a project. The precipitation data can be obtained from various NOAA publications. Average monthly and annual data for many locations throughout Michigan are readily available on the Michigan State University Climatology web site at http://climate.geo.msu.edu. Rainfall and climate data are also available from the National climate Data Center at http://www.ncdc.noaa.gov/oa/climate/climatedata.html. Daily rainfall data can also be obtained from LWMD's Hydrologic Studies Program staff.

The wettest or driest years on record do not always provide the most critical analysis. For example, the wettest year may have abundant rainfall in the spring and fall, but have a relatively dry summer. Alternately, what appears to be a normal or drier year may have most of the rainfall concentrated in the summer months. It may be more useful to examine the data and look specifically at the May-Sep rainfall to determine what years to analyze.

The precipitation data should be tabulated by month when evaluating the annual water budget. The analysis is facilitated by setting up the data in an Excel spreadsheet.

Evaporation

Evaporation, as distinguished from evapotranspiration, is the process by which liquid water from an open water surface is converted directly to water vapor. The National Weather Service (NWS) measures evaporation in an evaporation pan that is four feet in diameter, ten inches deep, and elevated approximately six inches above the ground to allow for air circulation around the entire pan. Evaporation data is currently collected at five weather stations across Michigan. Monthly pan evaporation data for the five stations in Michigan can be requested from the LWMD's Hydrologic Studies Program staff.

The evaporation measured in the pan is always greater than what would occur from a lake or pond. The measured evaporation must be multiplied by a coefficient to convert the observed values to an estimated value for lakes and ponds. That coefficient is usually around 0.7. Alternately, the NWS has published an atlas depicting estimated evaporation from a lake surface, on both an annual basis and for the growing season of May-October (1982). Since evaporation is a relatively minor concern during the colder months, the May-October map should be a reasonably good estimate of evaporation losses. This map is shown in Figure 2.

Although the map in Figure 2 may be adequate for most analyses, it may be necessary to distribute this evaporation over each of the six months. Based on recorded data at the evaporation stations in Michigan, the seasonal total can be distributed as follows:

Month	Percent of total May-October evaporation
May	18
June	20
July	23
August	18
September	12
October	9

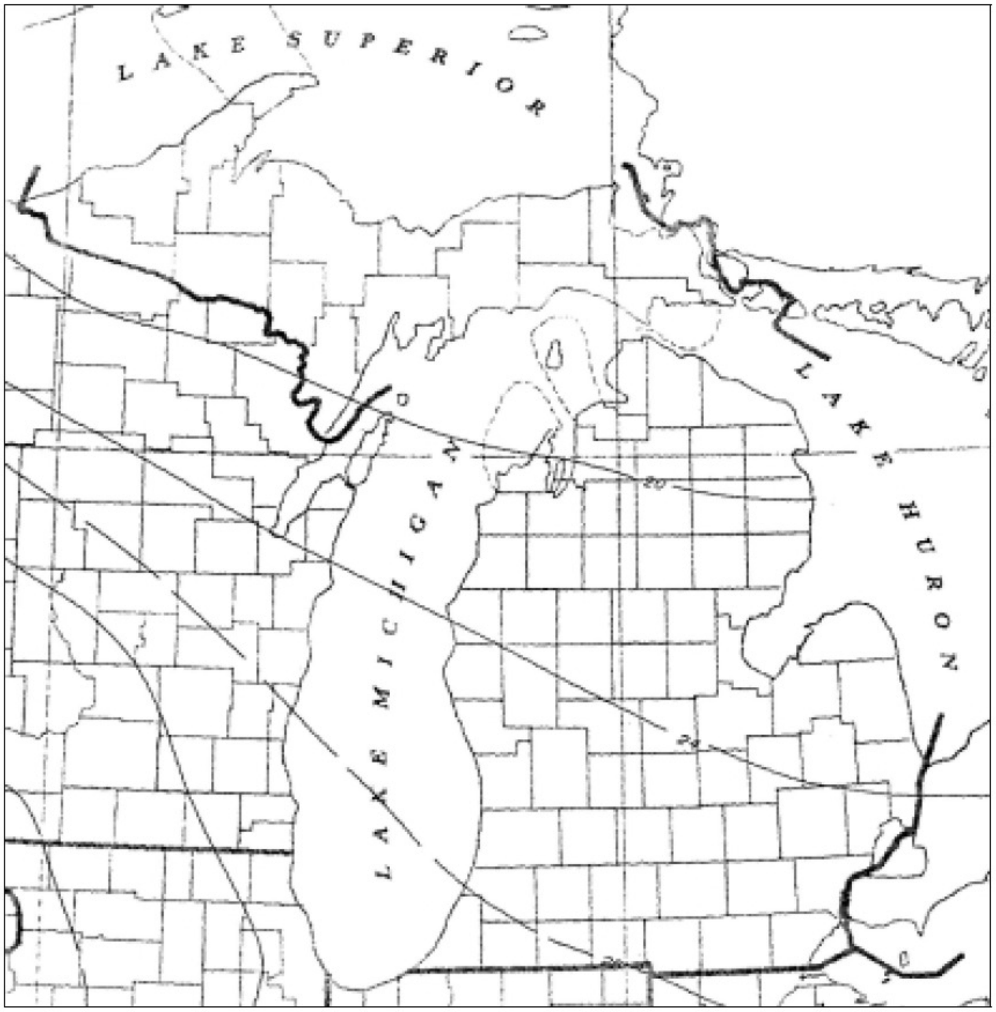


Figure 2 – May - October evaporation (in inches) from an open water surface

Evapotranspiration

Evapotranspiration is similar to evaporation, except that it applies to the combined effect of evaporation from the land surface and transpiration from growing plants. While evaporation is controlled exclusively by climatic factors, evapotranspiration also depends on the type of soil and plants. Evapotranspiration is most often determined by first computing the potential evapotranspiration (PET), which is the maximum amount of water loss if the plants have a constant supply of soil moisture.

Evapotranspiration is computed using the method devised by Thornthwaite and Mather (1957). This method computes the PET, then adjusts it to estimate the actual evapotranspiration. The method is contained in the program EVAP, which is available from the NWS Great Lakes Environmental Research Laboratory (1996).

The only required user input is precipitation, temperature, and latitude. This program is available at (http://www.michigan.gov/documents/deq/lwm-evap_313231_7.zip) or, for LWMD staff, in the S:\HYDRO\EVAP directory.

In some cases, you may need to evaluate evapotranspiration for a specific month. Real-time and historical evapotranspiration data for Michigan can be accessed through the MSU Agricultural Weather Office web site at www.agweather.geo.msu.edu.

In practice, both evaporation and evapotranspiration are tabulated for each month, or the growing season of May-October, then the higher value is used in the water budget. In most cases, evaporation is a more important factor when evaluating an excavated lake, while evapotranspiration may be more significant for wetland projects.

Surface Runoff

Surface runoff is not normally an important component in these calculations unless the pond or wetland is at the bottom of a slope that normally collects and holds surface runoff. This runoff may be needed to keep the wetland from going dry in the summer or at least provide enough water on a seasonal basis. Down-gradient wetlands can also be deprived of water if the surface runoff is diverted to a stormwater basin or collected by storm sewers and rerouted to another discharge point. Please note that these computations are not particularly difficult, but they are tedious and laborious. The surface runoff component should only be determined if the other factors yield an inconclusive answer.

Surface runoff is computed using the runoff curve number method (RCN), which was developed by the Soil Conservation Service in 1954. The combination of a hydrologic soil group and a land use and treatment class is a hydrologic soil-cover complex. Each combination is assigned a RCN, which is an index to its runoff potential. The RCNs for various combinations of soils and land use based on antecedent runoff condition II are shown in Table 1. If the antecedent runoff condition is the ARC I or III criterion, the RCN must be adjusted. The following adjustments show the equivalent RCN for ARC I and III.

$$RCN(I) = \frac{4.2 * RCN(II)}{10 - 0.058 * RCN(II)}$$

$$RCN(III) = \frac{23 * RCN(II)}{10 + 0.13 * RCN(II)}$$

Land use	Treatment or practice	Hydrologic condition	Hydrologic soil group			
			A	B	C	D
Fallow	Straight row		77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and terraced	Poor	66	74	80	82
		Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes or rotation meadow	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraced	Poor	63	73	80	83
		Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	30	59	75	83
		Good	30	35	70	79
Meadow			30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	30	55	70	77
Residential	⅛ acre		77	85	90	92
	¼ acre		61	75	83	87
	1/3 acre		57	72	81	86
	½ acre		54	70	80	85
	1 acre		51	68	79	84
Open spaces (parks, golf courses, cemeteries, etc.)	Good condition: Grass cover > 75% of area		39	61	74	80
	Fair condition: Grass cover 50-75% of area		49	69	79	84
Commercial or business area (85% impervious)			89	92	94	95
Industrial district (72% impervious)			81	88	91	93
Farmsteads			59	74	82	86
Paved areas (roads, drive-ways, parking lots, roofs)			98	98	98	98
Water surfaces (lakes, ponds, reservoirs, etc.)			100	100	100	100
Swamp	At least 1/3 is open water		85	85	85	85
	Vegetated		78	78	78	78

Table 1 - Runoff curve numbers for various land use/soils combinations (ARC-II)

Computing the surface runoff involves the following steps:

- Obtain daily precipitation data from a representative climate station within the same climate area as the wetland to determine the average, driest, and wettest years.
- Compute the average RCN of the area that drains to the wetland. Also compute the RCN for ARC I and III.
- For the computed RCNs, determine the rainfall required before runoff will occur. This is computed by $I_a=0.2*((1000/RCN)-10)$. Do this for the RCN corresponding to all three ARCs.
- Examine the 5-day precipitation before each event in the years you are analyzing to determine the antecedent runoff condition.
- Using the appropriate RCN, compute the daily runoff for each day where the rainfall is great enough to produce runoff.

The daily data can be tabulated monthly and annually, as illustrated in Figure 3.

Although runoff can be grouped into monthly, seasonal, or annual values, the RCN method is only valid for individual events. Therefore, you generally need to apply I_a to each daily rainfall before computing any runoff. In some cases, a single storm may be continuous over two consecutive days and can be analyzed as one event.

Year	Runoff (in)										
	No. of Events	Month									Total runoff for year
		March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	
1990	3			0.54							0.87
1989	3			1.58							1.70
1988	0										0.00
1987	2			0.16							0.24
1986	0										0.00
1985	2							0.75			0.85
1984	1					0.25					0.25
1983	2				0.01						0.02
1982	3					0.25					0.29
1981	3							0.08			0.10
1980	4						0.19				0.24
1979	2						0.13				0.17
1978	2			0.33							0.37
1977	4			0.13							0.36
1976	0										0.00
1975	3				0.17						0.23
1974	1				0.3						0.30
1973	2	0.17									0.22
1972	3						0.14				0.16
1971	1				0.04						0.04
1970	2						0.12				0.13
1969	7				0.32						0.81
1968	2					0.39					0.45
1967	2				0.08						0.12
1966	1					0.05					0.05
1965	4								0.27		0.40
1964	1			0.16							0.16
1963	1				0.28						0.28
1962	0										0.00
1961	0										0.00
1960	1			0.27							0.27
1959	1								0.43		0.43
1958	2					0.05					0.07
1957	1				0.28						0.28
1956	1			0.03							0.03
1955	1		0.21								0.28
1954	3										0.00
1953	0										0.00
1952	0										0.00
1951	4			0.65							0.78

Figure 3 - Example of surface runoff computations

Groundwater flow

Groundwater flow can be an important consideration when evaluating applications for sand and gravel mining. The main concern of a mining operation that excavates a lake or pond is that it exposes the groundwater to the air, which increases losses through evaporation. For this case, the water budget is calculated using groundwater flow, precipitation, and evaporation. Surface runoff is usually a minor consideration for these projects.

In order to determine the groundwater flow component, one needs to have an estimate of the hydraulic conductivity (K) of the soil, or its ability to transmit water. The K can be estimated from well records and is usually determined by the applicant. The total groundwater flow into the project area also requires the cross sectional area and the slope (S_0) of the groundwater head contours. The saturated thickness of the aquifer (B) can usually be determined from well records. The width (W) of the aquifer that flows to the project area requires knowledge of the groundwater head contours. A good estimate of this value is the maximum width of the excavated lake, viewed looking "into" the direction of the groundwater flow. The slope of the groundwater head contours is determined from well records or other recorded water levels and should be calculated by the applicant.

The total groundwater flow (GF) into the excavated lake is then:

$$GF\ (ft^3/day) = K\ (ft/day) * B\ (ft) * W\ (ft) * S_0\ (ft/ft) \qquad (2)$$

This equation is known as Darcy's law. The results are typically converted to units of gallons per day (gpd).

If the change in storage shows there is a net deficit, the effect on water levels in nearby wetlands or ponds can be estimated by assuming this net deficit is equivalent to a pumping well located at the center of the lake. The net deficit in gpd is converted to gallons per minute (gpm) for these computations. A simple well hydraulics analysis based on the Theis equation is used to compute the drawdown. The calculations have been incorporated into an Excel spreadsheet, DRAWDOWN.XLS, located at (http://www.michigan.gov/documents/deq/lwm-evap_313231_7.zip) or, for LWMD staff, in the S:\HYDRO\EVAP directory.

EXAMPLES

Since most of the data are in units of acres, inches, and gallons, the following conversion factors may be useful:

Multiply	By	To obtain
acre-inch/day	27,156	gpd
$feet^3$/day	7.481	gpd
gpd	6.94×10^{-4}	gpm

Example 1

An applicant proposes a project to wet-mine for sand and gravel in southwest Ingham County. The excavation will create a 10-acre lake. A wetland is located 300 feet away from the proposed excavation. Estimate what effect the excavation will have on water levels in the wetland.

Since the project will not involve dewatering, the primary effect on the water budget is that the lake will expose the groundwater to the air, which will result in an increased loss from evaporation. We will assume the evapotranspiration and surface runoff components of the water budget are minor and will not be computed. We can also neglect the groundwater flow term since the natural flow through the area is not being changed. Therefore, equation 1 becomes:

$$\Delta S = P - E$$

Additional data supplied by the applicant show the following:

Saturated thickness of aquifer flowing into the excavation (B) =40 feet
Width of the proposed excavation perpendicular to the flow (W) =1200 feet
Slope of the groundwater table (S_0) =0.008 feet/foot
Hydraulic conductivity (K) =100 feet/day

To determine the evaporation, we use Figure 2 and find that the May through October evaporation in southwest Ingham County is approximately 25 inches. Since there are 184 days from May 1 through October 31, the daily evaporation is **0.136 in/day**.

Normal monthly precipitation data from the MSU Agricultural Weather Office web site at www.agweather.geo.msu.edu show that the May through October rainfall for this portion of Ingham County is approximately 18 inches, or an average of **0.098 inches/day**.

Then, using equation 1, ΔS is **-0.038 inches/day**. The negative sign indicates there is a net deficit. This net deficit of 0.038 inches/day from the 10-acre lake surface equals 0.38 acre-inches/day. These units are converted to **10,300 gpd** or **7 gallons per minute** (gpm).

We can calculate the normal rate of groundwater flow into the lake using equation 2 ($GF=K*B*W*S_0$). Substituting these data into equation 2 gives us a groundwater flow, GF, of 38,400 ft^3/day or **287,000 gpd**.

Based on the normal groundwater inflow to the excavation, the net evaporation deficit represents a seasonal, groundwater flow rate reduction of 2 percent. DRAWDOWN.XLS is used to determine what effect this deficit will have on water levels in the wetland. Data needed for the calculations are the transmissivity (T) and storativity (S) of the aquifer, the distance from the well to the point of interest, the pumping rate, and the number of days the well is pumping.

The transmissivity is equal to the hydraulic conductivity times the saturated thickness of the aquifer (T=K*B). The units are ft^2/day. For our example, T equals 4000 ft^2/day for a K of 100 feet/day.

The storativity should be determined by the applicant. In the absence of field data, the storativity of an unconfined aquifer usually ranges from 0.01 - 0.3, while a confined aquifer ranges from 0.005 - 0.00005. Storativity is dimensionless, so there are no units. For our example, we will assume S = 0.01.

Since we are evaluating the net evaporation deficit from the May through October time period, we will use 184 days for the duration of the pumping. The distance from the edge of the lake to the wetland is 300 feet. The distance from the edge of the lake to the center (where we assume the pumping well would be located) is 700 ft. Thus, the total distance from the well to the wetland is 1000 feet.

The input data used in DRAWDOWN.XLS is

T=4000 ft^2/day (for K=100 feet/day)
S=0.01
Well pumping rate=7 gpm
Time=184 days
Distance=1000 feet
Distance increment=100 feet

Output shows the drawdown at the wetland is 0.14 feet. Therefore, the net effect of the excavation will be to lower the water level in the wetland about 0.1 feet.

Example 2

Given the same data in example 1, assume that the applicant wishes to dewater the excavation to mine the sand and gravel. How will this affect water levels in the wetland?

We already noted that the normal groundwater flow into the excavation will be 287,000 gpd. There will be no evaporation, since there will be no open water surface. However, we still need to account for the precipitation that falls directly into the excavation.

The May through October precipitation of 0.098 inches/day is equal to 27,000 gpd. Thus, the total amount of water that needs to be dewatered is 314,000 gpd or 218 gpm.

DRAWDOWN.XLS is run with these data and shows a drawdown of 4.3 feet. Thus, dewatering the excavation to mine the sand and gravel will lower the wetland water level approximately 4-5 feet.

Example 3

A proposed subdivision plans to collect stormwater runoff and divert it into detention basins. However, diverting this runoff will eliminate the surface runoff that now flows into a wetland on the 'downhill' side of the development. We need to determine how this development will impact the wetland.

The only change to the existing condition is that surface runoff to the wetland is being reduced. We don't really need to evaluate the other terms in the water budget, but can assume that the water supply, including the surface runoff, is adequate or else there wouldn't be a wetland in the first place. So equation 1 reduces to:

$$\Delta S=-SRO$$

The surface runoff term is negative since SRO is being reduced.

Example 4
Assume that the wetland in example 3 didn't exist but the applicant was proposing to create a new wetland as part of a mitigation plan. We need to determine if there will be a sufficient supply of water to maintain the functions of the new wetland.

We will have the same surface runoff deficit as determined in example 3. However, in this case, we will need to evaluate the rest of the terms in equation 1. We would expect that evapotranspiration will exceed precipitation and increase the net deficit, and that groundwater flow will be needed to make up that deficit and make the wetland viable.

Evapotranspiration is computed using program EVAP. The input data includes the latitude of the site (42.5°), the monthly average temperature (°C) and precipitation (mm), and the soil moisture handling capacity (assumed to be 250 mm for this site). The input data and computed evapotranspiration are shown in the following table. Note that program EVAP works with metric units and you need to convert the ET into inches.

Month	Precipitation		Temperature	Evapotranspiration (ET)	
	(mm)	(inch)	(°C)	(mm)	(inch)
May	73.3	2.9	14.17	82.30	3.3
June	92.7	3.7	19.44	117.70	4.7
July	72.8	2.9	21.50	119.80	4.8
August	81.1	3.2	20.56	106.10	4.2
September	69.8	2.8	16.67	75.80	3.0
October	58.0	2.3	10.61	42.75	1.7

The total evapotranspiration is 23.7 inches and the total precipitation over the same time is 22.8 inches.

REFERENCES:

Thornwaite, C.W., and J.R. Mather, Instructions and Tables for Computing Potential Evapotranspiration and the Water Balance, Drexel Institute of Technology, Laboratory of Climatology, Publications in Climatology 10(3), 311 pp. (1957)

Farnsworth, R.K., E.L Peck, and E.S. Thompson, Evaporation Atlas for the Contiguous 48 United States, NOAA Technical Report NWS 33, U.S. Dept. of Commerce, Washington, D.C., 26 pp., 4 maps (1982)

Sellinger, C.E., Computer Program for Estimating Evapotranspiration Using the Thornwaite Method, NOAA Technical Memorandum ERL GLERL-101, U.S. Dept. of Commerce, Washington, D.C., 9 pp. (1996)

APPENDIX E

Estimated Evaporation Rates

Pan Evaporation from NOAA NCDC at Lake City, MI

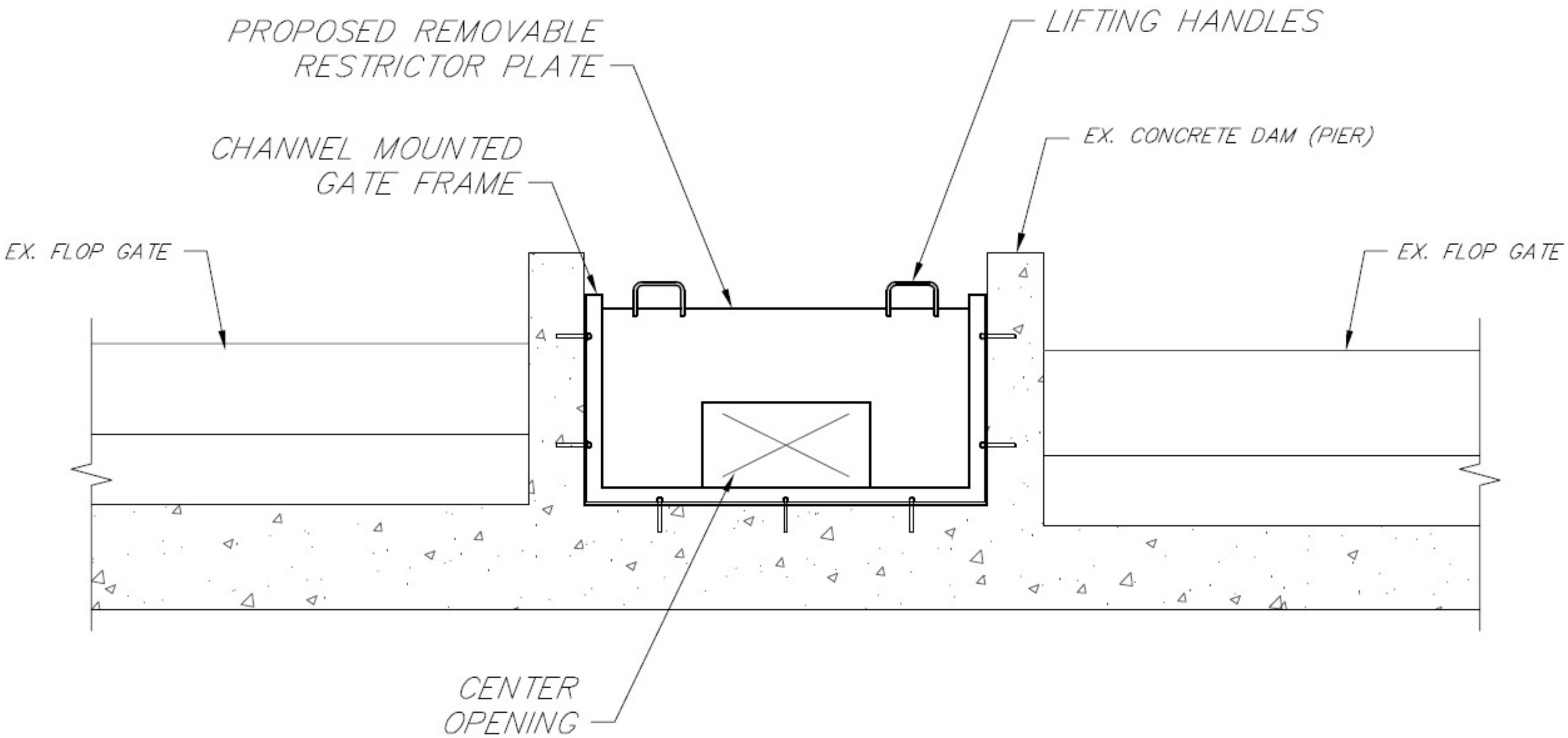
Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1967						546	553			256			1355
1968										195			195
1969						516		630					1146
1970							522	658	313	215			1708
1971						701	684	604	350	200			2539
1972						562	711	448	333	187			2241
1973					455	477	622	534	351	210			2649
1974						459	654	522	333				1968
1975					559	531	704	540	249	259			2842
1976						657	652	586	470				2365
1977						688		528					1216
1978					486	707	670	628	364	217			3072
1979					415	690	629	440	419				2593
1980					498	593	588	486	331	176			2672
1981				404	484	615	633	489	305	142			3072
1982					522	499	607	502	267	261			2658
1983					368	672	749	531	342				2662
1984					398	664	674	533	313	214			2796
1985					564	568	709	460	320	200			2821
1986					499	564	635	490	359	159			2706
1987					606	767	780	514	337				3004
1988					651	903	788	650	400				3392
1989					543	485	705	525	358				2616
1990					477	534	675	429	356	205			2676
1991					501	720	640	563	374				2798
1992					593	636	539	510	337	234			2849
1993					447	551	683	530	280				2491
1994					613	646	613	450	348	228			2898
1995					428	716	623	492	375				2634
1996									348	204			552
1997					424	725	673	416	315				2553
1998					646	604	718	574	451	220			3213
1999					584	670	699	575	435	224			3187
2000					497	612	594		428	280			2411
2001					577	594	775	651	297	196			3090
2002					454	597	818	580	452				2901
2003					402	605	651	590	359				2607
2004					370	608	593	533	474	219			2797
2005					424	697	634	565	424	233			2977
2006					463	619	633	600	309				2624
2007					551	711	668	575	407	233			3145
2008					489	537	637	621	334	225			2843
2009													
MEAN				**404**	**500**	**622**	**661**	**541**	**358**	**215**			**2513**
Inches				**4.04**	**5.00**	**6.22**	**6.61**	**5.41**	**3.58**	**2.15**			**25.13**
Adjusted (x0.7)				**2.83**	**3.50**	**4.35**	**4.63**	**3.79**	**2.51**	**1.51**			**17.59**
in/day				**0.094**	**0.113**	**0.145**	**0.149**	**0.122**	**0.084**	**0.049**			

Summary Average
0.110 in/day

*The factor of 0.7 used to apply pan evaporation rates was used based on *General Guidelines for Calculating a Water Budget* published by the Land and Water Management Division of the Michigan DNRE (March 2010).

APPENDIX F

Concept Design of Flow Restrictor



PROPOSED RESTRICTOR PLATE DETAIL

SCALE: NO SCALE