Streamflow and Lake Level Changes

Model Calibration Report (RS73A)
And
Hydrologic/Hydraulic Modeling Results (RS73B)

PolyMet Mining, Inc.

September 2008

BARR

4700 West 77th Street
Minneapolis, MN 55435-4803
Phone: (952) 832-2600
Fax: (952) 832-2601

Executive Summary

PolyMet Mining, Inc. (PolyMet) is in the process of environmental review for its NorthMet Project, near Babbitt in northern Minnesota. As part of this evaluation, Barr Engineering Co. (Barr) has been retained by PolyMet to evaluate the potential impacts of the proposed NorthMet Mine Site on the Partridge River flows and stream morphology and on the Colby Lake and Whitewater Reservoir water levels. This RS73 report presents the results of this evaluation during various stages of the proposed 20-year period of mining operations. The report consists of two parts: (1) RS73A – Streamflow and Lake Level Changes: Model Calibration Report for the PolyMet NorthMet Mine Site; and (2) RS73B – Streamflow and Lake Level Changes: Hydrologic/Hydraulic Modeling Results for the PolyMet NorthMet Mine Site.

The Mine Site is located at the headwaters of the Partridge River watershed and covers an area of 4.7 square miles, which is larger than the actual area to be impacted by the mine facilities (1.1 square miles at the end of Mine Year 1, and 2.4 square miles by the end of Mine Year 20). The study area for quantitative hydrologic and hydraulic assessment of the Partridge River was defined in the final Scoping Decision Document (SDD) as the catchment area of U.S. Geological Survey (USGS) gaging station #04015475 – Partridge River above Colby Lake at Hoyt Lakes, Minnesota. This catchment area is 103.4 square miles, and it includes the Mine Site. The SDD defines a qualitative assessment of the Colby Lake-Whitewater Reservoir hydrologic system, with a catchment area of 127.8 square miles. The Partridge River flows through Colby Lake. Colby Lake is connected to Whitewater Reservoir on the south through Diversion Works, which were constructed in 1955 to augment the storage capacity to supply make-up water for taconite mining operations that were active until 2000.

RS73A presents the development, calibration and validation of a hydrologic/hydraulic model for the Partridge River study area defined in the SDD. The model was designed to evaluate relative changes on the average, minimum, and maximum flows in the Partridge River that result from the Mine Site development; the model was not designed to predict instantaneous flow values. The U.S. Environmental Protection Agency (USEPA)'s Storm Water Management Model (SWMM), with an interface provided by XP Software (XP-SWMM) was selected for this modeling effort. This is a physically based, unsteady flow model that allows simultaneous hydrologic and hydraulic modeling across the study area. Physical characteristics including topography, watershed area, land use, and soil type were determined for 75 sub-watersheds within the modeled area. Model input parameters including precipitation, evaporation, infiltration, snowmelt, groundwater characteristics, and hydraulic characteristics were derived from literature as well as field data. The hydrologic/hydraulic model provides a good match of observed baseflows during winter, and performs well in capturing the timing and order of magnitude of peak flows associated with spring snowmelt and subsequent summer floods.

RS73B presents the results of using the hydrologic/hydraulic model for the Mine Years 1, 5, 10, 15 and 20 to estimate relative changes with respect to base conditions (i.e., without mining project) in characteristic flow parameters at seven locations along the Partridge River. In addition, one hypothetical high-impact scenario reflecting larger than planned impacted areas was evaluated in RS73B to account for potential uncertainties in the Mine Site development (e.g., conditions that limit the timing of reclaiming of stockpiles, or stockpile footprint change due to unexpected foundation conditions). In all cases, the XP-SWMM simulations corresponded to the period of model validation (1978-1988), which includes dry, average, and wet years comparable to those recommended by the Minnesota Department of Natural Resources (MDNR). The results of these simulations indicate that the impacts on the mean and maximum flows in the Partridge River are less than 10 percent throughout the stages of Mine Site development. These changes are greatest in the vicinity of the Mine Site but decrease to less than 5 percent at the outlet of the study area defined in the SDD. On the other hand, the results of the simulations with XP-SWMM combined with the results of groundwater modeling with MODFLOW indicate that the impacts on the minimum flows in the Partridge River are more pronounced than the predicted impacts for mean and maximum flows. This is driven primarily by the water table drawdown -cone of depression- effect caused by the dewatering of the open pits. Predicted impacts on minimum flows vary from a maximum reduction of 22 percent north of the Mine Site, to less than 7 percent downstream of the confluence of the Partridge River north and south branches, to approximately 3 percent at the outlet of the study area defined in the SDD. These impacts on minimum flows are within the recorded variability in daily flows during the 30-day period of lowest flows in a given water year.

The RS26 report indicated there is some potential for fluvial geomorphic impacts on the Partridge River as a result of increased flows at the Mine Site. RS26 also identifies the most sensitive reach as a channelized reach along the railroad tracks north of the Mine Site. Since the proposed mining project now has a water reuse/recycle strategy, flows in the Partridge River are expected to stay the same or slightly decrease and therefore no impacts are expected on the stream morphology of the Partridge River.

RS73B also presents the results of using the hydrologic/hydraulic model together with the projected range of make-up water demand for NorthMet Process Plant to conduct a water balance assessment of the Colby Lake-Whitewater Reservoir hydrologic system. The main conclusion from the predictive water balance simulations is that the make-up water demand of the Process Plant can be satisfied by the Colby Lake-Whitewater Reservoir hydrologic system during mining operations while staying in compliance with the water appropriation criteria established in the existing water appropriation permit 49-135. The expected maximum withdrawal rate of water to support NorthMet is approximately half the average withdrawal rate of LTV Steel Mining Company (LTVSMC) between 1988 and 1993. The supply of make-up water for NorthMet can be achieved while limiting water level fluctuations in Colby Lake with respect to the current conditions under which there is no withdrawal of water for mining operations. The increase in water level fluctuations at Colby Lake would be less than 0.3 feet, whereas the increase in water level fluctuations in Whitewater Reservoir (between April and October) would be approximately 1.4 feet with the likeliest average annual make-up water demand of 3,500 gallons per minute and average flow conditions. The maximum water level fluctuation of 4.2 feet in Whitewater Reservoir (between April and October) is significantly smaller than the maximum water level fluctuation of 14.3 feet observed during periods of past mining activity.

1.0 Introduction

PolyMet Mining, Inc. (PolyMet) is in the process of environmental review for its NorthMet Project, near Babbitt in northern Minnesota. As part of this evaluation, Barr Engineering Co. (Barr) has been retained by PolyMet to complete a series of support documents required for the Project Description of the proposed project.

This report was divided into a series of two reports to obtain comments on the modeling approach prior to completing the analyses:

- The first, entitled *RS73A – Streamflow and Lake Level Changes: Model Calibration Report for the PolyMet NorthMet Mine Site*, was issued as Draft 02 on August 31, 2007. This report presents the methodology used for developing and calibrating the hydrologic/hydraulic model. This report is being published as Draft 03 in conjunction with Draft 03 – RS73B. There are no changes between Draft 02 – RS73A and Draft 03 – RS73A.

- The second, entitled *RS73B – Streamflow and Lake Level Changes: Hydrologic/Hydraulic Modeling Results for the PolyMet NorthMet Mine Site*, evaluates the impacts of the Mine Site on the Partridge River flows and Colby Lake-Whitewater Reservoir water levels and presents the results of those evaluations. This report was issued on October 15, 2007 as Draft 02. Draft 03 of this report is being published due to modeling updates performed since Draft 02.

1.1 *Preceding Reports*

The scenarios evaluated for this report were based on the following preceding reports:

- The Tailings Basin Water Balance Report (RS13) determined the likely range of make-up water demand for the NorthMet Process Plant.

- The Mine Plan Report (RS18) provided the locations of the pits, stockpiles, and other mine-related features at different stages of the Mine Site development.

- The Mine Waste Water Management Systems Report (RS22) defined the areas that will not be contributing runoff to the Partridge River depending of the stage of Mine Site development. The runoff from these areas are treated as process water; that is, precipitation runoff and groundwater that has contacted disturbed surfaces such as open pits and unreclaimed stockpiles and may not meet water discharge limits. Process water will be collected, treated (if required) and diverted to a different watershed for use in the Process Plant.

- The Mine Surface Water Runoff Systems Report (RS24) provided estimates of the runoff contribution from natural undisturbed areas and from reclaimed stockpiles depending of the stage of Mine Site development. The runoff from these areas is treated as stormwater; that is, precipitation runoff that has not contacted disturbed surfaces and will be routed to the Partridge River following existing drainage patterns as much as possible.

- The Mine Diking/Ditching Effectiveness Study Report (RS25) defined the perimeter diking system around the exterior of the Mine Site and the diking along the rim of the pits, which changed drainage patterns at different stages of the Mine Site development.

- The Mine Site Water Balance Report (RS21) summarized the results of the RS22, RS24, and RS25 evaluations.

- The Partridge River Level 1 Rosgen Geomorphic Survey (RS26) identified the reaches that were potentially sensitive to changes in stream flows.

This report uses data from the RS22, RS24 and RS25 evaluations. Figure 3 of the RS73B report illustrates the interaction between these studies. Readers interested in reviewing all of the Mine Site water management reports may find the following sequence most beneficial for their review: RS73A, RS25, RS22, RS24, RS21, and RS73B. The closure of the Mine Site water management systems is described in RS52 Closure Plan.

1.2 *Objectives*

The objectives of the RS73 study were based on the approach to define cumulative effects described in the final Scoping Decision Document and the Work Plan (attached as Appendix A) that further defined the scope based on discussions with the agencies. The Work Plan has been modified from the January 23, 2006 version that was provided to the agencies to incorporate final comments.

The objectives of the study have changed in some respects from those listed in the Work Plan (these revisions were discussed with John Adams and Mike Liljegren from the MDNR):

- The Work Plan was developed assuming that the mining operations would increase discharges from the Mine Site to the Partridge River, which would require detailed information on the magnitude and timing of the increased flows to define the potential impacts on the receiving watercourse(s). Detailed hydrologic/hydraulic modeling of the Mine Site was proposed. The NorthMet project is now proposing a reuse/recycle strategy, with no discharge to surface waters of the State, though stormwater (runoff from undisturbed or reclaimed portions of the Mine Site) will be routed to the Partridge River following existing drainage patterns. Flows in the Partridge River are expected to stay the same or decrease because a portion of the original Mine Site runoff (i.e., the process water) will be reclaimed for use in the Process Plant. Therefore, detailed hydrologic analysis of the Mine Site water management systems with the hydrologic/hydraulic model developed for the Partridge River watershed is no longer warranted. However, the overall effects that diverting Mine Site process water would have on the Partridge River flows and Colby Lake-Whitewater Reservoir water levels were addressed in this report using the referred hydrologic/hydraulic model.

- The Work Plan was developed assuming that cumulative impacts to the physical character of streams and lakes would occur from increases or decreases in flow or changes in the pattern of flow due to various watershed impacts, such as point discharges (e.g., mine dewatering discharges) and changes in watershed runoff caused by alterations in the percentage distribution of land use (mining, timber harvest, residential development, road construction, etc.) In other words, the hydrologic/hydraulic model of the Partridge River watershed was intended to evaluate not only impacts related to the Mine Site development, but also impacts from other activities in the watershed. In addition to PolyMet's proposed project, two main potential changes in the hydrology of the Partridge River watershed were anticipated. The first envisaged change was the possibility of Northshore Mining Company (Northshore) reinitiating full-scale mining of inactive portions of the Peter Mitchell Pit located north of the Mine Site. Portions of the Peter Mitchell Pit have been allowed to fill with water, but pumping from active mining areas and discharges to the Partridge River currently continue. Further inquiries indicated that the feasibility of Northshore proceeding with dewatering is small. The second envisaged change was the increase in the rate of timber harvesting in the Partridge River watershed. Information about forest stand information available from both the MDNR and the U.S. Forest Service (USFS) Superior National Forest (SNF) indicate that only 5.6 percent of the watershed has been harvested since 1980, and the corresponding annual rate of timber harvesting is not anticipated to increase in the near future, hence the related impacts on the Partridge River flows are not expected to be significant (Verry, 2000).

Therefore, detailed hydrologic analysis of the effects of these potential changes with the hydrologic/hydraulic model developed for the Partridge River watershed is no longer warranted. However, the referred hydrologic/hydraulic model has the capability to evaluate hydrologic impacts of activities that are not related to the Mine Site development.

- The Work Plan indicates that the model may be extended beyond the gaging station record using meteorological data to analyze both wet and dry climatic conditions by creating a synthetic, local streamflow record. However, analysis of the gage data indicate that the 10-year period of flow data at the Partridge River includes a very wet and a very dry year within the period of record, hence the extended record may not provide data that significantly enhances the overall results of the analysis. Therefore, the model was not extended and the results were based on the existing period of record. Statistical analyses were conducted to provide key parameters that define the flow and stream morphology impacts.

The objectives of the RS73A report (issued in August 2007) are to present the development, calibration and validation of a hydrologic/hydraulic model of the existing Partridge River watershed.

The objectives of the RS73B report are to present the results of the hydrologic/hydraulic model, to evaluate expected relative changes on the flows and stream morphology impacts to the Partridge River, and to define the increase in water level fluctuations at Colby Lake and Whitewater Reservoir.

1.3 *Report Outline*

The remainder of this report is organized as follows:

- RS73A Report: This report has not been modified from the August 31, 2007 version.

- RS73B Report: Portions of the RS73 Introduction Section have been repeated and expanded in the RS73B report for readers who do not have the combined RS73 Report available. This report has been updated from the October 14, 2007 version due to modeling updates performed since Draft 02.

- Appendix A: This is the Work Plan that was developed for this report.

Streamflow and Lake Level Changes (RS73A)

Model Calibration Report for the
PolyMet NorthMet Mine Site

PolyMet Mining, Inc.

September 2008

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BARR

4700 West 77th Street
Minneapolis, MN 55435-4803
Phone: (952) 832-2600
Fax: (952) 832-2601

RS73A Streamflow and Lake Level Changes: Model Calibration Report for the PolyMet NorthMet Mine Site

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1.0 Introduction

PolyMet Mining Inc. (PolyMet) is conducting studies to evaluate the technical, environmental and economic feasibility of developing the NorthMet Project, near Babbitt in northern Minnesota. As part of these studies, Barr Engineering Co. (Barr) has been retained by PolyMet to complete a series of support documents required for the Environmental Impact Study (EIS) of the proposed mining project.

This RS73A report is one of the support documents referred to above. This RS73A report was first issued as Draft 01 on November 20, 2006. Draft 02 – RS73A was on August 31, 2007. There are no changes between Draft 02 – RS73A and this report, Draft 03 – RS7A. This report is being published as Draft 03 because it is the first part of a series of two reports. The second report, entitled *RS73B – Streamflow and Lake Level Changes: Hydrologic/Hydraulic Modeling Results for the PolyMet NorthMet Mine Site*, was issued as Draft 02 – RS73B on October 15, 2007. Draft 03 – RS73B has been issued in conjunction with this report.

The purpose of this RS73A report is to present the methodology and results of calibrating and validating a hydrologic/hydraulic model that will be used to assess the cumulative impacts of the Mine Site development and closure on the water quantity in downstream rivers and water bodies (i.e., the Partridge River, Colby Lake and Whitewater Reservoir). The term Mine Site refers primarily to the areas considered for open pits, ore stockpiles, mine waste rock and overburden stockpiles, access roads, and other related facilities and civil works; it does not include the Processing Plant and Tailings Basin, which are located in a different watershed (i.e., the Embarrass River watershed). Once the Mine Site plan and stockpiles design are finalized, the hydrologic/hydraulic model developed here will be used to simulate different water management scenarios, the results of which will be presented in a complementary RS73B report for the proposed 20-year period of mining operations. In addition, the output from this model will serve as input for an evaluation of the probable cumulative impacts of the proposed, combined mining / processing operation on the Colby Lake-Whitewater Reservoir system (to be summarized in RS73B).

PolyMet's Mine Site area covers about 4.7 square miles. It is located at the headwaters of the Partridge River watershed, a tributary of the St. Louis River that is part of the Lake Superior southwestern drainage basin. The study area for quantitative hydrologic and hydraulic assessment of the potential impacts associated with the development and operation of the proposed Mine Site was defined in the final Scoping Decision Document (SDD) as the catchment area of U.S. Geological

Survey (USGS) gaging station #04015475 – Partridge River above Colby Lake at Hoyt Lakes, Minnesota (Figure 1). This catchment area is 103.4 square miles. Hence, the Mine Site area represents 4.5 percent of the study area (Figure 2). The hydrologic/hydraulic model developed here will be used to evaluate expected relative changes on the average, minimum and maximum flows along the Partridge River during different stages of the NorthMet Project. The model is not intended to predict instantaneous flow values, but to provide estimates of overall trends in the flow pattern as the mining project is implemented.

The U.S. Environmental Protection Agency (USEPA)'s Storm Water Management Model (SWMM), with a computerized graphical interface provided by XP Software (XP-SWMM), was chosen as the computer-modeling package for this study. Some of the advantages of using XP-SWMM in this modeling effort are the following:

- It is a physically-based model.
- It allows analyzing both single storm events and continuous long-term periods.
- It allows simultaneous hydrologic and hydraulic modeling across the study area.
- It allows hydrograph input from other areas (such as landlocked areas or other mine areas).
- Its hydrologic module accounts for spatial distribution of rainfall, snowfall and snowmelt, infiltration, groundwater and runoff volumes and flows.
- Its hydraulic module accounts for a full dynamic flow routing, including the analysis of ditch and natural channel networks as well as the evaluation of the effects of fluctuations in ponding areas.
- It has the capability to define the interaction between surface water and groundwater (see schematic in Figure 3), which can be particularly important for large wetland areas and other locations where groundwater has a large impact on the surface water flows.

The model was selected and developed during the initial stages of the water management analyses when point discharges to the Partridge River were planned, thereby increasing flows in the study area. However, PolyMet's proposed plan now avoids and eliminates point discharge of process water (see definition in RS22) from the Mine Site during the proposed 20-year operating life of the NorthMet Project and probably for an extended period during closure. Surface runoff at the Mine Site from undisturbed ground or reclaimed surfaces will continue to be routed to the Partridge River.

During mining operations, there will be a reduction of flows to the Partridge River: surface runoff from the Mine Site that has contacted waste rock, mine dewatering from the active open pits, and all other process water from disturbed surfaces will be routed to the Tailings Basin (located in the Embarrass River watershed) for reuse/recycle. As indicated above, the Mine Site covers about 4.7 square miles, whereas the Partridge River watershed at the USGS gage #04015475, defined as the study area in the SDD of the NorthMet Project, covers 103.4 square miles (the Mine Site area is less than 5 percent the study area). The Mine Site is larger than the actual area to be occupied by the mine facilities (open pits, waste rock and overburden stockpiles, lean ore stockpiles, etc.), which ranges from approximately 1.1 square miles at the end of Mine Year 1 to approximately 2.4 square miles by the end of Mine Year 20 when mining operations are expected to cease. Thus, runoff from less than 3 percent of the (pre-NorthMet Project) Partridge River watershed to the USGS gage #04015475 will be diverted to the Tailings Basin. Therefore, it is expected that the NorthMet Project impacts to the Partridge River flows will be modest. These impacts will be analyzed using the hydrologic/hydraulic model of the Partridge River watershed.

Some time after closure, overflow from the flooded West Pit will occur. Another report, the Closure Plan (RS52) for the NorthMet Project, will describe the proposed water management system during and after Mine Site closure. The hydrologic/hydraulic model may be used for analysis of closure alternatives and computation of potential impacts from open pit overflows. However, only modest changes to the flows in the Partridge River are expected during and after closure.

The hydrologic/hydraulic model implemented here for the study area of 103.4 square miles will provide watershed data for the overall water balance for the Mine Site area (to be summarized in the RS21 report).

This report (RS73A) provides data on the physical characteristics of the watershed, describes model parameters and components, describes other surface water issues in the watershed and presents the results of the model calibration and validation.

2.0 Physical Characteristics of Study Area

2.1 Topography

Three different sources of electronic topographic data were combined to create a digital elevation model (DEM) of the study area:

- Elevation data from the National Elevation Dataset (NED) were used in the majority of the Partridge River watershed. The NED has a 10-meter horizontal resolution and is derived from the USGS quadrangle maps. Mass point elevations were obtained from the center points of each pixel in the dataset.

- Mass points and break lines were provided by PolyMet for the area within the Mine Site, from which 2-foot contours were constructed using triangulation. Also included with these data were ground surface contours at a 2-foot vertical interval, spot elevations, and locations of various topographic features such as streams, roads, and vegetation coverage.

- Digital terrain model data (mass points and break lines) from the Mesaba Project was provided by the Minnesota Department of Natural Resources (MDNR) for the northern portion of the watershed, mostly in and around the Peter Mitchell Pit. The data meets mapping standards for 5-foot contours.

These three datasets were merged using the following priority:

- PolyMet data.

- Mesaba Project data.

- NED data.

None of these data were allowed to overlap. An approximate 10 meter buffer was used between the datasets to allow for transitioning. The NED data was used for all areas where PolyMet data and Mesaba data were not available. The mass points and break lines from these datasets were triangulated using ESRI ArcMap 9.1 3D Analyst extension. A grid with 4 meter spacing was then created from the resulting triangular irregular network (TIN) model.

2.2 Watershed Delineation

The study area corresponds to the Partridge River watershed delineated by the catchment area of USGS gaging station #04015475 – Partridge River above Colby Lake at Hoyt Lakes, Minnesota (see

Appendix A). This watershed includes all tributary streams upstream from Colby Lake, except Wyman Creek. The study area was divided into 75 sub-watersheds. One important criterion used in this division is the definition of smaller sub-watersheds in the areas within or near the Mine Site, as the greatest changes in the hydrology are anticipated to occur immediately downstream of the Mine Site area. Figure 4 shows the names and boundaries of all sub-watersheds, and the general flow directions in the Partridge River watershed (see also Figure 2).

Delineation of the sub-watersheds was based on a preliminary version of the Minnesota Lake Watershed Delineation (Lakeshed) Project delineation of the St. Louis River watershed. These watersheds were delineated by MDNR staff using custom software developed in ESRI ArcView 3.3 and ESRI ArcInfo, on a hydrologically corrected DEM. The Lakeshed Project watersheds were further subdivided at the approximate location of culverts and bridges, confluence of streams, and at the Mine Site. Detailed sub-watersheds within the Mine Site boundary were derived using watershed delineation tools in the ESRI ArcMap 9.1 Spatial Analyst extension and the DEM referred to in Section 2.1. The two sets of sub-watersheds were combined manually using ESRI ArcMap software. Sub-watershed delineations were also visually inspected.

Hydrologic and hydraulic information from this set of sub-watersheds was input to the XP-SWMM model. Initial test runs of the XP-SWMM model suggested that some sub-watersheds needed further subdivision for more accurate hydraulic modeling (more specifically, to reduce continuity errors associated with numerical instabilities). These sub-watersheds were subdivided manually based on visual interpretation of the topographic data and aerial photography available.

2.3 Land Use / Land Cover Data

There is a variety of historic and current land use / land cover information available for northeast Minnesota, which typically includes all or portions of the Partridge River watershed. Because this is a very rural area, much of the information available is land cover data extracted from satellite imagery rather than land use information based on zoning and development plans. Land cover data are available for years 1969, 1992, 1998 and 2001. They all include full spatial coverage of the study area, though with varying resolution. A summary of all the land use / land cover / vegetation data compiled is listed in Table 1.

The 1992 GAP Analysis coverage was selected for this analysis because its data categories are the most informative for hydrologic modeling, including all of the major vegetation types, and distinguishing between upland and lowland forests and brushland (Figure 5). Minnesota GAP vegetation information is presented at four levels, from the most detailed (Level 4) to the most highly aggregated (Level 1). The Level 3 classification was used here to create aggregate classes, which in turn were used to extract information on infiltration characteristics for the hydrologic model. A visual comparison with the U.S. Fish and Wildlife Service (USFWS) National Wetlands Inventory (NWI) dataset showed that the Lowland Forest / Wetland land cover type is very similar to the wetland areas defined in the NWI dataset for the Partridge River watershed (Figure 6). Water bodies and wetland areas cover about 48 percent of the watershed according to the 1992 GAP Analysis coverage, whereas the corresponding cover is only 39 percent according to the NWI dataset (Table 2). Part of the difference is due to an increase from 1978 to 1992 in the open water area in the Peter Mitchell Pit, but part is associated with the area classified as lowland forest in the GAP Analysis coverage but not in the NWI dataset for the central part of the study watershed.

The land cover data was thus used to calculate the impervious area for each sub-watershed defined in the XP-SWMM model, by applying the imperviousness values associated with the different land cover classifications presented in Table 3. The majority of the impervious area in the study area is open water.

In addition to land cover data, there are other data sources that focus on vegetative coverage. There is statewide pre-settlement vegetation data from the MDNR, which is based on the U.S. General Land Office Survey Notes from 1847-1907. There is also forest stand information available from both the MDNR and the U.S. Forest Service (USFS) Superior National Forest (SNF). The data from the SNF also includes the year of origin of the stand, possibly indicating when and where timber harvesting activities were occurring. This data is limited to areas located within the respective forest boundaries. Forest stand information from the USFS provides a means of quantifying the historical acreages of timber harvesting in the SNF during a given time period as well as the approximate likely areas of future harvesting. Figure 7 shows the stand age within the Partridge River watershed. Since only 5.6 percent of the watershed has been harvested since 1980, the impacts of these changes on the Partridge River flows are not expected to be significant (Verry, 2000). Wetlands information is also available from the NWI. More detailed wetlands information is available within the Mine Site area. These wetlands were field delineated, and additional information was collected by Barr in 2005 (see RS44).

2.4 Soils Data

Several different sources of soils data are available for the study area; the sources and their coverage are summarized in Table 4. Digital Soil Survey Geographic Dataset (SSURGO) soils data, considered to be the best available soils information created on a countywide basis, are not available for the county in which the study watershed is located (that is, St. Louis County, Minnesota). Hard copies of a recently completed county survey have been received for the area around the Mine Site in the Embarrass portion of St. Louis County: this data was not electronic and was therefore used only for confirmation of other data. There is detailed electronic soil information available from the USFS. This data presents detailed Ecological Land Type (ELT) information, which provides a very comprehensive description of soil characteristics including whether hydric soils are present or not. Figure 8 shows the extent of the available soils data within the context of the Mine Site area. The detailed soils data is limited to areas within the SNF, therefore it is incomplete for the purposes of this hydrologic model.

For areas not within the proposed Mine Site or the SNF, detailed soils information are not available (see Figure 9). There are two sources of very general soils information that have statewide coverage. These sources are the State Soil Geographic Database (STATSGO) soils data and the Minnesota Soils Atlas. Though providing good reference information at a more regional scale, it is recommended these soils data sources are not used for decision-making at the county level. Since none of the soil data sources provide full coverage of the study area or sufficient detail about soils in each sub-watershed, none of these data sources were used to define specific soils in each sub-watershed. Instead, the sub-watersheds were divided into upland and lowland soil types based on the NWI coverage for each sub-watershed. Characteristics of the upland and lowland soil types were based on the detailed information from the USFS and county survey.

3.0 Model Parameters and Components

3.1 Computational Procedures Used by XP-SWMM

XP-SWMM is an unsteady flow model that allows simultaneous modeling across a study watershed. Figure 3 is a schematic of the connection between the different components included in the computational modules of XP-SWMM (i.e., the “Runoff” and the “Hydraulic” modules).

The study area was divided into 75 sub-watersheds (see Section 2.2). Information about the hydrologic and hydraulic characteristics of these sub-watersheds together with precipitation and weather data was used to generate runoff at each computational time step of the simulation period. The USEPA non-linear runoff method was used to compute runoff; the overland flow hydrographs in this deterministic hydrologic model are generated by a routing procedure that makes use of the Manning’s equation and a lumped water continuity equation. The runoff produced in the sub-watersheds is then routed through the existing ditch and natural channel networks by solving the Saint-Venant shallow water equations. Flow velocities and water depths at different locations along the flow network are obtained at each computational time step of the simulation period. The model accounts for flows in ponding areas as well as the effects of backwater conditions due to hydraulic structures or confluence of streams.

It is worth noting here that many of the model parameters presented in this Section 3.1 were used for model calibration. More specifically, the ranges listed in Section 3.0 of this RS73A report were evaluated during the calibration process. Final values defined during calibration are described in Section 5.2.

3.2 Global and Local Parameters

Input data can be defined in XP-SWMM globally (uniformly across all the study area), or separately for each sub-watershed. Examples of parameters defined globally in the hydrologic model of the Partridge River watershed include groundwater, evapotranspiration, snowmelt and depression storage. On the other hand, geometric and physical parameters, precipitation and infiltration are defined locally for each sub-watershed.

3.3 Geometric and Physical Parameters

Each sub-watershed is defined with the following basic geometric and physical input parameters that control the timing of runoff contributed from the sub-watersheds to the stream channels:

- Catchment areas were calculated using GIS software, as described in Section 2.2.

- Impervious percentages were calculated as area-weighted averages, based on the land cover type distribution within each sub-watershed and the imperviousness reference values indicated in Table 3.

- Average watershed slopes were calculated using ESRI ArcView scripts developed by Barr.

- Watershed width is defined as twice the length of the main drainage channel, with adjustments made for catchments that are skewed (that is, when the areas on both sides of the main drainage channel are not equal). An ESRI ArcView script was used to calculate this parameter for each sub-watershed, based on a digitized average flowpath.

- The surface roughness for overland flow is defined using a Manning's "n" coefficient. It has been set as 0.02 for impervious surfaces and 0.50 for pervious surfaces, based on information gathered from several sources (Haan and Johnson, 1982; Cronshey, 1986; Ponce, 1989; US Corps of Engineers, 1998).

XP-SWMM uses slope, watershed width and surface roughness to estimate the shape of the hydrograph produced by each sub-watershed.

3.4 Precipitation

The mean annual precipitation for the study area is 29.2 inches for the period October 1, 1971 through September 30, 2001, which corresponds to the definition of the climate normal by the Climate Prediction Center of the National Weather Service (NWS). Approximately 75 percent of the annual precipitation occurs between May and October, whereas approximately 9 percent of the annual precipitation corresponds to the water equivalent of snowfall between December and February.

The two weather stations closest to the study area are located within the cities of Hoyt Lakes and Babbitt. The periods of precipitation record in these two stations are sufficiently long to characterize wet and dry periods in the study area. It may be especially critical for this large watershed (103.4 square miles), however, to work with at least three stations that surround the watershed. There is evidence that the spatial gradient of precipitation during storm events can be significant in this study watershed. State-wide, precipitation increases from the extreme northwest corner of the state to the southeast, with a secondary maximum in the northeast region. The study area is located in this northeast region of Minnesota that has a secondary maximum. Therefore, additional precipitation data from other nearby weather stations is required to adequately represent the spatial variability of precipitation.

Table 5 summarizes the information available on daily precipitation at Minnesota High Density Network (MN HIDEN) and NWS stations located within St. Louis, Itasca, Aitkin, Carlton and Lake Counties. Figure 10 shows the location of these weather stations. The sixteen stations highlighted in Table 5 were selected for further evaluation, as these stations are located within a radius of approximately 30 miles from the Mine Site. This preliminary selection was based on a) the typical aerial coverage of the weather fronts in the Midwest (Huff and Angel, 1992), and b) the distance required to include stations located on all sides of the study area, so that the anticipated spatial gradient of precipitation can be better characterized.

Figure 11 is a plot of the cumulative precipitation at seven out of ten stations which have complete records during the water year 1984-1985, which is the period that has been selected for model calibration (see Section 5.1). Although the agreement is not excellent, the seasonal trends and the total precipitation for this water year are comparable (the maximum total precipitation value at Tower 3S is 22 percent greater than the minimum value at Winton Power Plant). Overall, the gages west of the study area had the highest total precipitation values. When the precipitation data is examined on a daily basis, important differences have been observed for the precipitation values recorded at these seven stations. Figure 12 illustrates these differences over a 5-week period during the summer of 1985. Therefore, accounting for the spatial variability of precipitation could be critical for hydrological simulations of runoff events within the Partridge River watershed.

As indicated above, daily precipitation data were compiled from stations located within a radius of approximately 30 miles from the Mine Site. Negative values were assigned to missing data; negative values were ignored in the subsequent analysis. An ESRI ArcView 3.3 script was developed to create a SURFER data file for each day of record, and a batch script developed in SURFER was used to estimate precipitation in 500 x 500 meter cell grids. The Kriging (geostatistics) method was used to perform the spatial interpolation. The resulting grids were converted to ESRI ASCII grids, which were used in turn by another ESRI ArcView 3.3 script -“Summarize by Zones” command in Spatial Analyst- to compute watershed-wide average precipitation values for each of the sub-watersheds (or group of sub-watersheds) defined in the XP-SWMM model. The geostatistical approach was also adopted by Adams et al. (2004) to conduct the hydrological assessment of long-term watershed reclamation plans for the abandoned open pits and tailing impoundments of Cliffs-Erie Mining Company.

Figure 13 through Figure 15 show three examples of the Kriging interpolation results for storms in which the spatial gradient of precipitation was very pronounced. It is interesting to note that in Figures 13 and 14 the gradient is increasing from southwest to northeast, whereas in Figure 15 the gradient is increasing from west to east. The total daily precipitation varies by up to 1.04 inches for the storm event presented in Figure 13. It is also apparent in these figures, as well as in the ones prepared for the five largest annual storm events that occurred during the period 1978-1988, that the sub-watersheds can be grouped in nine geographic regions (Figure 16) that are relatively homogeneous in their precipitation patterns. The Kriging method was applied to produce time series of daily precipitation for each of these nine groups as well as for the individual 75 sub-watersheds.

The “Runoff” module of the XP-SWMM model computes the hydrologic balance in each sub-watershed, and typically uses a computational time step that is less than a day. If daily precipitation data were input to the model, these daily values would be evenly split throughout the day by XP-SWMM. This is not a sound procedure when storm events occur; the standard even split would result in smaller than anticipated rainfall intensities during the peak of the storm, which would result in an artificial increase of infiltration and consequently an artificial reduction of runoff volumes. The second-quartile distribution proposed by Huff and Angel (1992) was used to convert the spatially distributed precipitation from daily into hourly values; this procedure was automated using a MS Visual Basic for Applications script run whenever the daily precipitation was greater than 0.10 inches. These time series of hourly precipitation were input for each of the sub-watersheds included in the XP-SWMM model of the study area.

3.5 Evaporation

Evaporation plays several vital roles in continuous simulations. It is important in estimating the amount of depression storage available prior to a given storm event and therefore ultimately plays a key role in sub-watershed runoff estimates. Secondly, evaporation impacts the surface water elevations of a pond and the volume of water in ponds. This in turn affects the volume available to store runoff prior to conveying any excess to the next downstream basin. Finally, as opposed to open water evaporation, evapotranspiration is one of the components determining the amount of water storage in the sub-watershed soils, the corresponding groundwater recharge to the channel network, and the amount of precipitation that can infiltrate into the ground: evapotranspiration is analyzed within the groundwater subroutine (see Section 3.8). This can be particularly important in an area with significant wetlands coverage, as is the case of the Partridge River watershed.

XP-SWMM does not contain a module to calculate evaporation from climate input; instead, it requires monthly evaporation estimates as input. These monthly rates are subtracted from the rainfall intensities for open water and are also used to compute the available depression storage.

Mean monthly and annual evaporation for northern Minnesota are available from several sources. Climate average annual evaporation was measured by Meyer (1942) as approximately 22 inches. Mean annual evaporation was estimated from stream gage measurements as 18 inches in the Copper-Nickel study (Siegel and Ericson, 1980), whereas for the region in which the study area is located Baker et al. (1979) suggests a mean annual potential evapotranspiration (PET) of 21.8 inches and a mean annual actual evapotranspiration (ET) of 16 inches. Pan evaporation measurements from Hoyt Lakes for the period 1966-1983 give no evaporation in the winter months, with a yearly total evaporation of 18.7 inches to 20.8 inches depending on whether a pan correction factor of 0.70 or 0.78 is used; previous studies conducted by Barr suggest 0.78 is a more reasonable value for this region. Average monthly pan evaporation data from Hoyt Lakes with a pan correction factor of 0.78 were thus used in the Partridge River model. No evaporation was considered during the winter, but a sublimation factor aimed to slightly reduce the depth of the snowpack was included in the model calibration (see Section 3.7).

3.6 Depression Storage and Infiltration

Before any runoff is generated, an initial volume of rainfall is removed due to initial water abstraction resulting from interception, surface ponding and surface wetting. This is represented in XP-SWMM by the “Depression Storage” parameter. Depression storage was used as a calibration parameter to adjust the volume of runoff for the entire study area. XP-SWMM recommends a depression storage value of 0.10 inches on turf-covered pervious surfaces; this value appears reasonable for our study area. Furthermore, less than 10 percent of the annual precipitation occurs on days with precipitation less than 0.10 inches. Values ranging from 0.05 to 0.15 inches were considered during the calibration. Depression storage from pervious surfaces is replenished during dry periods by infiltration and evaporation. Since the only impervious surfaces in our study area are water surfaces, no depression storage was included for impervious surfaces.

The Horton infiltration method was used to simulate the reduction of the infiltration capacity over the course of a storm. It is given by an exponential decay equation in which F_p = infiltration rate into soil is expressed in terms of F_c = minimum or final value of F_p; F_0 = maximum or initial value of F_p; k = decay coefficient; and t = time from beginning of storm. XP-SWMM uses an integrated form of this equation, so that the infiltration rate only decreases during time-steps when it is actually raining or when surface water is ponded. The infiltration capacity of the soil is restored during dry periods using a Horton type exponential equation with the rate constant equal to 1 percent of the decay coefficient.

Infiltration parameters were estimated for each sub-watershed using an area-weighted average of parameters for upland and lowland soils (see Section 2.4). Several references indicate that the maximum infiltration rate for peat soils is 13 inches per hour, whereas for sandy loam soils is 3 inches per hour (e.g., Rawls et al., 1992; Linsley et al., 1958). The XP-SWMM help documents indicate, however, that for conditions in which soils are subject to drainage but not to drying out (as is the case with wetlands and areas with high groundwater tables), the values above should be divided by three. Hence, the values selected in this model for the parameter F_0 were 4.25 inches per hour for lowland soils and 1.00 inch per hour for upland soils. In the case of the minimum (asymptotic) infiltration rate, the values recommended for the predominant soil types in the study area range from 0.05 to 0.30 inches per hour, which correspond to hydrologic soil groups B and C. Values used in this model for the parameter F_c were 0.20 inches per hour for lowland soils and 0.15 inches per hour for upland soils. Finally, the exponential decay coefficient is usually assumed to vary between 0.00083 and 0.00139 1/s, which corresponds to 95 percent and 99 percent reductions of the maximum infiltration rate in one hour, respectively. Values used in this model for the parameter k were 0.00083 1/s for lowland soils and 0.00115 1/s for upland soils.

3.7 Snowmelt

XP-SWMM keeps a record of the snow depth over the winter by discriminating between rain and snow precipitation events, based on a dividing air temperature. Precipitation is considered snow if it occurs at an air temperature below the dividing temperature, and conversely, it is considered rain if it occurs at a temperature above the dividing temperature. The dividing temperature was set to 34°F, as recommended in the XP-SWMM help documents. In addition, a snow gage correction factor is included to account for the possibility of snow sublimation and systematic errors in snowfall measurement; values from 0.7 to 1.0 were considered for the Partridge River model.

The snowmelt subroutine in XP-SWMM has two components: a heat balance on the snowpack, and a snowmelt calculation. The heat balance keeps track of the amount of energy needed to raise the temperature of the snowpack to 32°F (Brooks et al., 1997), so that melting can begin. The antecedent temperature weighting index is an important parameter in the heat balance, defining the relative significance of air temperature during the previous days relative to the current air temperature in determining the temperature of the snowpack; values between 0.1 and 0.5 were used. The free-water holding capacity parameter defines the amount of water that is retained in the snowpack before runoff occurs (for more details, see copy of Appendix II-SWMM Manual in Appendix C of this report); values ranging between 5 and 10 percent were recommended by the XP-SWMM help documents, as corresponds to the deep snow packs typical of northern Minnesota.

Once the temperature of the snowpack has reached the melting point, the snowmelt is estimated using the temperature index method, in which *SMELT* = snowmelt rate is expressed in terms of *DHM* = melt coefficient; *TA* = air temperature; and *TBASE* = snowmelt base temperature. During rain on snow events, a modified version is used to parameterize the melt coefficient based on the prevailing weather conditions. Initial runs of the XP-SWMM model for the Partridge River watershed suggested using values between 32°F and 40°F for the base temperature *TBASE*. While most references recommend using 32°F, the dense canopy coverage observed in the forests of the study area has a shading effect that typically delays warming of the snow during the spring. This is in part supported by other studies in boreal forests (e.g., Gray and Prowse, 1992), with base temperatures in the range of 37°F to 39°F. Furthermore, Verry (1986) indicates that forests usually retard snowmelt. The Partridge River watershed is heavily forested with dense undergrowth, therefore higher temperatures are required to achieve snowmelt. Gage data confirms that significant snowmelt is observed only after temperatures are within the range of 37°F to 39°F. The melt coefficients are derived from a sinusoidal curve that varies over the year, and uses two values for the best fitting. The SWMM 4.0 manual suggests 0.001 and 0.006 inches per day per degree Fahrenheit in December and June, respectively (Huber and Dickinson, 1992). Slightly smaller values are recommended for forested areas (e.g., Rango and Martinec, 1995).

3.8 Groundwater

The groundwater subroutine in XP-SWMM is based on a simple two-layer soil model consisting of a saturated and an unsaturated zone (see schematic in Figure 17). The volume of water that infiltrates into the soil is calculated at each time step using the Horton infiltration equation. This water is then input into the groundwater subroutine for calculations of water table depth and groundwater outflow (recharge) to the stream channel. When the water table reaches the surface, infiltrated water is routed back to the surface, adding to the runoff. The groundwater parameters were estimated from the XP-SWMM help files and a previous study that developed a model for water table fluctuations in peat soils (Letts et al., 2000). Since the water table is very near the surface in the lowlands (see RS44), the groundwater subroutine tends to be limited by how quickly the water can exit the saturated zone through evapotranspiration and groundwater outflow. Groundwater outflow is modeled by the Dupuit-Forcheimer approximation, in which *GWFLW* = groundwater rate is expressed in terms of

K = horizontal hydraulic conductivity; L = distance to the maximum height of the water table;
$D1$ = the average water table elevation; BC = the elevation of the bottom of the channel; and
TW = the elevation of the water in the channel.

The tailwater at the downstream end of the modeled reach was set to 0.5 to 2.0 feet above the channel bottom, and the value of ($TW - BC$) was set constant throughout the model run. The groundwater outflow parameter was estimated assuming the hydraulic gradient occurs over a length L of 100 to 335 feet, and a saturated hydraulic conductivity K characteristic of hemic peat (typically found within the watershed) of 0.28 inches per hour (Letts et al, 2000). Thus, it is implicitly assumed that the wetlands are adjacent to the channel, not the uplands. It is worth mentioning that the hydraulic conductivity for peat may vary over several orders of magnitude, depending on the level of decomposition of the peat. Values as high as 39.7 inches per hour are suggested for fibric peat near the surface, and values as low as 0.014 inches per hour are recommended for highly decomposed layers (Letts et al., 2000; Boelter and Verry, 1979).

Within the groundwater model, evapotranspiration is modeled as a partition of the overall evaporation that is entered as input to the model (see Section 3.4). Evaporation first occurs from depression storage at the surface. The depth of water that is still available for evaporation is then passed to the groundwater model and partitioned between the upper (unsaturated) zone and the lower (saturated) zone, based on a single coefficient. This coefficient was set so that between 30 and 90 percent of the evapotranspiration should occur in the unsaturated zone, depending on the water availability. The evapotranspiration is limited based on the volume of water available in the soil layers and the wilting point specified for the soil.

Results from the XP-SWMM model presented in Section 5.3 indicate that the actual total evapotranspiration from the study watershed (including evaporation from open water surfaces) is 16.8 inches per year. This value is very similar to the mean evapotranspiration of 16 inches per year suggested by Baker et al. (1979).

3.9 Hydraulics

XP-SWMM routes flows to downstream reaches using dynamic flow equations. The basic differential equations for solving open channel unsteady flow problems are derived from the gradually varied, one-dimensional, unsteady flow equations for open channels, otherwise known as the Saint-Venant shallow water equations (Roesner, 1988). They are non-linear hyperbolic partial differential equations and analytical solutions are unknown or unwieldy except in simplified situations. Numerical methods must be used to solve the equations since no general analytical solution exists. In addition to a numerical solution the equations require that upstream and downstream boundary conditions and initial conditions to be defined by the user. The Saint-Venant equations are valid as long as the flow is a gradually varied one-dimensional flow, vertical acceleration is negligible, hydrostatic pressure is a valid assumption, and the frictional resistance is the same as for steady flow.

The hydraulic data that is required by the model includes: pipe locations, sizes, types, materials, and elevations; natural channel cross-sections; storage basin elevation, volume, and outflow characteristics; and surface flow characteristics (overland flow upstream of the channels). We have used the best available data to estimate these characteristics (USGS quadrangle maps, field data collection, available plans, etc.). The study area contains large areas of wetlands, in particular near or within the footprint of the Mine Site, and storage in these wetlands may have a significant impact on the total flows. The storage that is available in upstream wetlands has been investigated based on previous studies, and have been adjusted during the calibration process. Information on stream configuration and crossings were obtained from readily available plans and limited field observations.

The Partridge River and its tributaries were modeled as natural cross-sections. Natural cross-sections were cut from the composite DEM using HEC-GeoRAS 4.1. Cross-sections were then imported into HEC-RAS and edited manually to add channel data. Since all of the topography data were derived from aerial survey methods, the elevations of the channels were defined with respect to the water surface rather than the channel bottom. Channel widths were measured from the 2003 FSA aerial photo. The channel bottom elevation was estimated based on field observations by reducing the elevation in the approximate channel boundaries by 4 to 6 inches in the upstream areas, and by 8 to 12 inches in the downstream areas. In cross-sections that were characterized by a wide, flat wetland area, a trapezoidal cross-section with 4(H):1(V) side slopes was added. In steeper cross-sections, all cross-section elevations located between the bank stations were lowered by the specified amounts indicated above. Cross-section survey data from the Rosgen Classification work (see RS26) were available at two cross-sections along the Partridge River just south of the Mine Site.

Originally, Manning’s “n” values for channels were based on recommendations from Chow (1959); values ranged from 0.032 in flatter, wetland areas to 0.050 in steep, rocky sections. It is anticipated, however, that flows are significantly slowed due to the existence of numerous beaver dams, so the prevailing hydraulics were adjusted during calibration in terms of higher values of the Manning’s “n” coefficient. Based on photographs of similar streams presented in Barnes (1967), a range of 0.059 to 0.073 was used for the Manning's "n" coefficient. A constant value of 0.10 was used for overbank areas, which were all covered in forest or brush.

The locations of crossings were defined using aerial photography and USGS quadrangle maps, and field verified. Crossing configurations were identified and sizes were approximated during the field site visit. The elevations of road and railroad overflows were determined differently in the Mine Site where 2-foot topography was available, than in the rest of the watershed where elevation data were only available from the 10-meter NED dataset:

- Within the Mine Site, road and railroad elevations could be determined from the topography data. In the field, rough surveys of the culverts were taken to measure culvert sizes and estimate culvert inverts relative to the top of the road or railroad. Thus, culvert invert elevations were estimated based on the road top elevations from the topography. These elevations were checked against the low elevations given by the topography at the approximate culvert inlet locations and generally found to agree within 1-2 feet. The outlet invert was set 1 foot below the inlet invert for all of the culverts within the Mine Site, for a slope of approximately 1 percent.

- In the remainder of the watershed, approximate culvert invert elevations were estimated from the DEM. Most bridge and culvert sizes within the study watershed were measured in the field, but a few had to be estimated based on their location in the watershed and the size of measured culverts upstream or downstream. Road top elevations were set to 2 feet above the top of the culvert, based on field observations and DEM elevations. Bridge deck elevations were estimated from the DEM.

4.0 Surface Water Data in Study Area

4.1 Flow Data

As indicated in Section 1.0 of this report, the study area corresponds to the catchment area of USGS gaging station #04015475 – Partridge River above Colby Lake at Hoyt Lakes. The period of record of daily flow data at this gaging station is 10 years (see Table 6). Additional flow data from other sites would be needed to extend the period of analysis in order to get more robust estimates of the magnitude of extreme hydrologic events.

Table 6 summarizes the information available for daily flows at USGS gaging stations located within the St. Louis River watershed in Minnesota. This Table 6 also includes information about two other USGS gaging stations (#05130500 and #05126000) that are not located within the St. Louis River watershed (see Figure 18), but have relatively long periods of record, are located close to the study area and have similar hydrological characteristics. In addition to the gaging station that defines the study area (#04015475), five other stations (highlighted in Table 6) were selected for further evaluation. This preliminary selection was based on distance from the study area, size of the catchment, hydrological setting of the watershed, and size and quality of the flow record.

A correlation analysis was performed for different combinations of paired-coincident flow records among the six USGS gaging stations referred to above. The results are presented in Table 7. The value of the correlation coefficient ranges between -1 and +1 by definition. The closer the correlation coefficient is to +1 the better indication that the time variation of the water discharge is similar in the two stations compared; in other words, high (or low) flows at one station at a given time correspond to high (or low) flows at the other station. Table 7 shows that, in statistical terms (prior to other considerations):

- The correlation between the Partridge River flow records at the USGS gaging stations above Colby Lake (#04015475) and on the South Branch (#04015455) is very high, one indication that the study area could be considered hydrologically homogeneous.

- The flow records of USGS gaging stations at Partridge River near Aurora (#04016000) and Dunka River (#05126000) show a good correlation with that of Partridge River above Colby Lake (#04015475). The flow records of these two stations appear suitable for comparison and/or extension of the flow record at the study site.

- The flow record of USGS gaging station at Second Creek near Aurora (#04015500) does not show as good a correlation (compared with the results referred to above) with that of Partridge River above Colby Lake (#04015475).

- The flow record of USGS gaging station at Sturgeon River (#05130500) does not show as good a correlation (compared with the results referred to above) with that of Partridge River above Colby Lake (#04015475).

4.2 Mine Discharges and Water Appropriations

Mining activities have impacted the hydrologic regime within these gaged watersheds and the impact of mine discharges on the gage data can be significant. Potential impacts on streamflows due to large-scale mining at the headwaters of Partridge River, Dunka River and Second Creek date back to 1956. Reserve Mining Company (acquired later by Northshore Mining Company -Northshore-) and Erie Mining Company (acquired later by LTV Steel Mining Company -LTVSMC-, and subsequently purchased by Cliffs-Erie Mining Company after closure of LTVSMC mining operations in May, 2000) began open pit mining operations in 1956. Figure 19 through Figure 22 show the approximate locations of past and current water appropriations from and discharges into the Partridge River, Dunka River and Second Creek. Water appropriations data was obtained from the MDNR from the website http://www.dnr.state.mn.us/waters/watermgmt_section/appropriations/wateruse.html. Water discharge data and GIS shapefiles were obtained from the Minnesota Pollution Control Agency (MPCA) from the website http://www.pca.state.mn.us/data/edaWater/index.cfm.

Although mine discharges to the three streams listed above have occurred periodically since 1956, there are no mine pumping records available prior to 1988. Based on phone conversations with personnel from Northshore Mining Company and an email communication from John Adams -MDNR- on September 7, 2005 it is not clear when mine discharges to the north branch of the Partridge River occurred before 1988. It is a fact, on the other hand, that the south branch of the Partridge River has not been impacted by mining activities.

There are mine pumping records from LTVSMC and Northshore beginning in 1988, and this information is available from the MPCA for the period 1999-2005. No records are available prior to 1988. In the specific case of the Peter Mitchell Pit, exploited by Northshore since 1956, mine discharges to the Partridge River, Dunka River and Second Creek have occurred and will continue in the future. Because these mine discharges depend on the area being mined, they never occur simultaneously to more than one stream. Therefore, there are periods in which no discharge occurs from the Peter Mitchell Pit to the Partridge River. When mine discharges do occur, the maximum flow rate has been 34 cubic feet per second, which corresponds to the maximum permitted discharge from the Peter Mitchell Pit to the Partridge River. The total maximum permitted discharge to the Partridge River is 36.3 cubic feet per second; other National Pollutant Discharge Elimination System (NPDES) permits to discharge in the Partridge River account for 2.3 cubic feet per second. Northshore's maximum permitted discharge to the Dunka River is 86 cubic feet per second; the total maximum permitted discharge to Second Creek is 62 cubic feet per second.

LTVSMC constructed Whitewater Reservoir (formerly known as Partridge Lake) in 1955 to help secure supply of make-up water from Colby Lake for its ore processing operations, while staying in compliance with the Water Appropriation Permit No. P.A. 49-135 issued by the State of Minnesota in 1950. Diversion works between the two water bodies were constructed to allow gravity flow from Colby Lake to Whitewater Reservoir and pumping from Whitewater Reservoir to Colby Lake. The Partridge River flows into and out of Colby Lake. The permit does not allow water to be appropriated when the water level in Colby Lake is below 1,439 feet above mean sea level unless it is replaced with an equal volume from Whitewater Reservoir. Minnesota Power recently acquired the diversion works between Whitewater Reservoir and Colby Lake, and it operates a power plant at the Laskin Energy Center. Information on water levels is available for some periods. However, precise information about the historic or current combined operation of the Colby Lake-Whitewater Reservoir hydrologic system is not available.

Water Appropriations Permit No. P.A. 49-135 allows for a maximum sustained withdrawal rate of 12,000 gallons per minute for any continuous 60-day period or a peak withdrawal rate of 15,000 gallons per minute at any time. As indicated above, the permit also requires that when the water level in Colby Lake falls below 1,439 feet above mean sea level due to low inflows, the withdrawal of water from Colby Lake is authorized up to the rate that can be pumped from Whitewater Reservoir to replace the water withdrawn.

4.3 Extension of Flow Data Record

It is clear from Section 4.2 that no sufficient and/or reliable information is available to determine the true effect of mine discharges and water appropriations on the flow records of the USGS gaging stations initially considered for extension of flow data for the study area. Therefore, applying standard procedures for transferring and/or scaling flows based on the records from other watersheds would not be reliable, unless paired data during coincident periods of record show the watersheds compared are hydrologically homogeneous. This was investigated briefly, as discussed below.

The flow record at the Partridge River gage above Colby Lake (USGS #04015475) may have been impacted by mine discharges on the north branch. The monthly average flow recorded at this gaging station during 1978-1988 varied between a minimum of 1.3 cubic feet per second and a maximum of 454 cubic feet per second; recall that discharges from the Peter Mitchell Pit could account for up to 34 cubic feet per second. Since the timing, duration and location of mining discharges may be different now than during 1978-1988, the present hydrologic regime of the Partridge River may not be well represented by the period of record at USGS #04015475. The data for the USGS gaging station at South Branch Partridge River (#04015455), however, are not affected at all by any mining discharges or water appropriations. Figure 23 shows that the water yields are similar at these two locations within the study area (see also Table 8). However, the period of record for the South Branch is only three years, and it would not significantly extend the record for the Partridge River watershed above Colby Lake.

The USGS gaging station at Partridge River near Aurora (#04016000) was not used because the flows at this location are influenced by the flows from Second Creek (see discussion below) as well as by the regulated water discharge from Colby Lake. As mentioned above, there is not sufficient information about the operational rules (water storage and release) of the Colby Lake-Whitewater Reservoir hydrologic system. The USGS gaging station at Second Creek near Aurora (#04015500) was not used because this watershed has been severely disturbed by mining activities, which presently occupy near 40 percent of the catchment area. In this regard, a comparison between pre-mining (based on topography circa 1900 from the Minnesota Geological Survey) and present conditions shows that the watershed boundaries have been significantly altered. Furthermore, Figure 23 shows that the seasonality of water yields is quite different than that observed in the two USGS stations along the Partridge River, upstream of Colby Lake.

The USGS gaging station at Dunka River near Babbitt (#05126000) is part of the USGS Rainy River hydrologic unit, which drains north toward Canada. The flow record of this station could have been useful for extension of gage data in the study area, if the mine related impacts were better defined. It has a period of record of daily flows of more than 16 years (though is not continuous), is located only 17 miles from the study area at a similar altitude (80 feet above USGS #04015475 and 60 feet below USGS #04015455), has a comparable land use distribution (see Table 8), and has a similar seasonality of water yields -flow per unit catchment area- (see Figure 23). In addition, Figure 18 shows that the catchment area for the stream location immediately downstream of the confluence of the north and south branches of the Partridge River is similar in size to the catchment area of the Dunka River at USGS #05126000 (53.4 square miles). However, the flow record at the Dunka River gage was likely impacted by mine discharges in a very significant way. Mining water has been discharged into the Dunka River upstream of the gaging station and seepage out of the Dunka River into one of the existing open pits has also occurred upstream of the gaging station. The combined impacts are not known. The monthly average flow recorded at this gaging station during 1978-1980 varied between a minimum of 0.3 cubic feet per second and a maximum of 211 cubic feet per second; permitted mining discharges could account for up to 86 cubic feet per second. Therefore, the flow record of the Dunka River was considered unusable for extension of the Partridge River flow record.

The USGS gaging station at Sturgeon River near Chisholm (#05130500) is also part of the USGS Rainy River hydrologic unit. The flow record of this station could have been very useful for extension of gage data in the study area, in particular because this station is still in operation. However, there appear to be some mine related impacts that significantly affect the winter flows. It has a period of record of daily flows of more than 60 years, and beginning in 1964, it has flow data recorded every 30 minutes and 407 sets of measurements on top width and cross-section of channel wetted area, flow velocity, gage height and water discharge. This station is located 37 miles from the study area, at a similar altitude (100 feet below USGS #04015475), has a comparable catchment area (1.8 times USGS #04015475), and has a similar land use distribution (see Table 8; the difference is primarily represented by areas occupied by shrub/scrub rather than forest as in USGS #04015475). The seasonality of the water yields is similar during the spring and summer, but not during the winter (see Figure 23). Mine discharges from the tailings basin of Hibbing Taconite Co. could explain the differences in water yields during the winter, which is the critical simulation period if the anticipated effect of the Mine Site development is a reduction in flows at the Partridge River. The monthly average flow recorded at this gaging station during 1978-1988 varied between a minimum of 4.0 cubic feet per second and a maximum of 499 cubic feet per second; permitted mining discharges could account for up to 27 cubic feet per second. MPCA records on mine discharges for the period 1999-2005 indicate that the maximum permitted discharge of 27 cubic feet per second occurred from the tailings basin of Hibbing Taconite Co. to the Sturgeon River in May 2001. This represents 5 percent of the corresponding average monthly flow recorded at USGS #05130500 (Sturgeon River near Chisholm). Therefore, the flow record of the Sturgeon River was considered unusable for extension of the Partridge River flow record.

5.0 Model Calibration and Validation

5.1 Calibration and Validation Approach

The calibration of the XP-SWMM model that was developed for the study area has been for the data corresponding to the water year 1984-1985 at USGS gaging station #04015475 (Partridge River above Colby Lake at Hoyt Lakes). This water year was selected because the ratio of the average gaged runoff to precipitation is about the same as the mean value of 0.40-0.45 suggested by Baker et al. (1979) for this region of Minnesota. The runoff to precipitation ratios for the years with overlapping data at the South Branch Partridge River near Babbitt and the Partridge River above Colby Lake at Hoyt Lakes gages are not ideal for calibration to typical conditions: the ratio for the 1978-1979 water year is 0.60, and for the 1979-1980 water year is 0.22. In addition, the 1978-1979 water year contains the flood of record, which is more than twice the annual maxima recorded during the remaining gage period, and therefore may skew the calibration.

The validation of the XP-SWMM model developed for the study area has been for the data corresponding to:

- USGS gaging station #04015475 (Partridge River above Colby Lake at Hoyt Lakes), for the period of record 1978-1988.
- USGS gaging station #04015455 (South Branch Partridge River near Babbitt), for the period of record 1978-1980.

Two statistics have been used to determine the degree of success of the XP-SWMM model in matching observed values (see e.g., Van Liew et al., 2003; Borah and Bera, 2004).

- The first statistic is the deviation of volume runoff D_v, which is defined as:

$$D_v = 100 \times \left(\frac{V_{obs} - V_{mod}}{V_{obs}} \right) \qquad (1)$$

  where V_{obs} = observed volume runoff for the simulation period; and V_{mod} = modeled volume runoff for the simulation period. The simulation period referred to may correspond to the whole period used in model calibration / validation. An alternative is to compute D_v for time intervals shorter (e.g., daily, weekly, monthly, seasonal or yearly) than the whole simulation period, and determine an overall D_v from the average or maximum value of the D_v computed for the shorter time intervals.

- The second statistic is the coefficient of efficiency *E*, which is defined as:

$$E = 1 - \left[\frac{\sum_{i=1}^{N} \left(Q_{obs}^{i} - Q_{mod}^{i} \right)^2}{\sum_{i=1}^{N} \left(Q_{obs}^{i} - \overline{Q} \right)^2} \right] \qquad (2)$$

  where Q_{obs}^{i} = i-th observed flow; Q_{mod}^{i} = i-th modeled flow; $\overline{Q}$ = mean of observed flows during the simulation period; and *N* = number of observed and modeled values during the simulation period. Similar to D_v, the coefficient of efficiency *E* could be computed for time intervals shorter than the simulation period.

The following criteria have been proposed to determine the degree of success in calibrating and validating the XP-SWMM model for the study area:

- The deviation of volume runoff D_v, computed on a water year basis, will vary between -40 and +40 for the model to be considered satisfactory. In other words, the annual volume of runoff values predicted with the model will be within ± 40 percent the observed values. The criterion was based on the use that will be given to the model (James, 2005). The hydrologic/hydraulic model of the study area will be used to evaluate relative changes on the average, minimum and maximum flows; the model is not intended to predict instantaneous flow values.

- The possible theoretical value of *E* is from minus infinity to one. Motovilov et al. (1999) suggest that the coefficient of efficiency *E* has to be greater than 0.36 for a model to be considered satisfactory. It is worth noting here that the coefficient of efficiency is not directly analogous to the correlation coefficient, which ranges between -1 and +1 by definition. A less strict interpretation is that negative values of the coefficient of efficiency *E* indicate a bad model performance, while values near to one indicate a very good model performance.

A third statistic was used to determine the degree of success of the XP-SWMM model in matching observed values specifically during periods of low flow. The emphasis on low flows is because low flows are critical for several of the issues under investigation for the EIS of the NorthMet Project. For instance, periods of low flow may portray the conditions under which potential impacts of the Mine Site on the water quality of the Partridge River and aquatic environment supported by this watercourse are more significant (see RS74).

The other two statistics presented above (the deviation of volume runoff D_v, and the coefficient of efficiency E) are not an appropriate measure of the model performance during periods of low flows (see Appendix B). The third statistic is a dimensionless version of the root mean square error *RMSE'*, which is defined as:

$$RMSE' = \frac{1}{\overline{Q}} \sqrt{\frac{\sum_{i=1}^{N} \left(Q_{obs}^{i} - Q_{mod}^{i} \right)^2}{N}} \qquad (3)$$

The 30-day period with the lowest flows during the period of evaluation (e.g., a water year) was selected as an appropriate time scale to represent low flow conditions because it is long enough to avoid comparing instantaneous observed and modeled flows, and it is short enough to avoid inclusion of runoff events caused by winter snowmelt. No references on acceptable ranges of *RMSE'* were found. Values in the order of 1.62 or less were considered acceptable as this would represent a discrepancy between observed and modeled flows of less than 0.10 inches in runoff over the 30-day period.

Calibration and validation are based on typical watershed conditions that are uniform over the area. Unusual data may indicate non-typical conditions such as debris jams, clogged culverts, ice jams; non-uniform conditions; data gaps; or a gage not working properly.

5.2 Calibration and Sensitivity Analyses

The calibration of the XP-SWMM model for the study area was carried out in two phases:

- **Phase 1.** Running the model with the groundwater component turned off, to test initial values of infiltration and snowmelt parameters to match runoff volumes.

- **Phase 2.** Running the model with the groundwater component turned on, to test the influence of the water table fluctuations and groundwater recharge on the modeled flows.

Runs conducted during Phase 1 showed that infiltration parameters had to be reduced significantly below recommended values in order to predict runoff volumes better matching observed values. The modeled hydrographs were also much more “flashy” than the ones recorded at the USGS gaging station; that is, both the raising and falling limbs of the modeled hydrograph were of very short duration compared to the recorded durations, with the largest disagreement on the recession curve. Furthermore, the model did a poor job in matching base flows.

The observed hydrograph for water year 1984-1985 has many of the properties observed in streamflow from perched bogs in Minnesota, as outlined by Boelter and Verry (1979). Large seasonal variations in streamflow can be anticipated as a consequence of fluctuations in the water table; in general, fluctuations in the water table do not necessarily have to be large to have an effect on runoff values when the extent of the catchment covered by wetlands is significant. A large percentage of the annual streamflow occurs during the spring as the water table rises to the surface as a consequence of melting of snow on the ground and thawing of the surficial soil layers. Reduced streamflow over the summer is expected following a fall in the water table due to evapotranspiration from the surficial unsaturated and deeper saturated soils layers. Low streamflows during late summer/early fall and through winter are typical, as surface runoff is negligible and groundwater is the primary source of flows.

Runs conducted during Phase 2 were intended to better understand the effect of the different model components and parameters on the annual volume of streamflow, the timing and magnitude of peak flows, and the magnitude of the baseflow. Volume of runoff, groundwater outflow, and evapotranspiration losses were recorded for each model run. The following paragraphs summarize the main results of the sensitivity analysis and model calibration. Values selected for the most sensitive model parameters in the final calibration run may or may not correspond precisely to the best fit observed in each sensitivity analysis (see Figure 24 through Figure 29), as some further adjustments were needed during the final model calibration to account for the combined effect of two or more model parameters. However, the values that provided the best fit to gage data were verified to be within the range of typical values expected.

- Most of the geometric and physical parameters were determined following the procedure explained above for delineation of sub-watersheds. There can be some uncertainty in the determination of the watershed width, so trial runs were conducted with the original values divided by two and multiplied by two. It was found that the watershed width can affect the timing of peak flows on the order of minutes or hours, but the effect is not that important when looking at the aggregated watershed defined as the study area.

- Depression storage may be adjusted to reduce the annual runoff volume, but this has little effect on the magnitude of peak flows.

- Infiltration parameters, including the infiltration regeneration factor used during dry periods, were found to be relatively unimportant once the groundwater component is turned on. In this study watershed, soils are highly permeable and the water table is at or near the surface for much of the year, so that infiltration is limited by the storage volume available in the soils. Therefore, infiltration tends to be controlled more by the rate of water removal from the saturated zone, which is part of the groundwater subroutine. In other words, infiltration rates depend not only on the soil type but also on the thickness of the unsaturated soil layer, with the latter defined by the depth of the water table. When the water table is near or at ground level then infiltration will be negligible and most precipitation will become runoff. When water table is deep, then infiltration will be high. The groundwater subroutine of XP-SWMM (see schematic in Figure 17) calculates a water balance for the unsaturated soil layer, which depends on (among other factors) the groundwater recharge to the channel, evapotranspiration rate and deep percolation rate. More details are provided in Appendix C.

- Initial tests suggested that the important groundwater parameters in this model are the "Initial Depth of the Unsaturated Zone", the "Fraction of Evapotranspiration to the Upper Zone", and the "Groundwater Outflow Coefficient". The results of the sensitivity analyses for these three parameters are shown in Figure 24 through Figure 26.
    - The "Initial Depth of the Unsaturated Zone" sets the water table depth at the beginning of the model run. It has an important effect on the runoff in the fall and early spring. If the initial depth is set significantly below the surface, fall precipitation and at least part of the spring snowmelt infiltrates into the soil, thus raising the water table elevation rather than appearing as runoff (see Figure 24). The best fit for the 1984-1985 water year occurred with this parameter set to 1.5 feet.
    - The "Fraction of Evapotranspiration to the Upper Zone" controls the fraction of evapotranspiration that comes out of the unsaturated zone versus the fraction that comes out of the saturated zone. This parameter has an important effect on the magnitude of peak flows during the summer. It allows the removal of water from the saturated zone, thus lowering the water table and allowing infiltration during the next precipitation event (see Figure 25). The best fit for the 1984-1985 water year occurred with this parameter set to 0.7.
    - The "Groundwater Outflow Coefficient" controls the rate of flow from the groundwater saturated zone to the adjacent channel. This parameter was calibrated by matching the baseflow that occurs over the winter (see Figure 26). The best fit for the 1984-1985 water year was defined after analyzing the combined effect with other parameters, with this parameter set to 0.00001 inches per day per square foot, corresponding to a saturated hydraulic conductivity of 0.28 inches per hour and the water table decreasing over a distance of 335 feet.
- Initial tests suggested that the important snowmelt parameters in this model are the "Snowmelt Base Temperature", the "June Melt Coefficient", and the "Snow Gage Correction

Factor". The results of the sensitivity analyses for these three parameters are shown in Figure 27 through Figure 29.

- The "Snowmelt Base Temperature" controls the timing of the snowmelt (see Figure 27). Snowmelt only occurs when the temperature rises above this temperature. The best fit for the 1984-1985 water year occurred with this parameter set to 38°F.
- The "June Melt Coefficient" controls the rate of snowmelt (see Figure 28). The best fit for the 1984-1985 water year was defined after analyzing the combined effect with other parameters, with this parameter set to 0.003.
- The "Snow Gage Correction Factor" allows for removal of snow from the snowpack before the snowmelt event occurs, either as a correction for erroneous precipitation gage data or for sublimation of snow. It affects the magnitude of the spring snowmelt peak (see Figure 29). The best fit for the 1984-1985 water year was defined after analyzing the combined effect with other parameters, with this parameter set to 1.0.

5.3 Calibration and Validation Results

Values for the XP-SWMM model parameters were set based on the hydrologic and hydraulic characteristics of the study area, and the calibration of the XP-SWMM model against gage data. The values used for the most sensitive model parameters are presented in Sections 3.0 and 5.2. It is important to mention that the modeled flows resulting from the calibration run for the water year 1984-1985 are not the same as those obtained for this water year from the validation run for the period 1978-1988 (see below). The difference is given by the initial conditions for the water year 1984-1985, which are thought to be better represented in the validation run as this accounts for the multi-annual variability in soil moisture conditions across the study watershed.

Figure 30 compares the flows obtained at the outlet of the study area with the calibrated XP-SWMM model for water year 1984-1985 to the flows recorded at the gaging station of Partridge River above Colby Lake. The calibrated model does a good job matching baseflows; the dimensionless version of the root mean square error is 1.6, which is below the acceptable limit suggested in Section 5.1. This is one of the primary objectives of this modeling effort, since the anticipated effect of the Mine Site development is to reduce flows along the Partridge River, in particular during periods of low flow (see RS73B) which can be critical for water quality (see RS74) and the aquatic environment.

Furthermore, Figure 30 shows that the calibrated model performs well in capturing the timing and magnitude of peak flows associated with the spring snowmelt event and subsequent summer floods.

However, the model is somewhat deficient in generating runoff volumes after passage of a flood hydrograph, which is represented by a deviation of volume runoff D_v of 34.2. This is within the criteria defined for calibration in Section 5.1. The deficiency of the model in generating runoff volumes on the falling limb of the hydrograph has a larger effect on the coefficient of efficiency E, which for the calibration simulation is 0.17. Although this value is outside the more strict range defined for calibration in Section 5.1, it is a positive value that is relatively close to unity for a scale of possible values between minus infinity and one. It should be re-emphasized that this hydrologic/hydraulic model will be used to evaluate expected relative changes on the average, minimum and maximum flows along the Partridge River during different stages of the mining project, not to predict instantaneous flow values. Therefore, the XP-SWMM model that has been presented here is considered to be calibrated.

Figure 31 compares the flows obtained at the outlet of the study area with the calibrated XP-SWMM model for the simulation period 1978-1988 to the flows recorded at the gaging station of Partridge River above Colby Lake. The calibrated model reproduces the overall trends in the flow pattern, and thus is considered to be validated. Supporting this conclusion, Table 9 indicates that during the critical low flow periods, seven out of the ten years of simulation the computed dimensionless root mean square error comply with the criterion established in Section 5.1. Of the three years that are not within the criteria, one is 1979-1980, which experienced extremely low runoff. Overall, the goodness-of-fit during periods of low flow is considered satisfactory given the uncertainty associated with potential discharges from the Peter Mitchell Pit (see Section 4.2).

In addition, Table 9 indicates that in terms of the deviation of volume runoff, eight out of the ten years of simulation comply with the criterion established in Section 5.1. Of the two years that are not within the criteria, one is 1978-1979, which experienced extremely high runoff. The other year had a deviation value (41) that was just above the criterion (40). The flow variability parameterized by the coefficient of efficiency is satisfactorily modeled four out of the ten years of simulation according to the criterion recommended by Motovilov et al. (1999), with five years in a row (1980 through 1985) having a value greater than that obtained in the model calibration. Overall, the coefficient of efficiency is a positive number eight out of the ten years of simulation. Therefore, the validation run confirms that the model is adequate for evaluation of the relative changes on the average and maximum flows along the Partridge River during different stages of the mining project.

Figure 32 compares observed and modeled flows at the gaging station of South Branch Partridge River. The results do not compare as well as at the gaging station above Colby Lake. The water year

1978-1979 was an anomalous wet year for which the results of the model validation were not as good as other years at either gage. In general, the period of record at the gage on the South Branch does not provide adequate data for statistical analysis and definition of the goodness-of-fit in the model validation.

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Tables

Table 1: Summary of historical and current Land Use/Land Cover Data available for study area

Land Data	Source	Year	Coverage	Additional Notes
Pre-settlement Vegetation	MDNR	1847-1907	Full	Map compiled in 1930 from US General Land Office Survey Notes
Land Use	LMIC	1969	Full	Statewide coverage including 9 Land Cover classes with 40-acre resolution based on ***aerial photos from 1968/1969***
NWI	USFWS	1977-1978	Full	Coarse resolution national wetland coverage
GAP Analysis	USGS	1992	Full	Based on same LandSat Imagery as NLCD but has more specific classifications and emphasizes natural plant communities
National Land Cover Dataset (NLCD)	MRLC (EPA, USFS, USGS, NOAA, NRCS, etc.)	1992	Full	Landsat 5 imagery
		2001	Full	Landsat 7 imagery; 21 land cover classifications
Landsat Land Cover	MRSC/ MDNR	1998	Full	Based on 30-meter Thematic Mapper satellite images from 1994-1996; covers NE Minnesota
Mine Features	MDNR	2003	Full	Outlines and identifies ***mine features*** in the Iron Range of Minnesota
Forestry Data	MDNR	2004	Partial	Data available only for areas located within MDNR forest land (1975-2004)
Forest Stand Type/Age	USFS	2005	Partial	Data available only for areas located within the Superior National Forest boundary – can be an indication of ***timber harvesting activity***
Barr Delineated Wetlands	Barr	2005	Partial	Wetland delineation in PolyMet Mine Site area

Table 2: Land Cover areas for the Partridge River watershed from the GAP Land Cover data set and NWI for wetlands only.

Land Cover Type	**Area (acres)**	**Area (%)**
Water	3076	4.7%
Wetland	28563	43.2%
Forest	31178	47.1%
Developed	2817	4.2%
Grassland	541	0.8%
Total	**66175**	**100.0%**
NWI Wetlands	25652	38.8%

Table 3: Land Cover Classification and Imperviousness

Land Cover Type	GAP Level 3 Classification	Percent Impervious
Open Water	Aquatic	100
	Marsh	100
Lowland Forest/Wetland	Black Ash	5
	Lowland Black Spruce	5
	Lowland Deciduous	5
	Lowland Northern White-Cedar	5
	Lowland Shrub	5
	Tamarack	5
Upland Forest	Aspen/White Birch	1
	Maple/Basswood	1
	Oak	1
	Pine	1
	Pine-Deciduous mix	1
	Spruce/Fir	1
	Spruce/Fir-Deciduous mix	1
	Upland Cedar	1
	Upland Conifer	1
	Upland Conifer-Deciduous mix	1
	Upland Deciduous	1
	Upland Shrub	1
Upland Crop/Grassland	Cropland	0
	Grassland	0
Mine Areas	Barren	10
	Developed	10

Table 4: Sources of Available Soils Information in the Study Area

Soil Data Source	Coverage	Additional Notes
STATSGO (State Soil Geographic Database) – USDA/NRCS	Full	Statewide Coverage - very general soils information to be used as a reference source, not to make decisions at the county level (ie. Regulations and permitting)
SSURGO (Soil Survey Geographic Dataset) – USDA/NRCS	None	Best available soils information done on a countywide basis. No survey available for Lake County. Survey data being compiled for Itasca County (not digital). Parts of St. Louis County are being surveyed (Embarrass, Meadow Lands, Crane Lake) while others are in digital review (Virginia, Duluth) but not available
USFS	Partial	Soils information coverage is limited to areas within the Superior National Forest
Minnesota Soils Atlas – LMIC (Univ of Minn) w/ NRCS/USDA/MnGS	Partial	Statewide coverage of generalized (1:250K scale) soils data (eg. HSG, pH, subsurface permeability, etc.) though there are patches of the state that have missing data, including some areas within the watersheds

Table 5: List of MN HIDEN and NWS weather stations near study area

Coop ID	Station name	County	Distance to site (mi)	Begin record	End record
210390	Babbitt 2 SE**	St. Louis	4.8	1920	1986
210387	Babbitt	St. Louis	6.4	1999	2005
213921	Hoyt Lakes 5 N	St. Louis	8.5	1958	1983
212576	Embarrass	St. Louis	11.0	1995	2005
218311	Tower 3S**	St. Louis	17.6	1926	2005
218307	Tower Dnr	St. Louis	19.5	1994	2005
212543	Ely	St. Louis	21.7	1998	2005
219101	Winton Power Plant**	Lake	23.4	1948	1995
210989	Brimson 1E**	St. Louis	23.6	1948	2005
218939	Whiteface Reservoir**	St. Louis	25.0	1949	1995
218613	Wales ENE	Lake	27.3	1948	2005
214068	Isabella 1 W**	Lake	27.6	1958	2004
212645	Eveleth Waste Water Plant	St. Louis	28.6	1987	2005
218543	Virginia**	St. Louis	28.7	1911	1985
211840	Cotton 10 E	St. Louis	33.6	1962	2002
212555	Ely 25 E	St. Louis	34.3	1998	2005
218421	Two Harbors 7 NW	Lake	36.2	1998	2005
219134	Wolf Ridge Elc	Lake	37.0	1993	2005
211771	Cook	St. Louis	37.4	1999	2005
218419	Two Harbors	Lake	43.3	1894	2005
213730	Hibbing Chisholm Airport	St. Louis	43.8	1963	2005
214096	Island Lake Reservoir	St. Louis	45.5	1949	1995
213727	Hibbing Power Substation	St. Louis	48.9	1948	1981
216213	Orr	St. Louis	50.7	1926	1954
211857	Crane Lake Ranger Station	St. Louis	51.5	1961	1977
215298	Meadowlands 1 NNW	St. Louis	51.8	1916	1985
211776	Cook 18 W	St. Louis	54.6	1959	1995
212248	Duluth International Airport	St. Louis	55.2	1941	2005
212246	Duluth Harbor Sta	St. Louis	59.0	1960	1994
212842	Floodwood 3 NE	St. Louis	61.5	1986	2005
211630	Cloquet	Carlton	68.3	1911	2005
214306	Kettle Falls	St. Louis	68.6	1944	2005
210754	Bigfork 5 ESE	Itasca	74.1	1959	1980

Note: As discussed in this report, the highlighted stations are located within an approximate 30-mile radius from the mine site.

** These seven stations have complete daily precipitation records during the water year 1984-1985.

Table 5 (continued)

Coop ID	Station name	County	Distance to site (mi)	Begin record	End record
214191	Kabetogama	St. Louis	75.4	1998	2005
213303	Grand Rapids Forestry Lab	Itasca	76.2	1915	2005
215175	Marcel 5NE	Itasca	78.7	1982	2005
216612	Pokegama Dam	Itasca	80.0	1887	2005
219173	Wright 4 NW	Carlton	81.0	1962	2005
217460	Sandy Lake Dam Libby	Aitkin	85.4	1893	2005
215598	Moose Lake 1SSE (Holyoke)	Carlton	89.7	1939	2005
216929	Rice Lake Nwr	Aitkin	97.1	1993	2005
219059	Winnibigoshish Dam	Itasca	98.7	1887	2005
210059	Aitkin 2E	Aitkin	111.0	1940	2005

Table 6: List of USGS stream gaging stations within the boundaries of the St. Louis River watershed, Minnesota and other nearby gages

Agency	Site number	Site name	Distance to site (mi)	Altitude (feet-MSL)	HUC	Drainage area (sq mi)	Daily flow data begin date	Daily flow data end date	Daily flow data count
USGS	04015410	Miller Creek near mouth at Duluth, MN	53.0	630	4010201		9/25/1992	9/30/1993	371
USGS	04015455	South Branch Partridge River near Babbitt, MN	9.2		4010201	18.5	6/1/1977	11/5/1980	1254
USGS	04015475	Partridge River above Colby Lake at Hoyt Lakes, MN	0.0		4010201	106.0	9/19/1978	11/2/1988	3698
USGS	04015500	Second Creek near Aurora, MN	3.3	1410	4010201	29.0	4/1/1955	9/30/1980	9315
USGS	04016000	Partridge River near Aurora, MN	3.2	1402	4010201	161.0	8/1/1942	9/30/1982	14671
USGS	04016500	St. Louis River near Aurora, MN	6.0	1371	4010201	293.0	8/1/1942	9/30/1987	16497
USGS	04017000	Embarrass River at Embarrass, MN	9.6	1410	4010201	88.3	8/1/1942	12/31/1964	8189
USGS	04018000	Embarrass River near McKinley, MN	13.2	1339	4010201	171.0	10/1/1953	9/30/1962	3287
USGS	04018700	Mud Hen Creek tributary near Central Lakes, MN	20.8		4010201		5/2/1985	6/10/1985	40
USGS	04018750	St. Louis River at Forbes, MN	25.0	1293	4010201	713.0	7/10/1964	3/20/1990	9345
USGS	04018900	East Two River near Iron Junction, MN	26.8	1335	4010201	40.0	6/24/1966	9/30/1979	4846
USGS	04019000	West Two River near Iron Junction, MN	28.5	1322	4010201	65.3	10/1/1953	9/30/1979	8400
USGS	04019300	West Swan River near Silica, MN	46.0	1360	4010201	16.3	4/1/1963	5/31/1979	5905
USGS	04019500	East Swan River near Toivola, MN	37.4	1260	4010201	112.0	10/1/1953	11/8/1971	5882
USGS	04020000	Swan River near Toivola, MN.	37.5	1252	4010201	254.0	10/1/1952	9/30/1961	3287
USGS	04021520	Stoney Brook at Pine Drive near Brookston, MN	57.0		4010201		5/26/2005	9/30/2005	128
USGS	04021530	Stoney Brook at Brookston, MN	51.3		4010201	97.3	6/22/1983	10/3/1984	438
USGS	04023150	Simian Creek near Brookston, MN	53.2	1238	4010201		7/8/1983	9/30/1984	307
USGS	04023600	Squaw Creek near Cloquet, MN	56.6	1205	4010201		7/19/1983	9/30/1984	296
USGS	04024000	St. Louis River at Scanlon, MN	58.7	1101	4010201	3430.0	1/1/1908	9/30/2005	35703
USGS	04024015	Otter Creek near Cloquet, MN	60.5	1228	4010201		7/19/1983	9/30/1984	296
USGS	05130500	Sturgeon River near Chisholm, MN	37.3	1305	9030001 (Rainy River)	180.0	8/1/1942	9/30/2005	23072
USGS	05126000	Dunka River near Babbitt, MN	16.8	1489	9030001 (Rainy River)	53.4	10/1/1951	11/5/1980	6123

Note: As discussed in this report, the highlighted stations were selected for further evaluation: yellow denotes the study area, blue highlights two stations outside the St. Louis River watershed, and green illustrates three other stations of interest for this study.

Table 7: Correlation analysis of USGS stream gaging station daily flow records

USGS #	04015475	04015455	04015500	04016000	05130500	05126000
04015475	1.00					
04015455	0.96	1.00				
04015500	0.78	0.74	1.00			
04016000	0.94	0.86	0.66	1.00		
05130500	0.83	0.86	0.76	0.81	1.00	
05126000	0.98	0.95	0.81	0.87	0.86	1.00

Table 8: Land use distribution in Partridge River, Dunka River and Sturgeon River basins (based on 2001 NLCD land cover data)

Land cover	Partridge River above Colby Lake (# 04015475)		South Branch Partridge River (# 04015455)		Second Creek near Aurora (# 04015500)		Sturgeon River (# 05130500)		Dunka River (# 05126000)	
	(acres)	**(%)**	**(acres)**	**(%)**	**(acres)**	**(%)**	**(acres)**	**(%)**	**(acres)**	**(%)**
Open water	2,264	3.4	848	7.5	1,779	12.7	6,008	5.2	521	1.4
Developed open space	74	0.1	0	0.0	32	0.2	1,735	1.5	152	0.4
Developed low intensity	16	0.0	0	0.0	48	0.3	417	0.4	57	0.2
Developed medium intensity	2	0.0	0	0.0	9	0.1	25	0.0	5	0.0
Developed high intensity	0	0.0	0	0.0	2	0.0	1	0.0	0	0.0
Barren land (rock / sand / clay) + Mining areas	1,626	2.5	0	0.0	3,768	26.9	1,894	1.6	2,274	6.2
Deciduous forest	18,525	27.5	3,311	29.4	3,602	25.7	40,496	35.1	8,631	23.6
Evergreen forest	37,921	57.3	6,227	55.4	2,589	18.5	33,512	29.0	19,863	54.4
Mixed forest	71	0.1	27	0.2	6	0.0	27	0.0	241	0.7
Shrub/Scrub	1,520	2.3	124	1.1	1,420	10.1	19,776	17.1	889	2.4
Grassland/Herbaceous	346	0.5	52	0.5	106	0.8	906	0.8	115	0.3
Pasture/Hay	56	0.1	0	0.0	9	0.1	1,441	1.2	0	0.0
Cultivated crops	35	0.1	20	0.2	0	0.0	202	0.2	8	0.0
Woody wetlands	2,826	4.3	506	4.5	250	1.8	7,341	6.4	2,802	7.7
Emergent herbaceous wetlands	1,187	1.8	130	1.2	379	2.7	1,634	1.4	968	2.6
Total (acres)	**66,169**		**11,247**		**13,998**		**115,416**		**36,524**	
Total (sq mi)	**103**		**18**		**22**		**180**		**57**	

Note: Highlighted information corresponds to main land cover types.

Table 9: Statistics used to determine degree of success of XP-SWMM model applied for validation period 1978-1988 at the outlet of study area

Water Year	Deviation of volume runoff	Coefficient of efficiency	Dimensionless root mean square error
1978-1988	33.1	0.24	1.15
1978-1979	76.6	0.09	0.68
1979-1980	23.6	-0.19	3.05
1980-1981	24.4	0.17	6.65
1981-1982	31.0	0.47	0.32
1982-1983	28.2	0.42	0.45
1983-1984	25.2	0.37	0.75
1984-1985	16.1	0.23	3.08
1985-1986	38.6	-0.03	0.04
1986-1987	41.0	0.39	0.27
1987-1988	1.8	0.15	1.55

Note: Statistics computed for the entire period used for model validation (1978-1988) do not represent an average of the values obtained for each water year, as the definitions of deviation of volume runoff, coefficient of efficiency and dimensionless root mean square error are not linear.

Figures

Figure 1: Location of study area

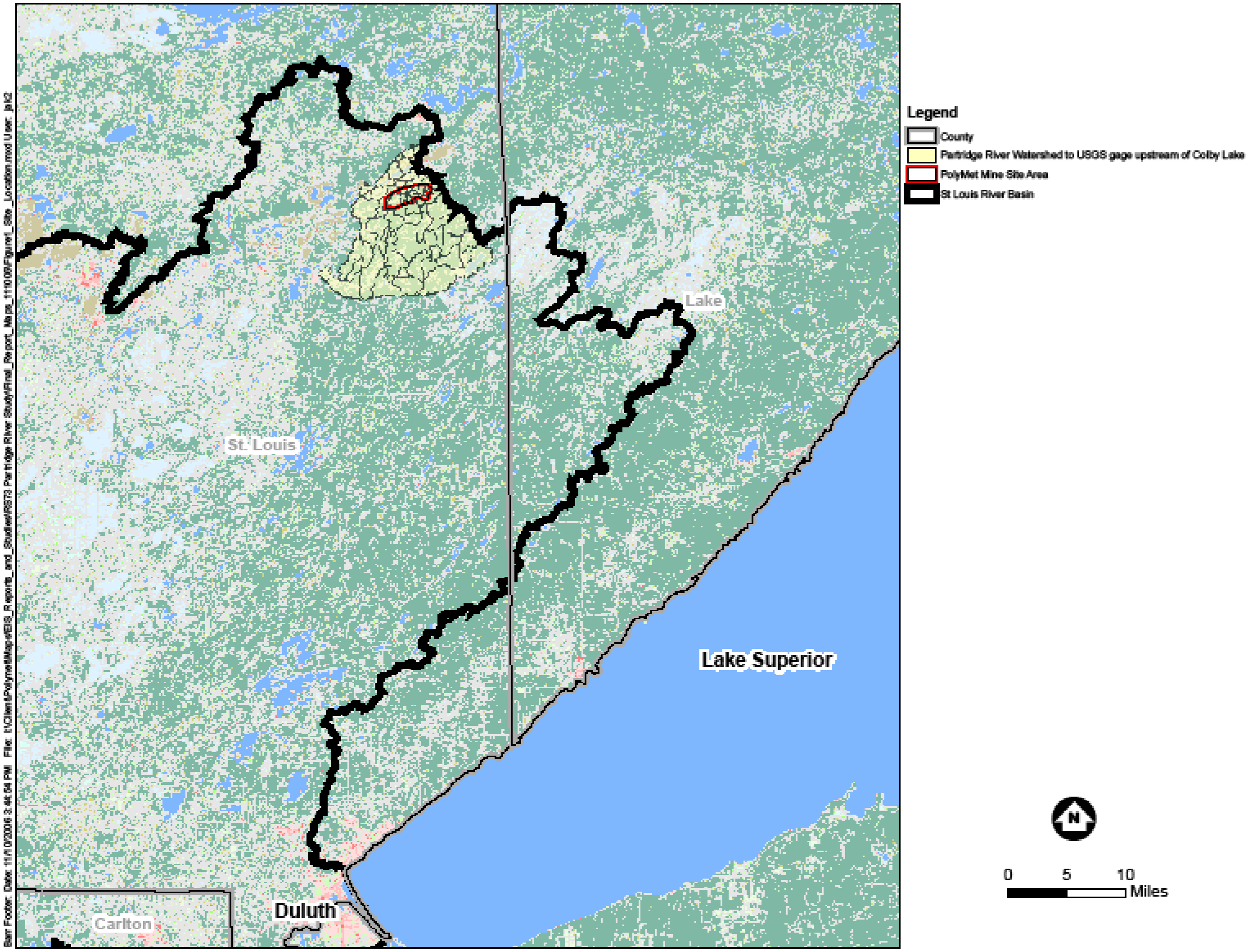


Figure 2: Main sub-watersheds of study area

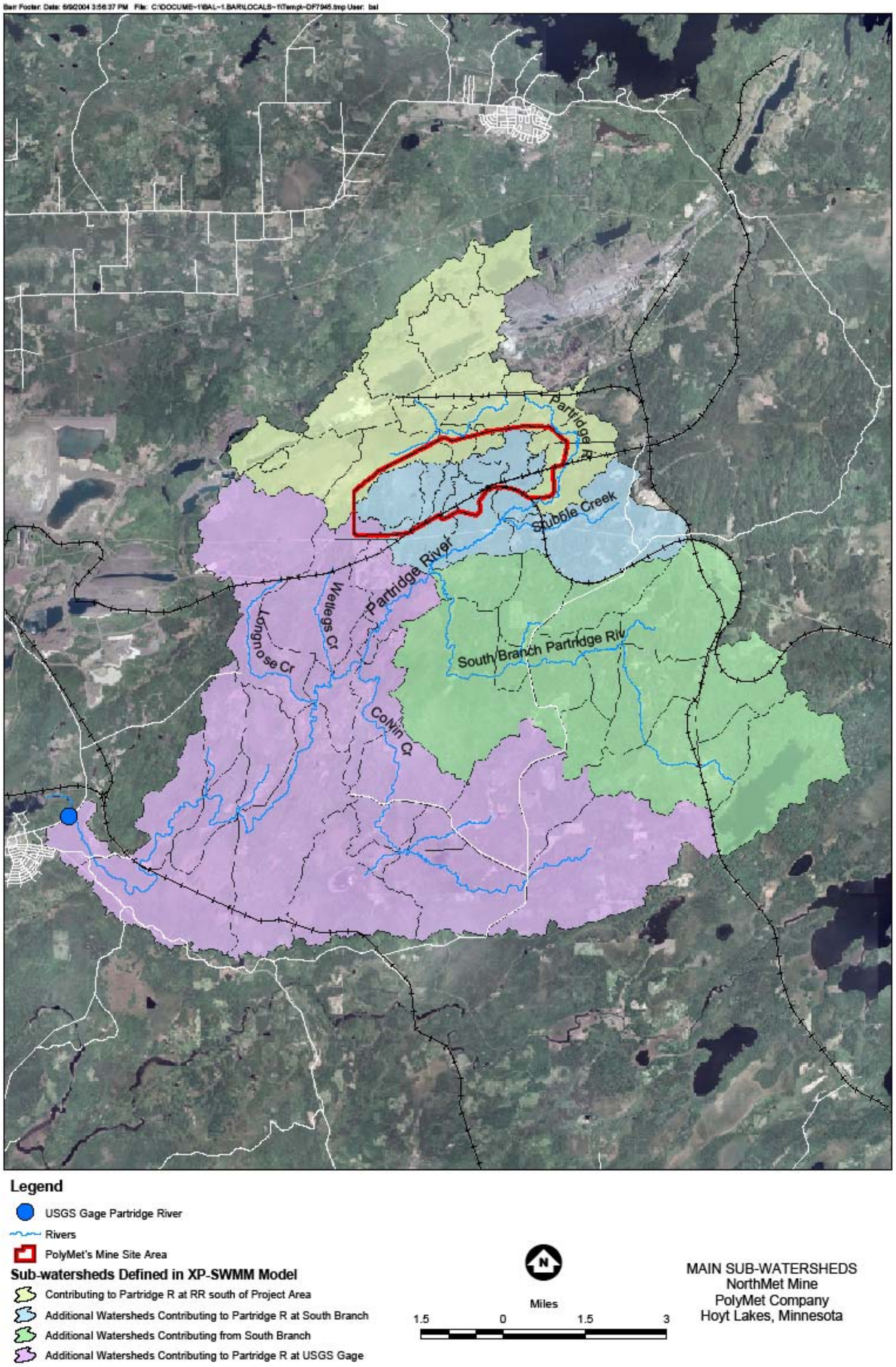


Figure 3: Schematic of water balance components / modules in XP-SWMM

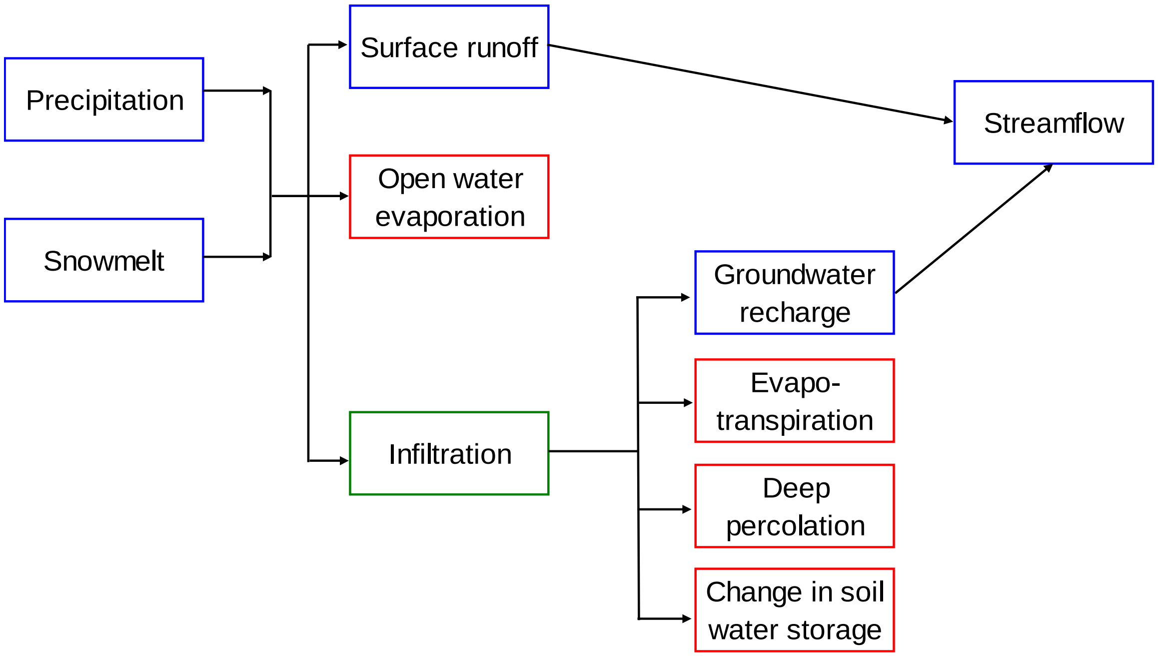


Figure 4: Sub-watersheds defined in XP-SWMM model of study area

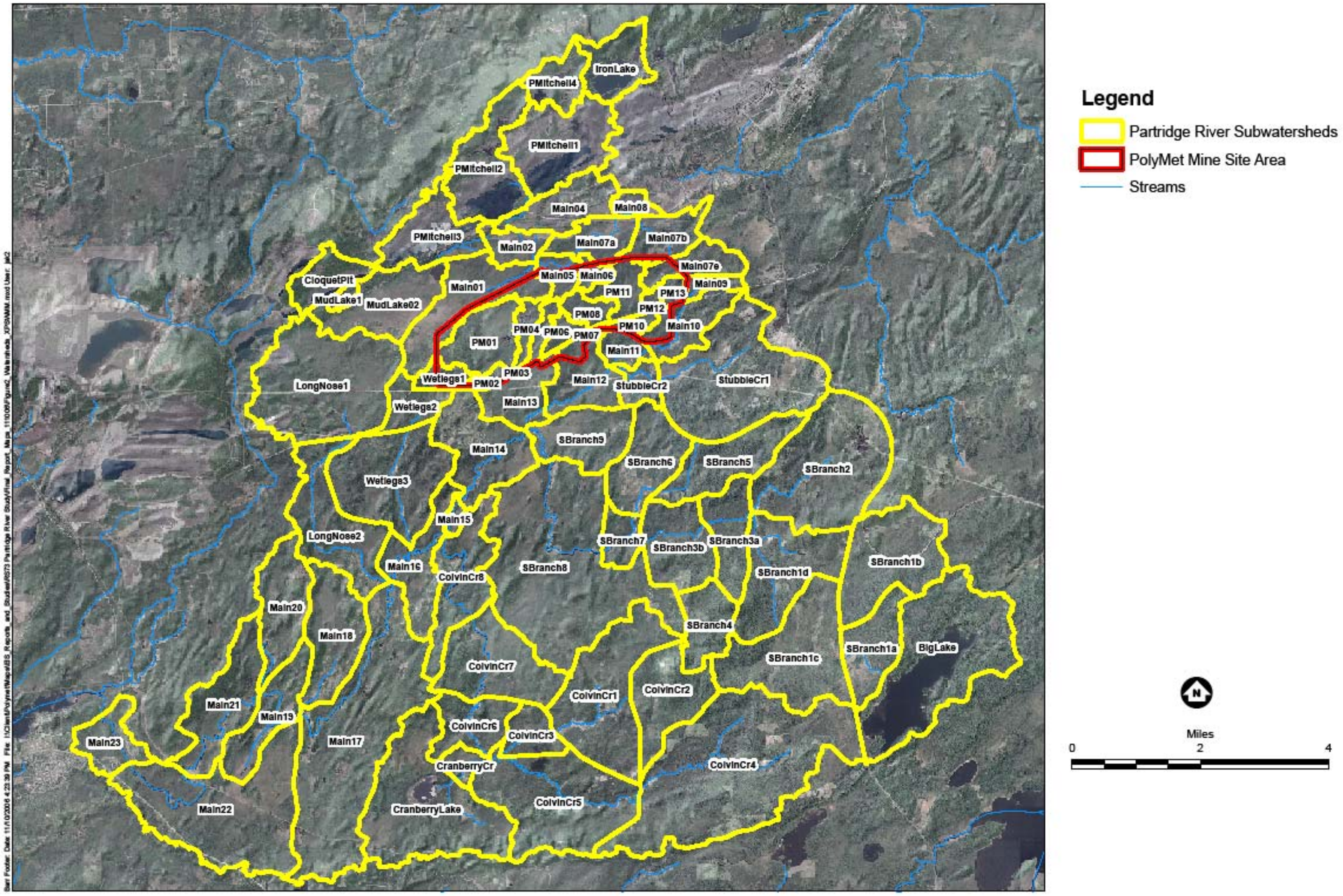


Figure 5: GAP Analysis coverage of study area

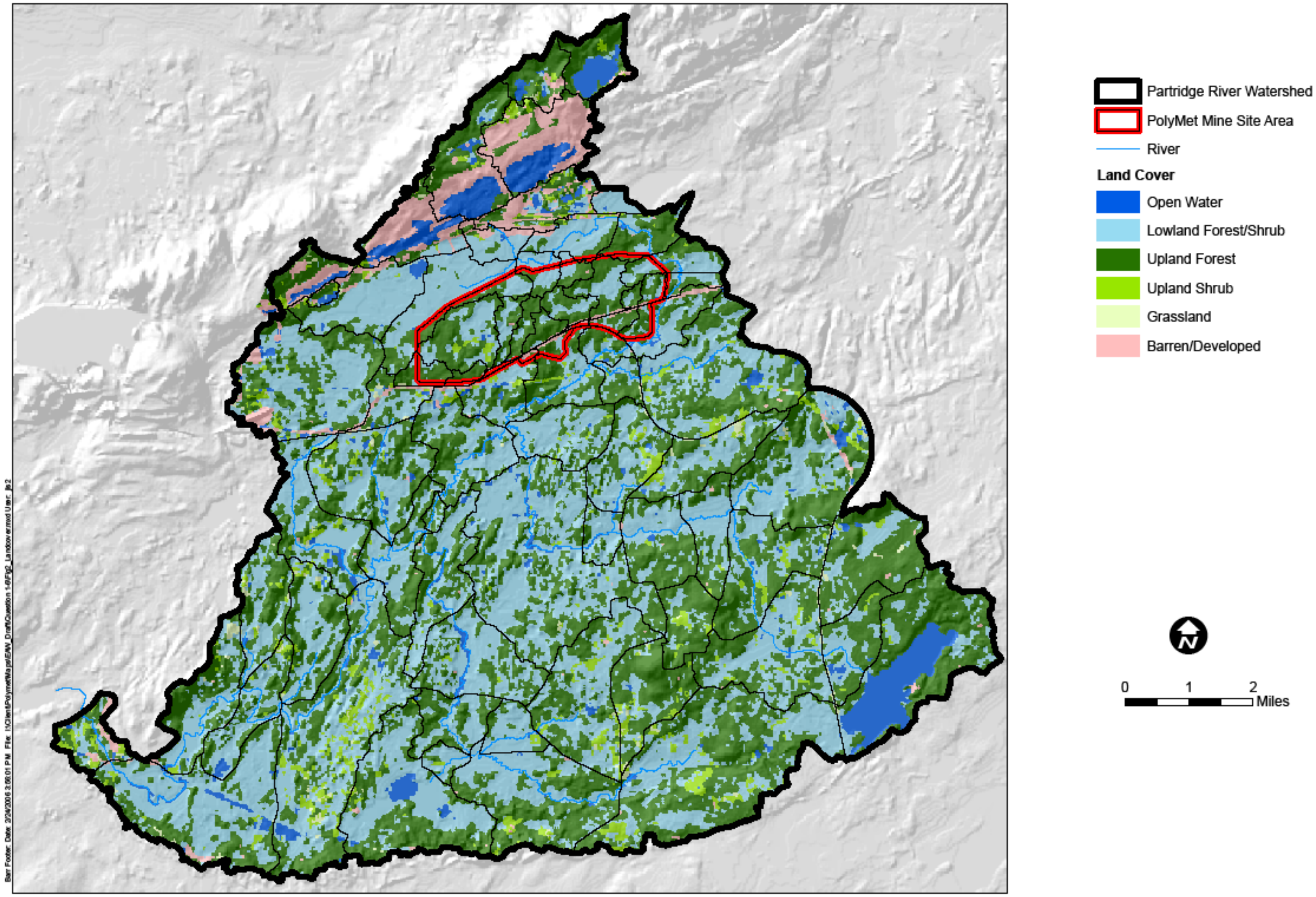


Figure 6: NWI wetlands coverage

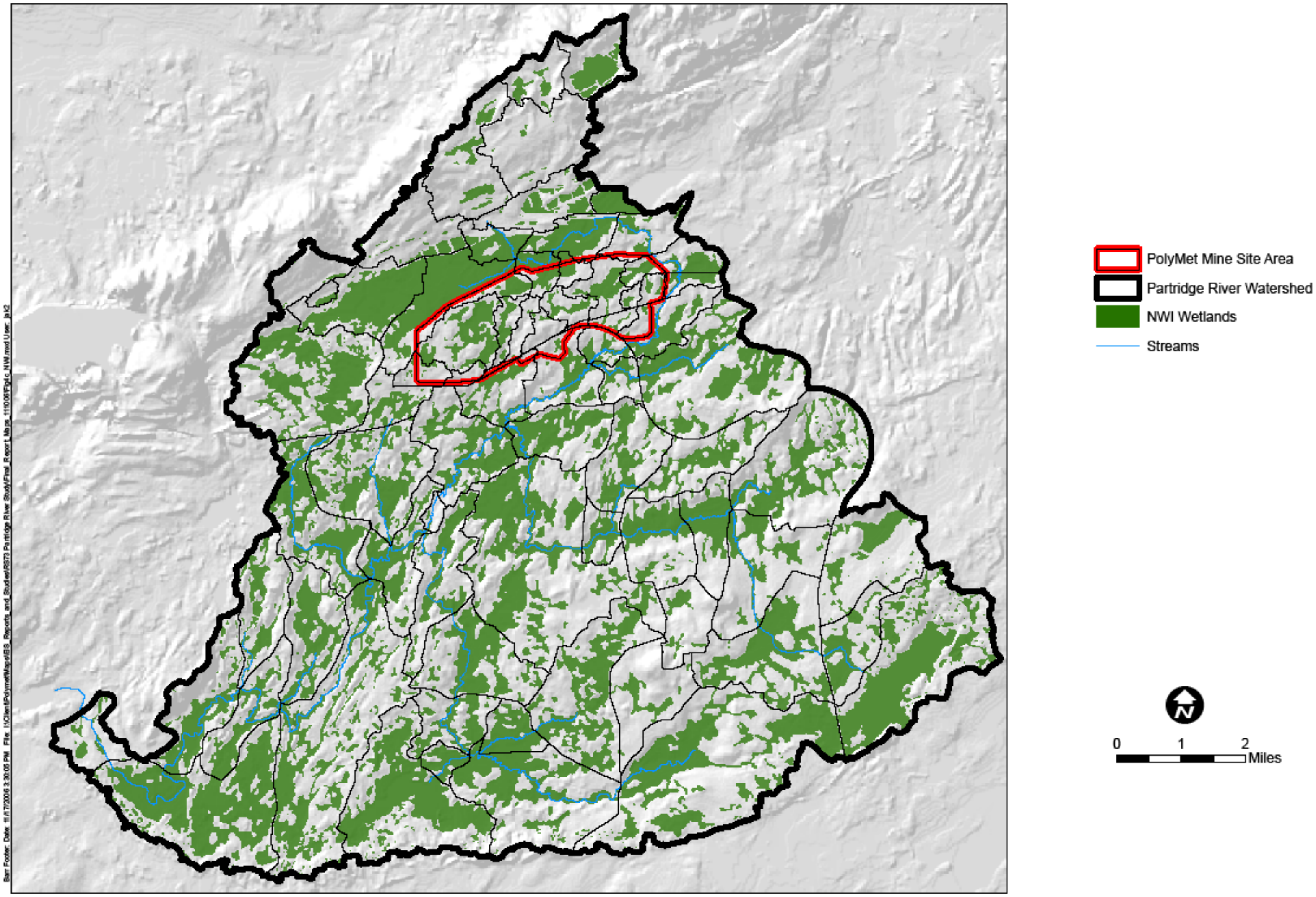


Figure 7: Age distribution of forest stand in study area, based on USFS-SNF

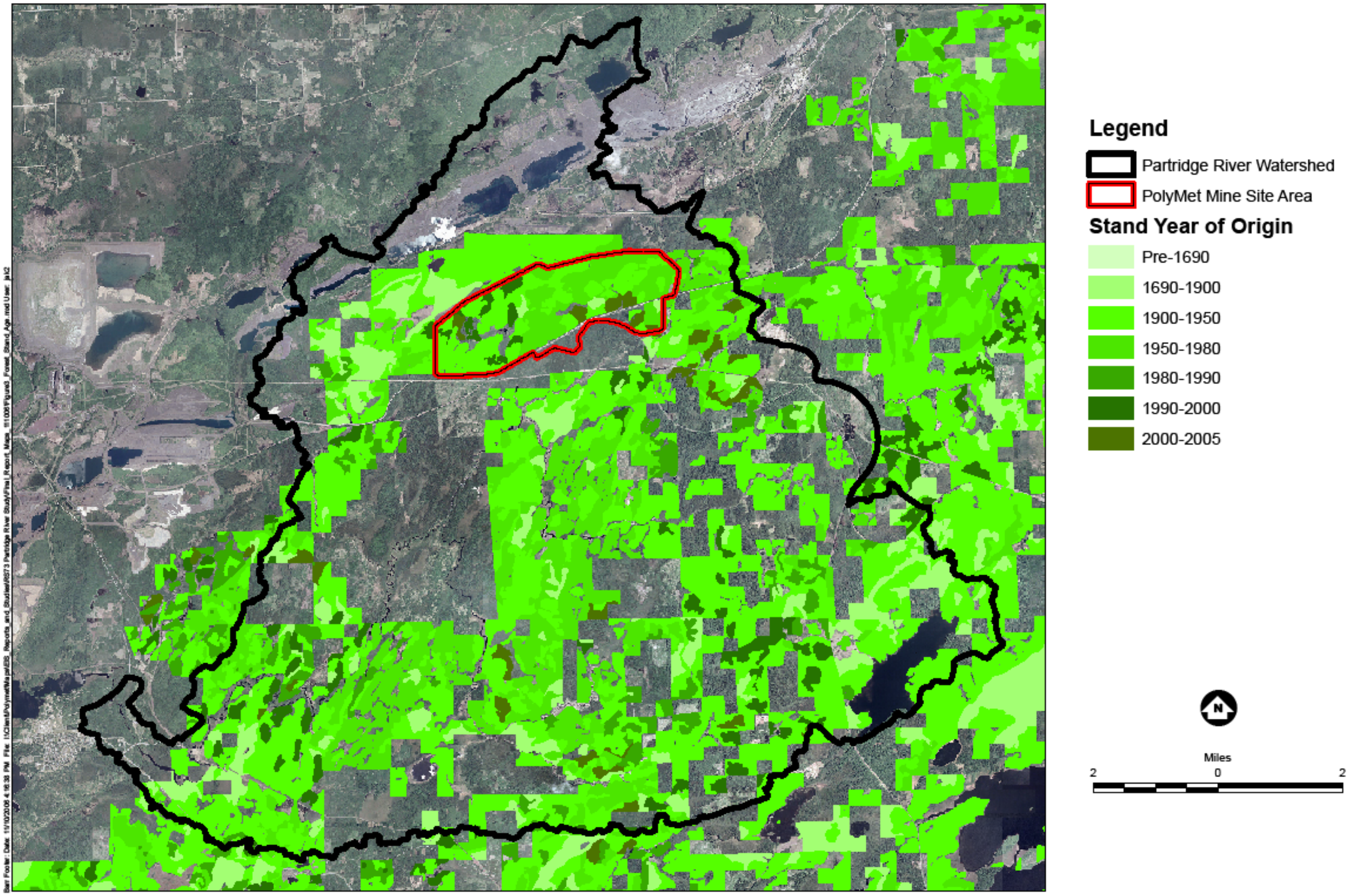


Figure 8: Detailed soils data in PolyMet's mine site area

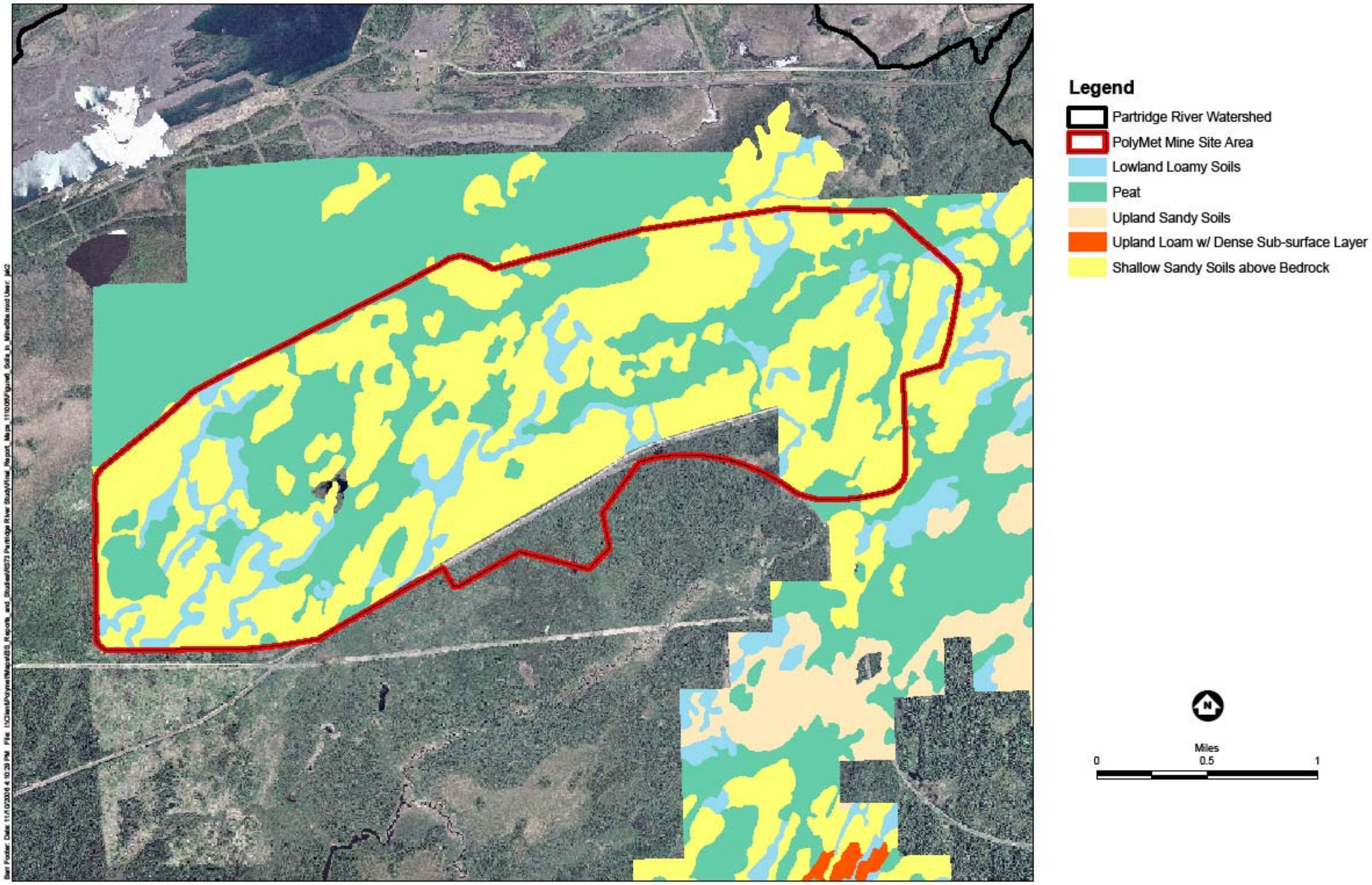


Figure 9: Soils data available for study area

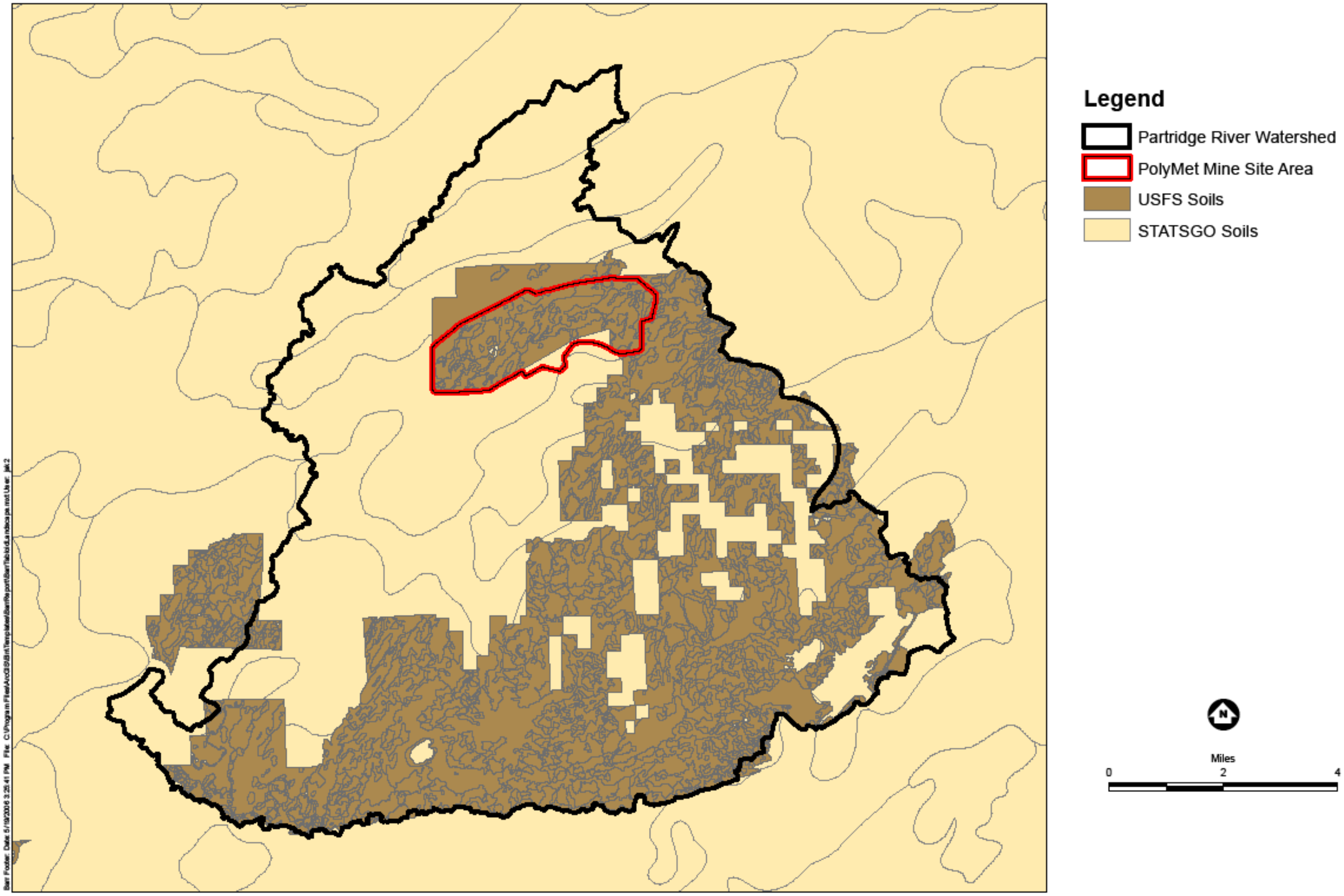


Figure 10: Location of weather stations

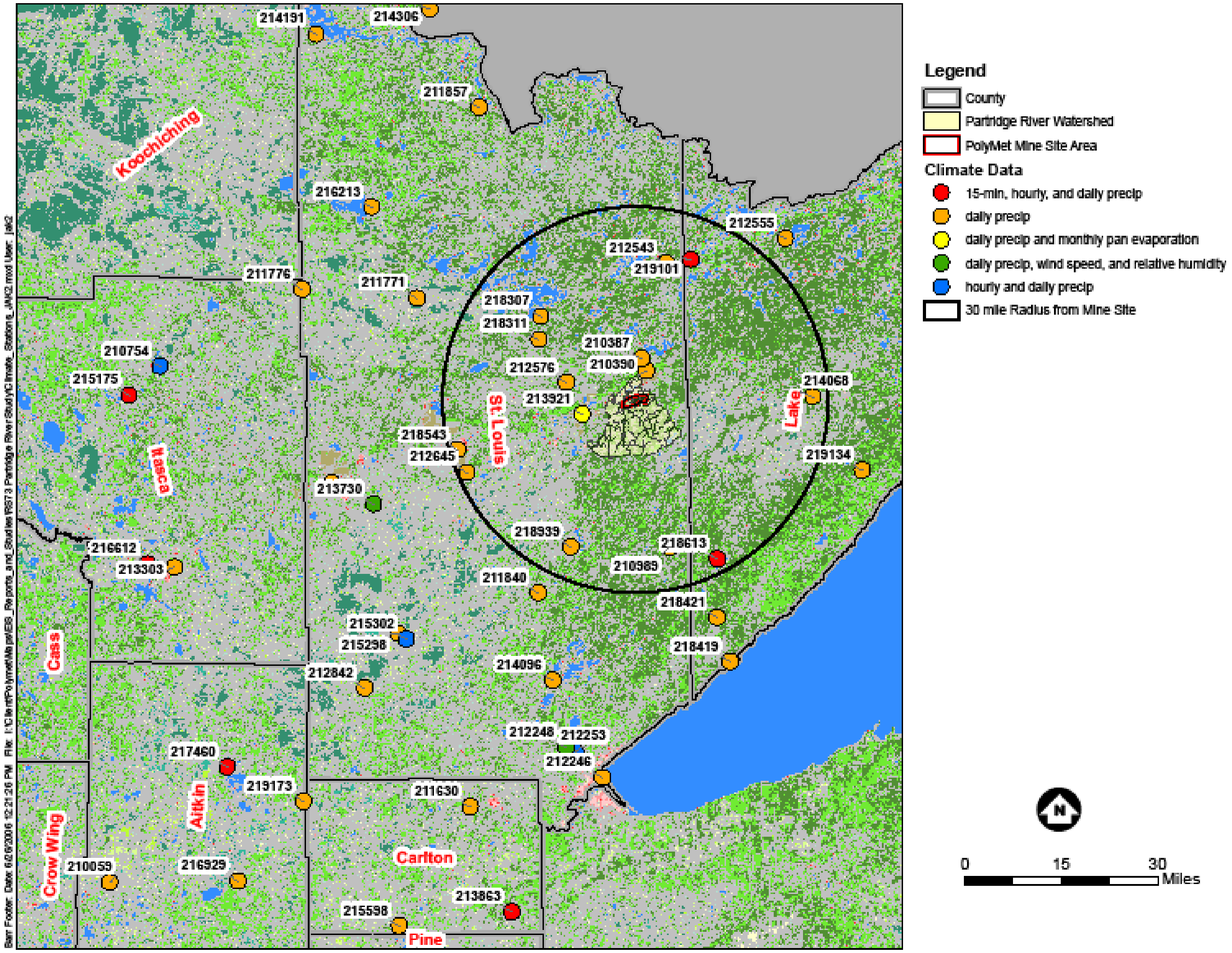


Figure 11: Cumulative precipitation at seven weather stations located within a 30-mile radius from the study area during water year 1984-1985

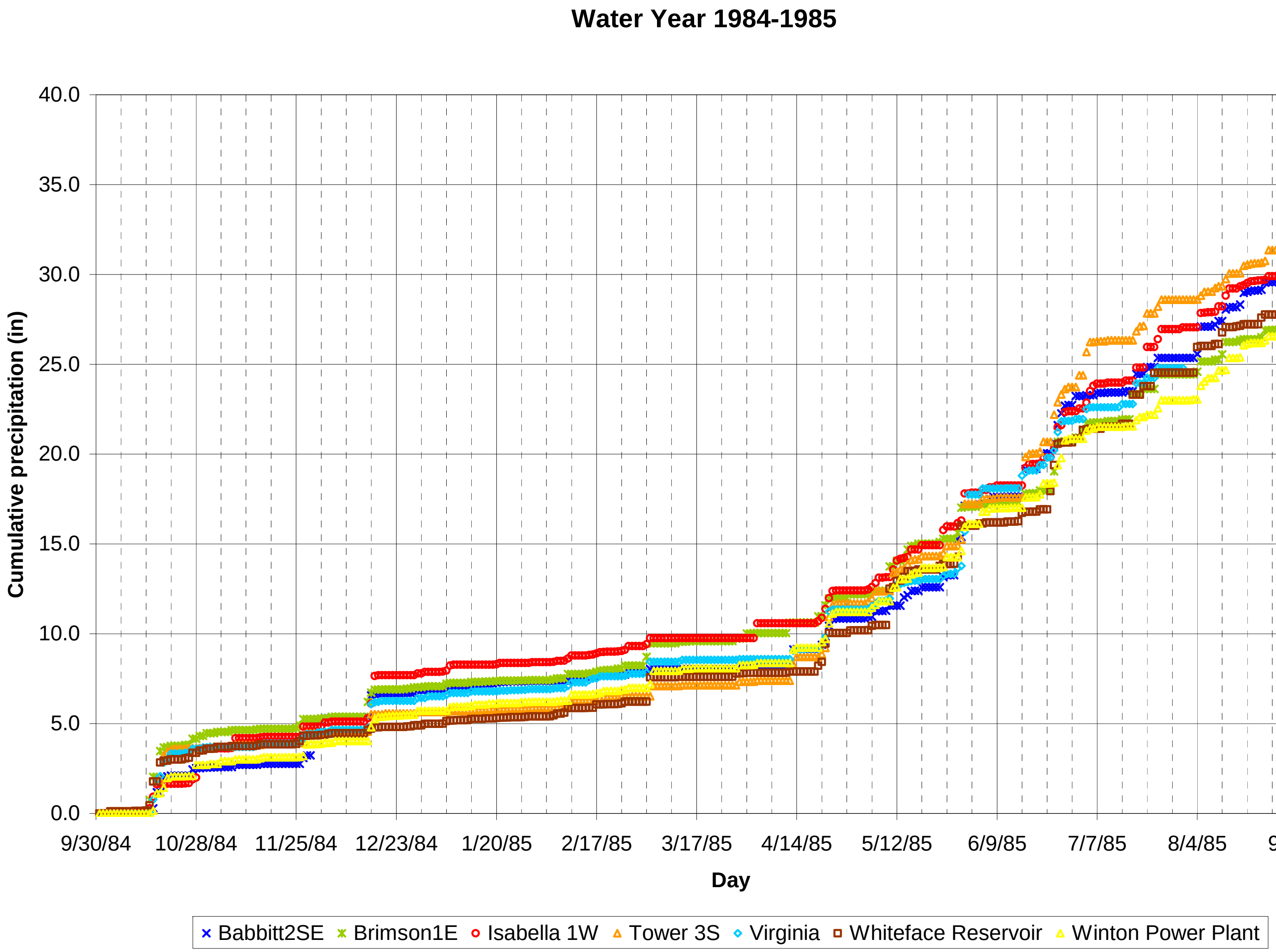


Figure 12: Daily precipitation at six weather stations located within a 30-mile radius from the study area for a 5-week period in water year 1984-1985

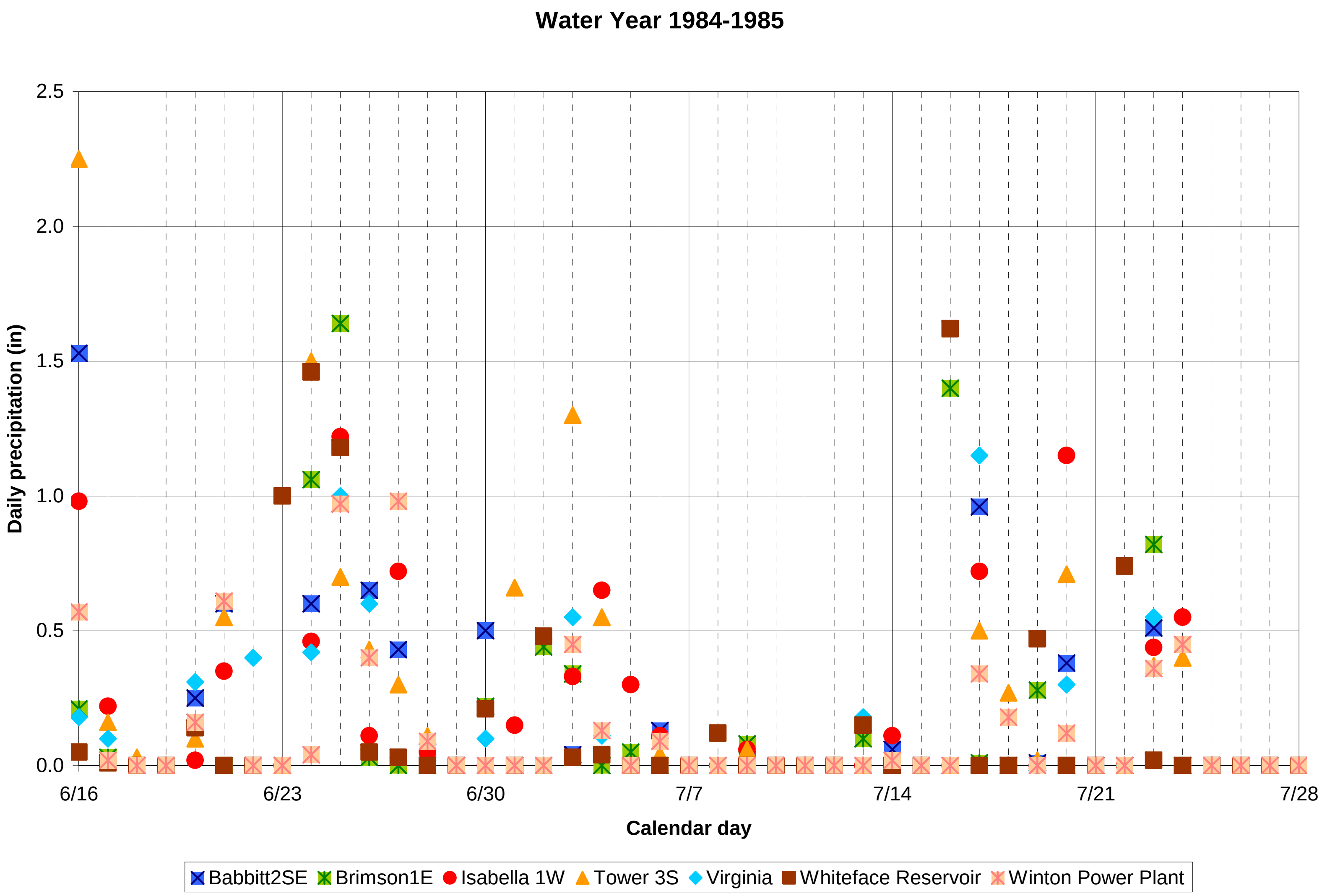


Figure 13: Spatially distributed precipitation in study area on August 7, 1982. Sub-watersheds depicted correspond to XP-SWMM model setup

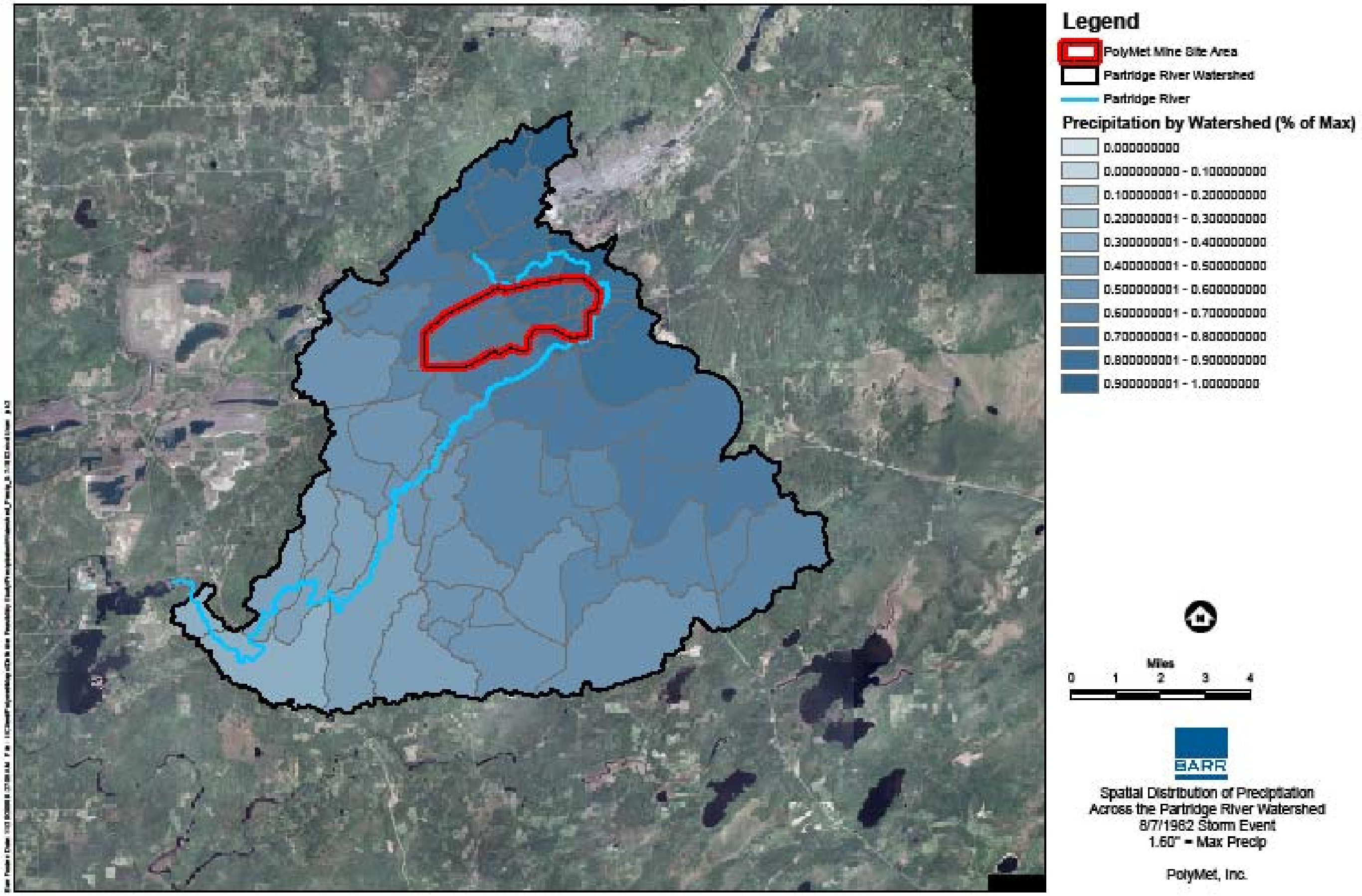
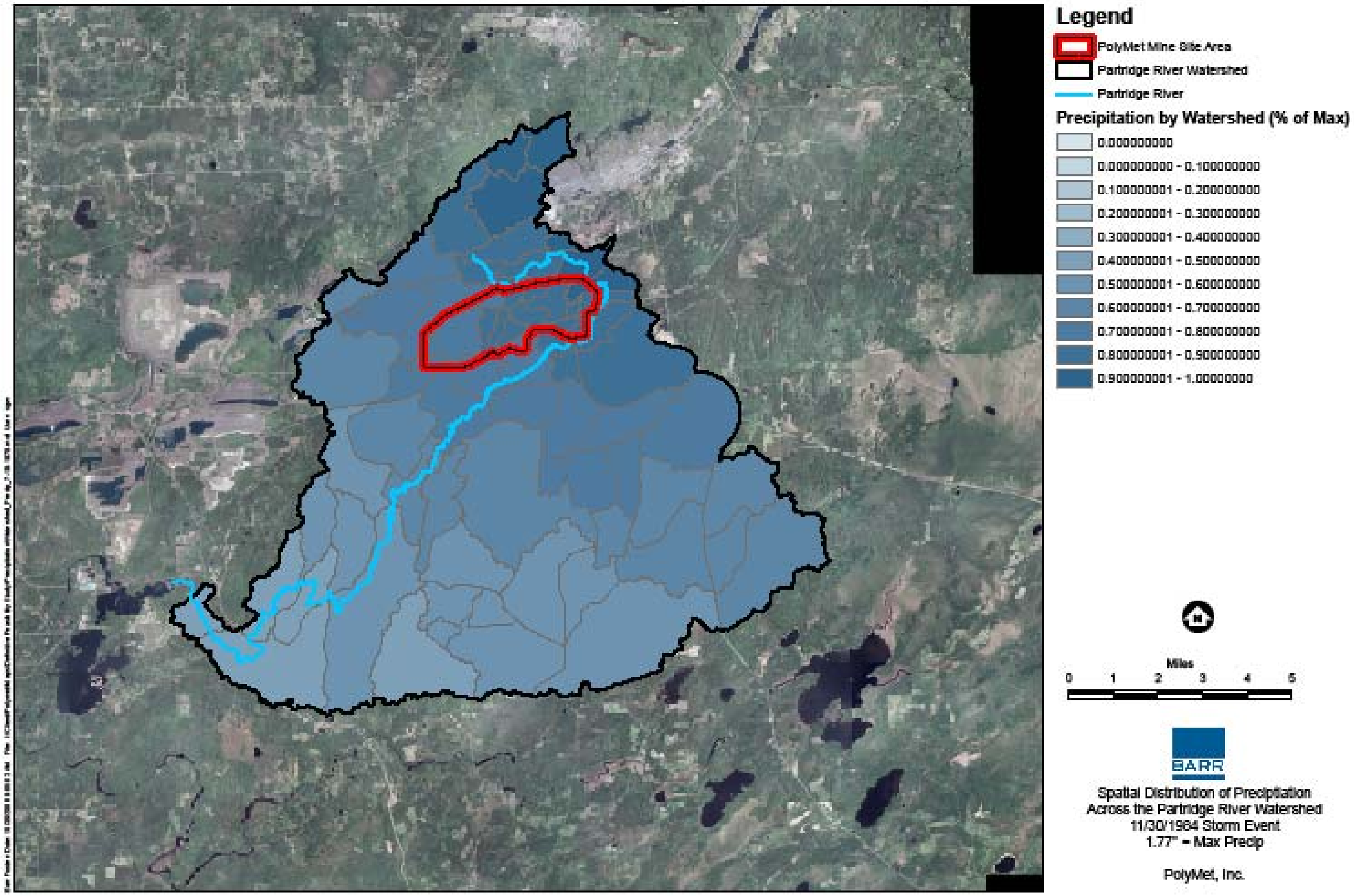


Figure 14: Spatially distributed precipitation in study area on November 30, 1984. Sub-watersheds depicted correspond to XP-SWMM model setup



Spatial Distribution of Precipitation
Across the Partridge River Watershed
11/30/1984 Storm Event
1.77" = Max Precip

PolyMet, Inc.

Figure 15: Spatially distributed precipitation in study area on August 23, 1988. Sub-watersheds depicted correspond to XP-SWMM model setup

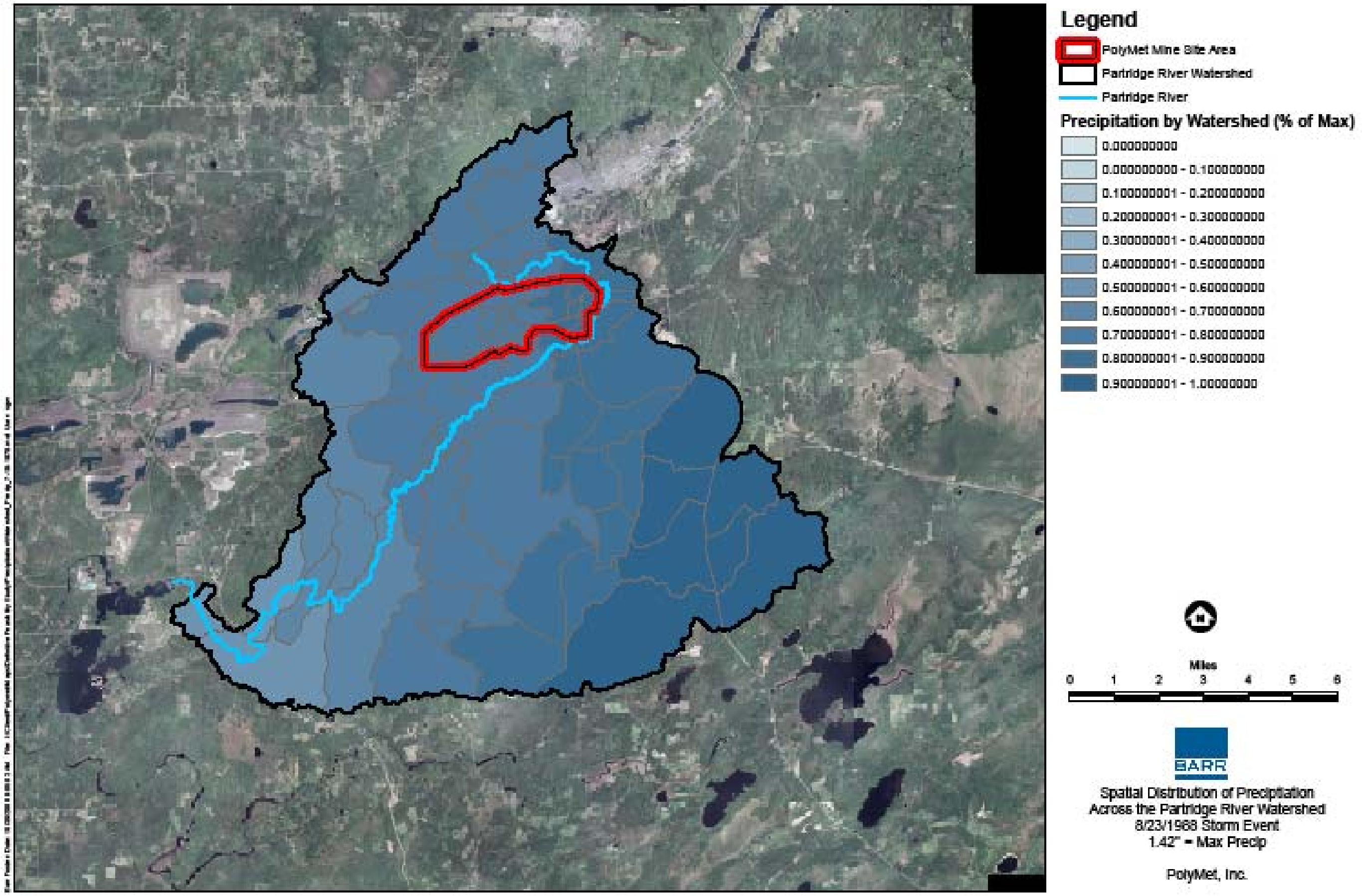


Figure 16: Watershed groups based on spatial precipitation

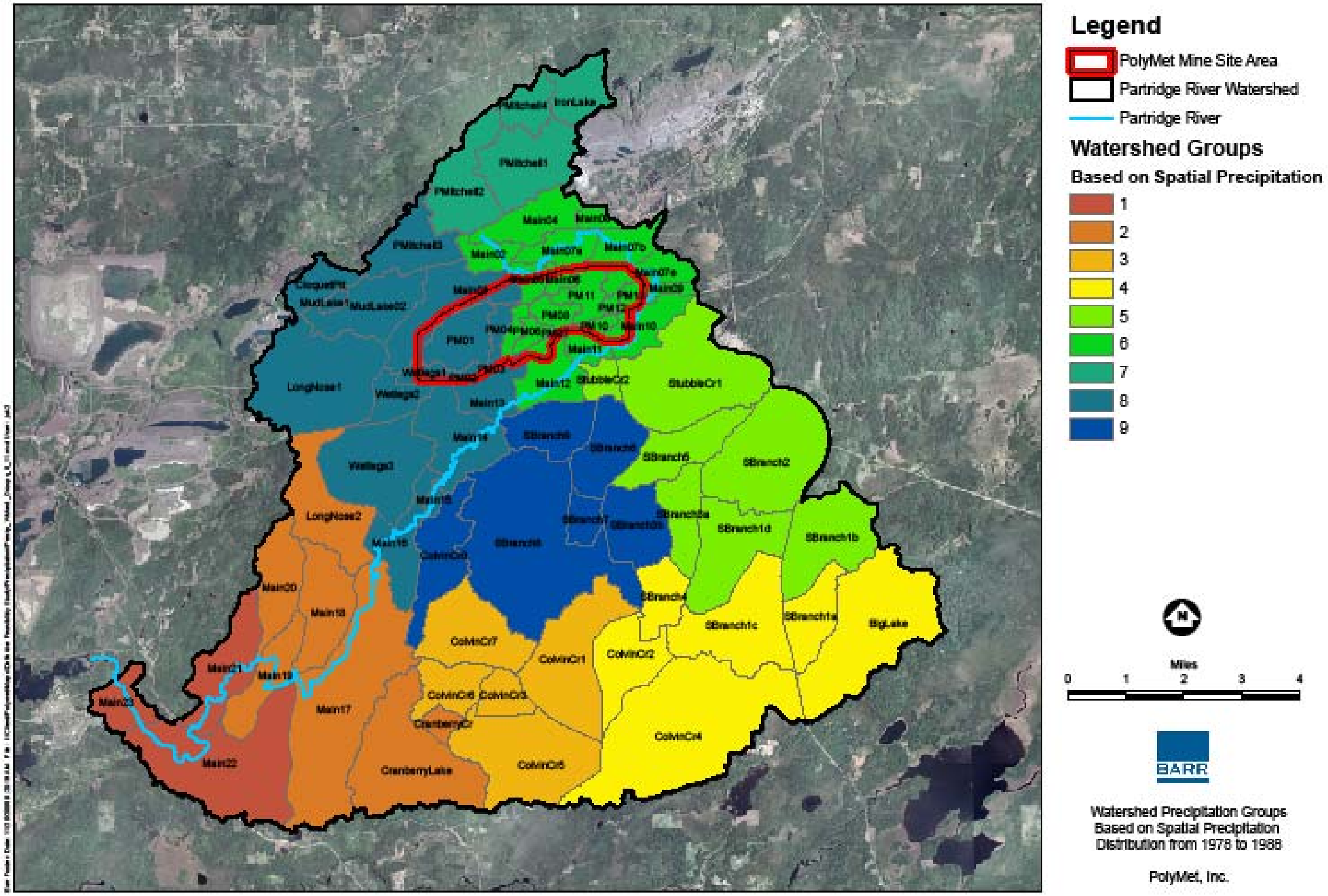


Figure 17: Schematic of groundwater module in XP-SWMM

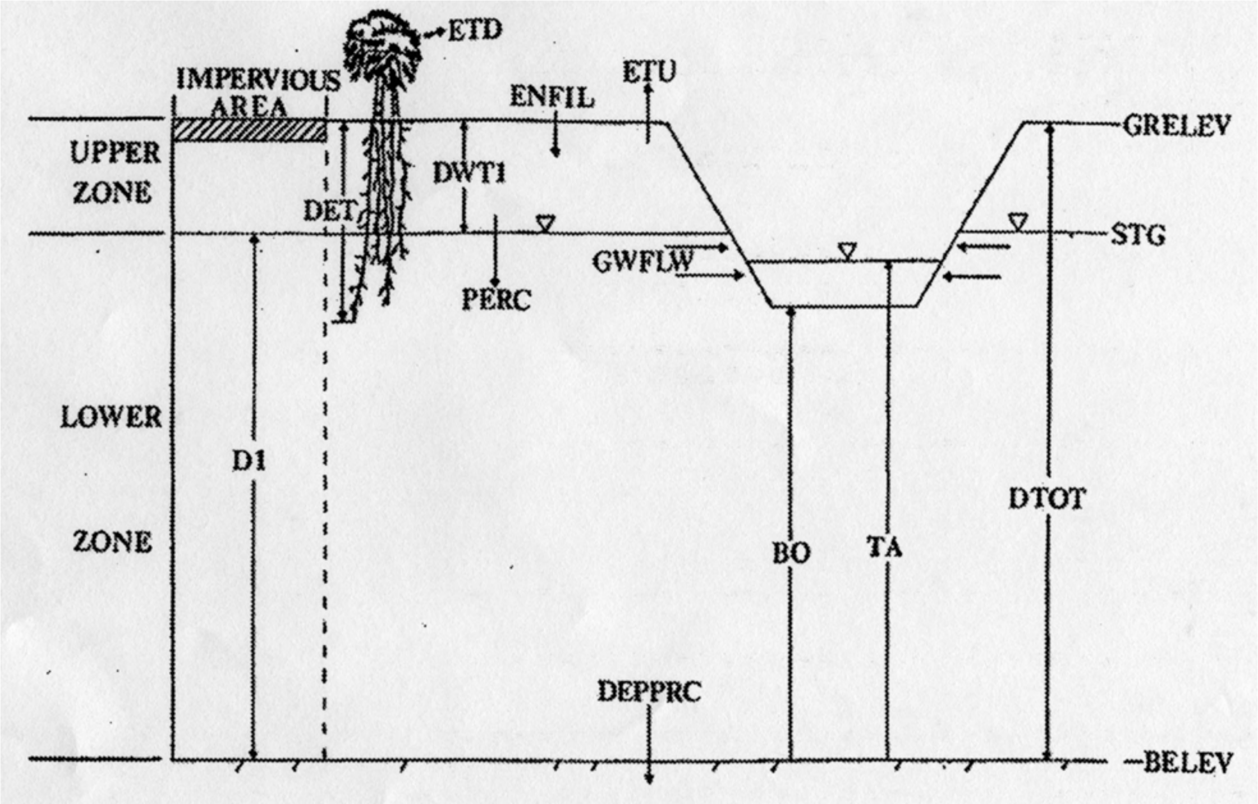


Figure 18: Location of USGS stream gaging stations

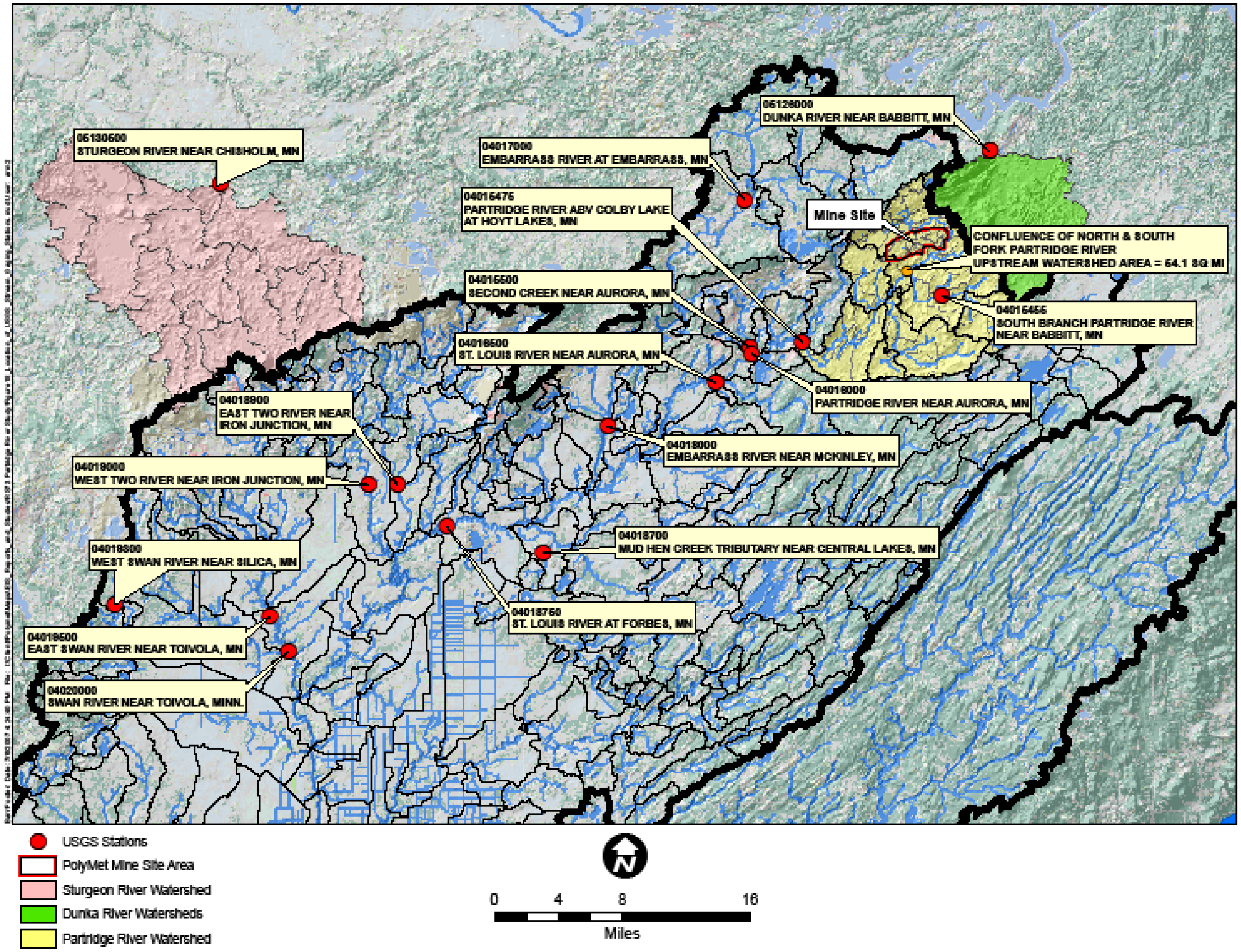
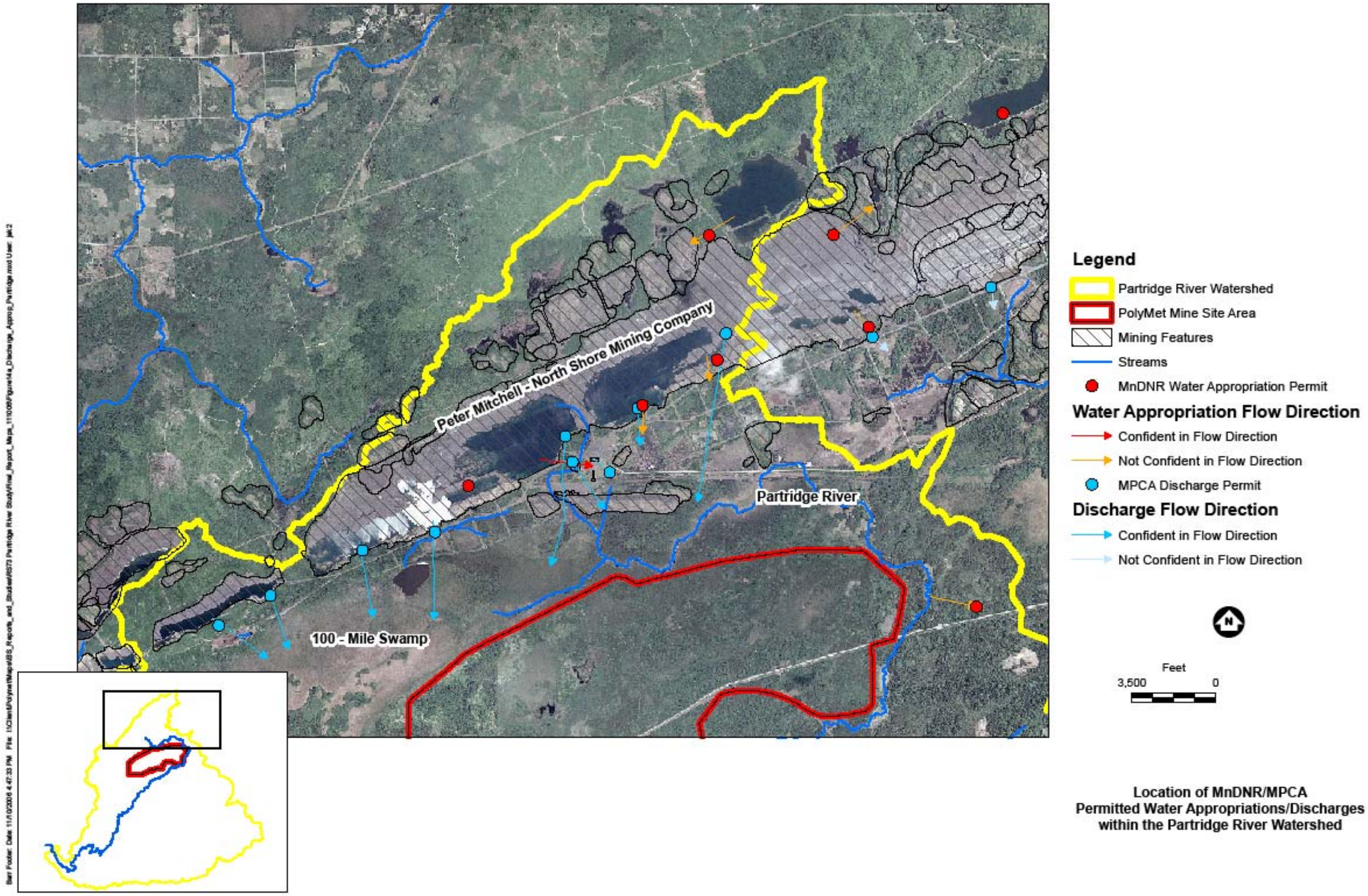


Figure 19: Approximate locations of water appropriations and discharges in the Partridge River watershed



Location of MnDNR/MPCA
Permitted Water Appropriations/Discharges
within the Partridge River Watershed

Figure 20: Approximate locations of water appropriations and discharges in the Dunka River watershed

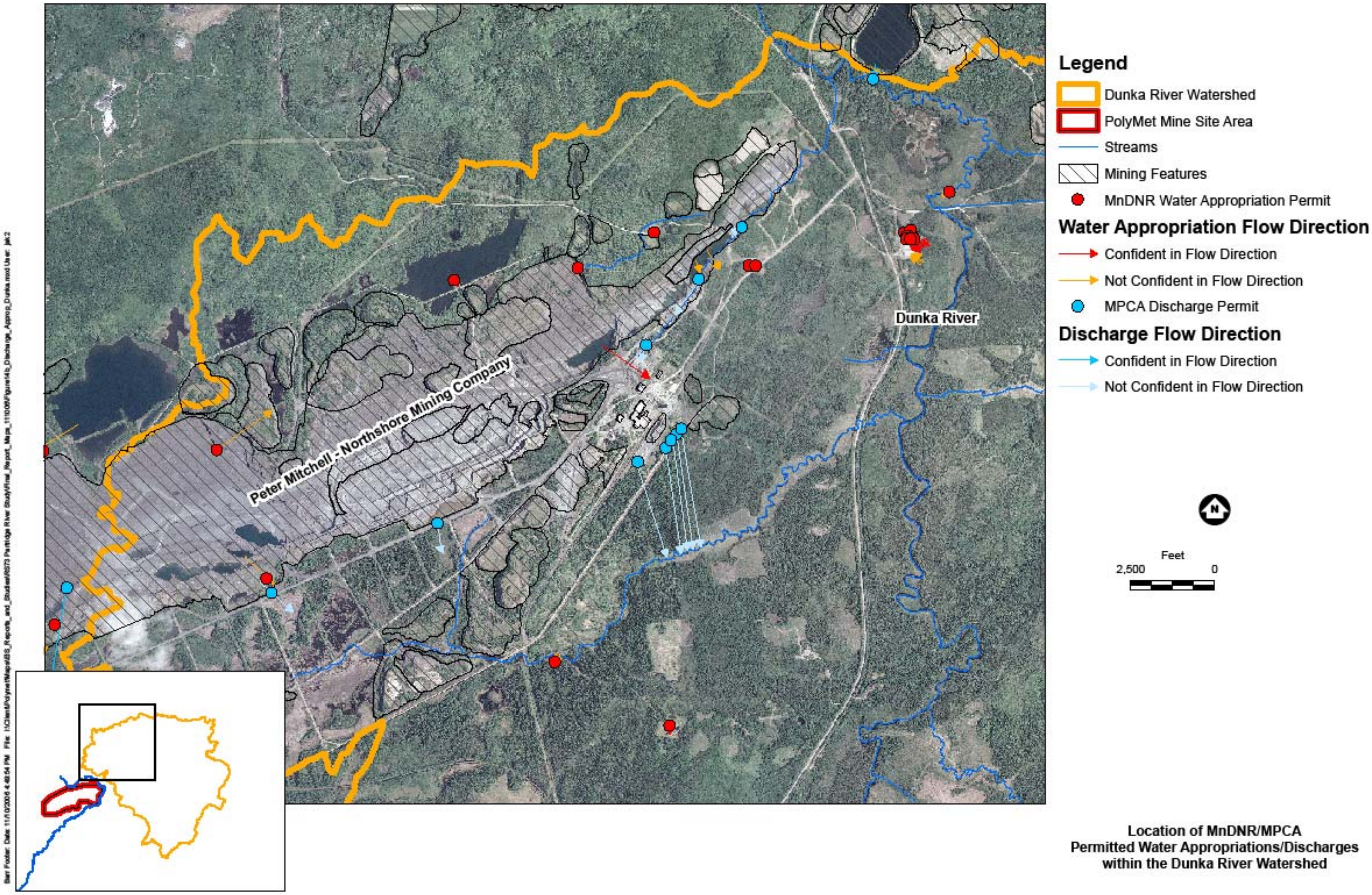


Figure 21: Approximate locations of water appropriations and discharges in the northeast sector of Second Creek watershed

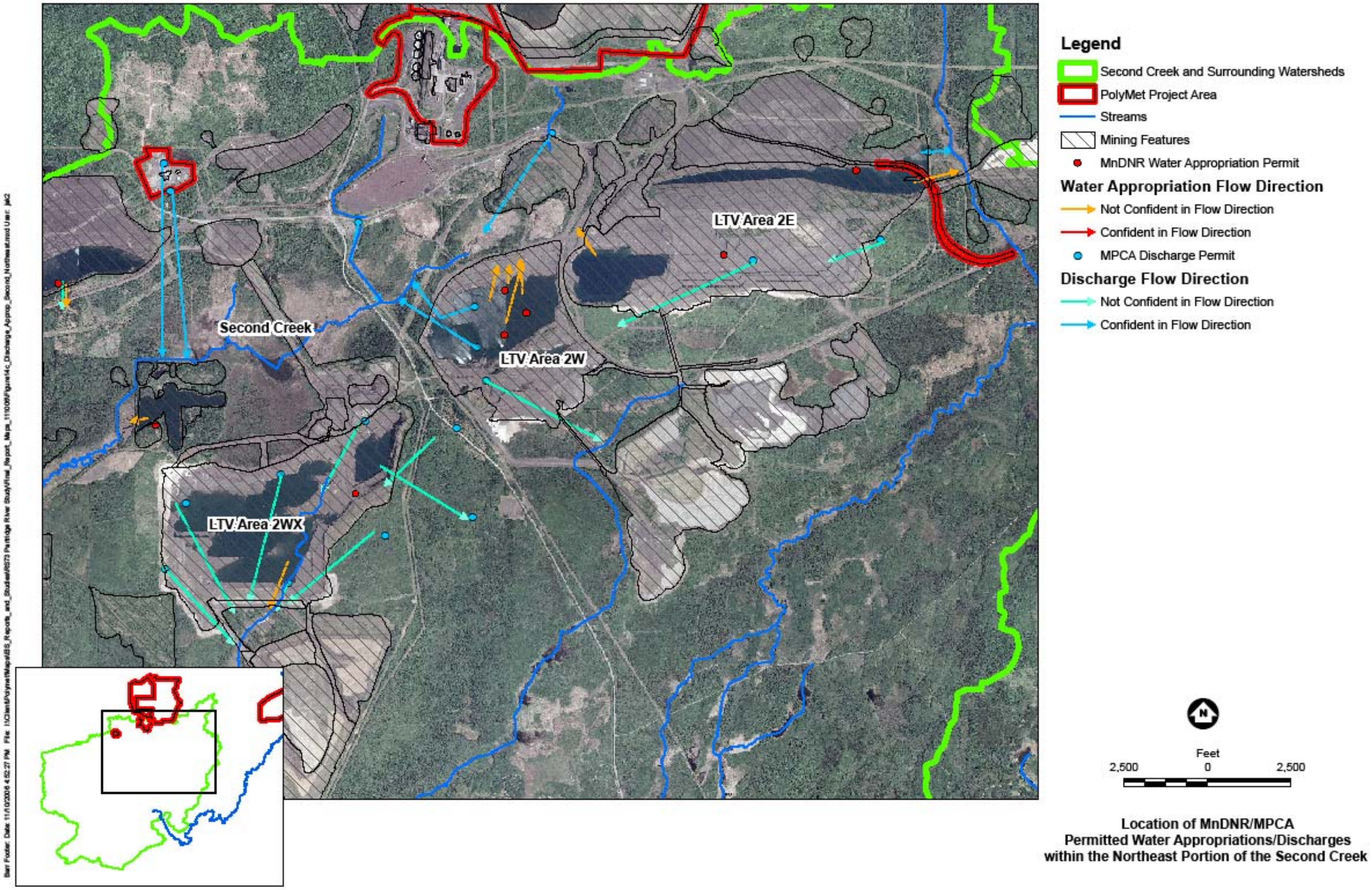


Figure 22: Approximate locations of water appropriations and discharges in the northwest sector of Second Creek watershed

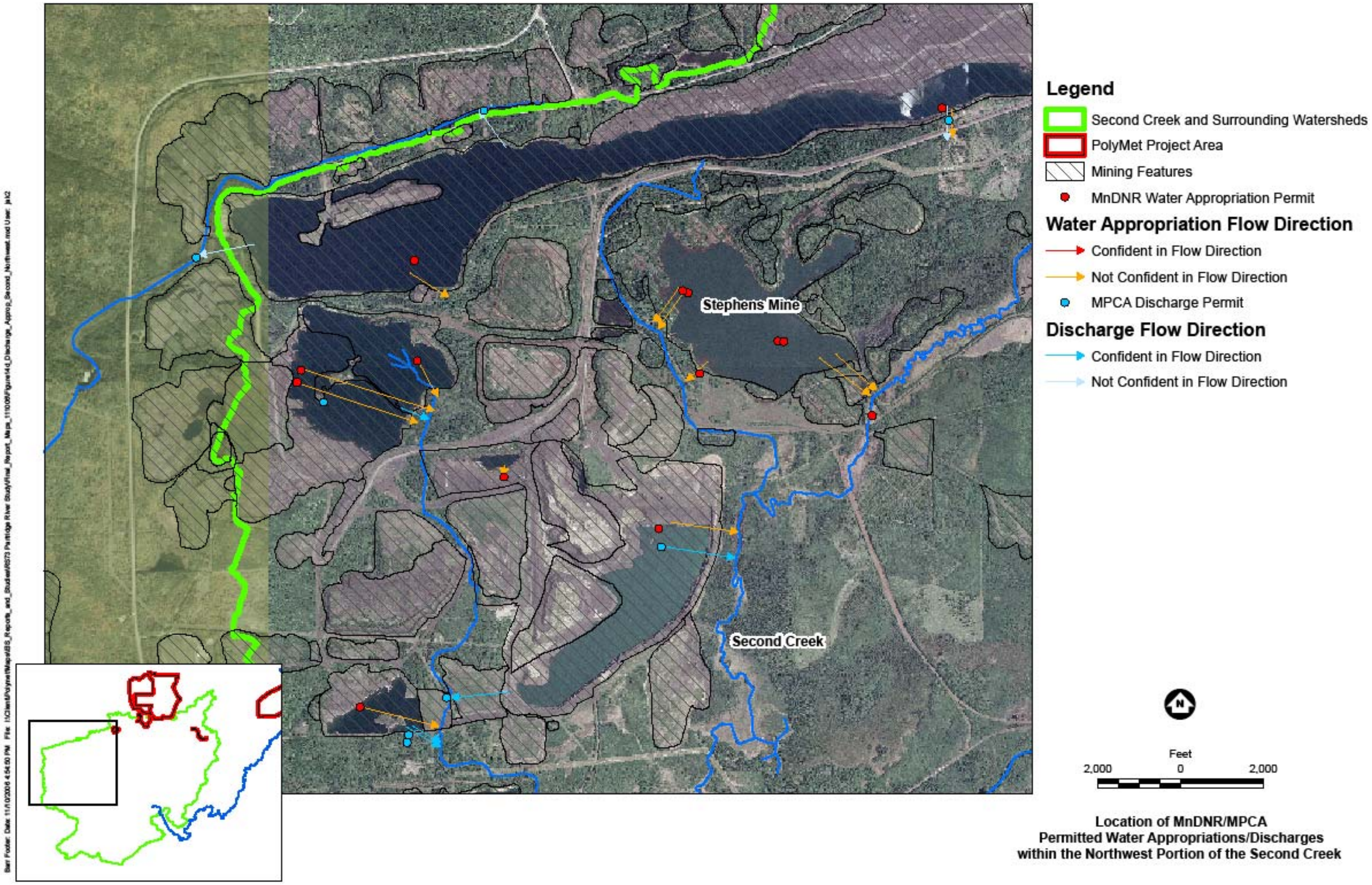


Figure 23: Runoff yield (cubic feet per second per square mile of catchment area -cfs/sq mi-) at USGS stream gaging stations

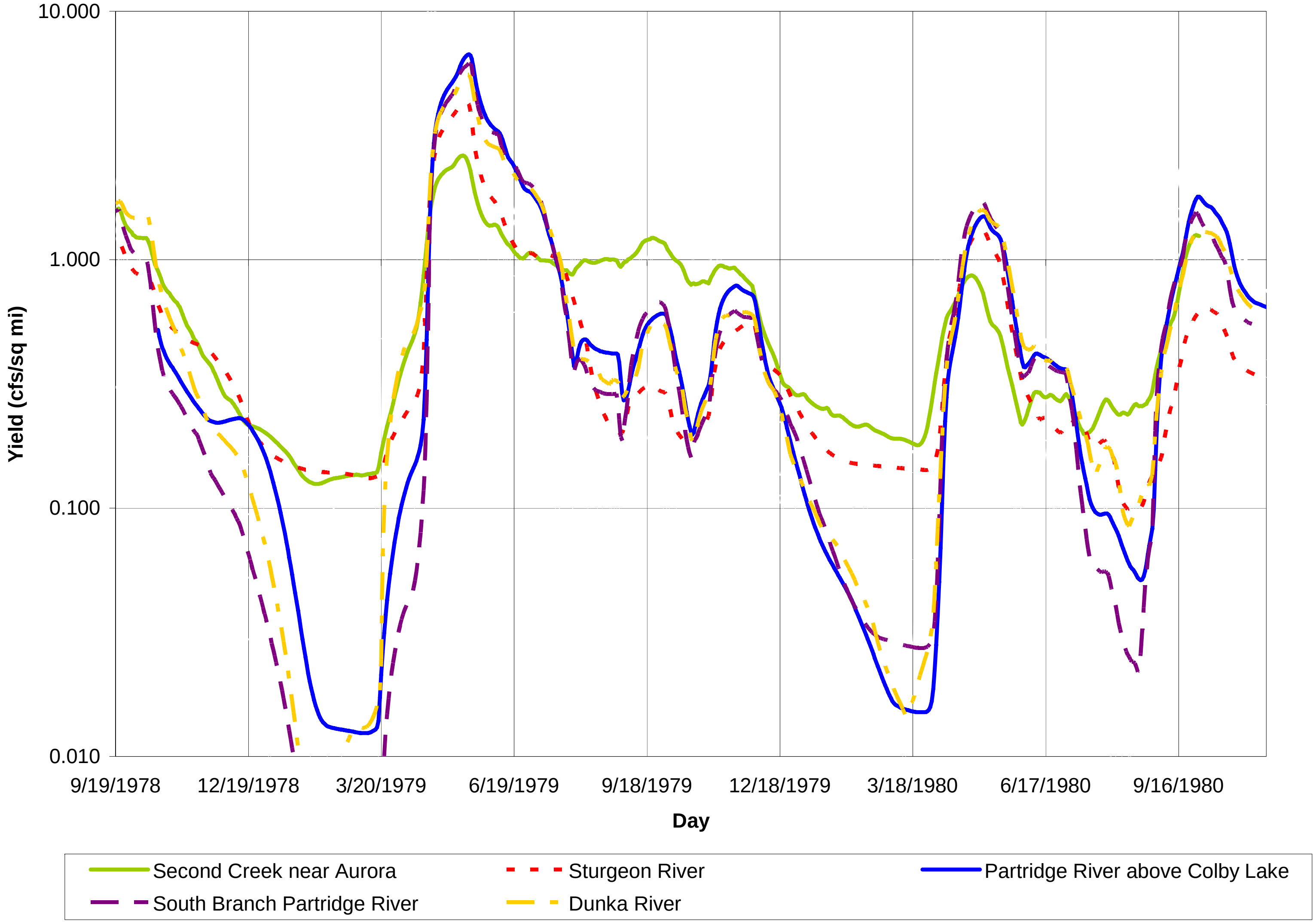


Figure 24: Sensitivity analysis of groundwater parameter “Initial Depth of the Unsaturated Zone”

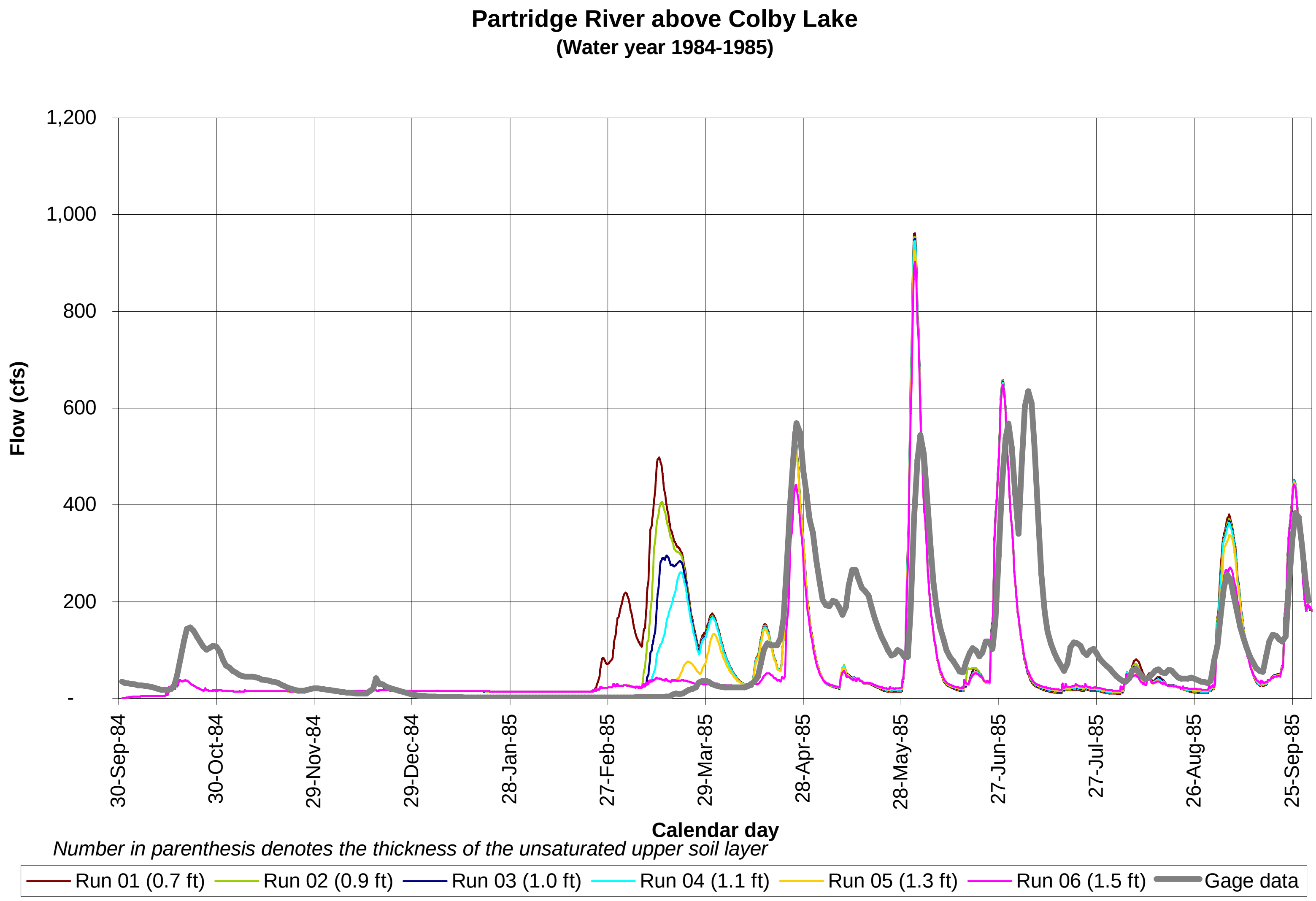


Figure 25: Sensitivity analysis of groundwater parameter “Fraction of Evapotranspiration to the Upper Zone”

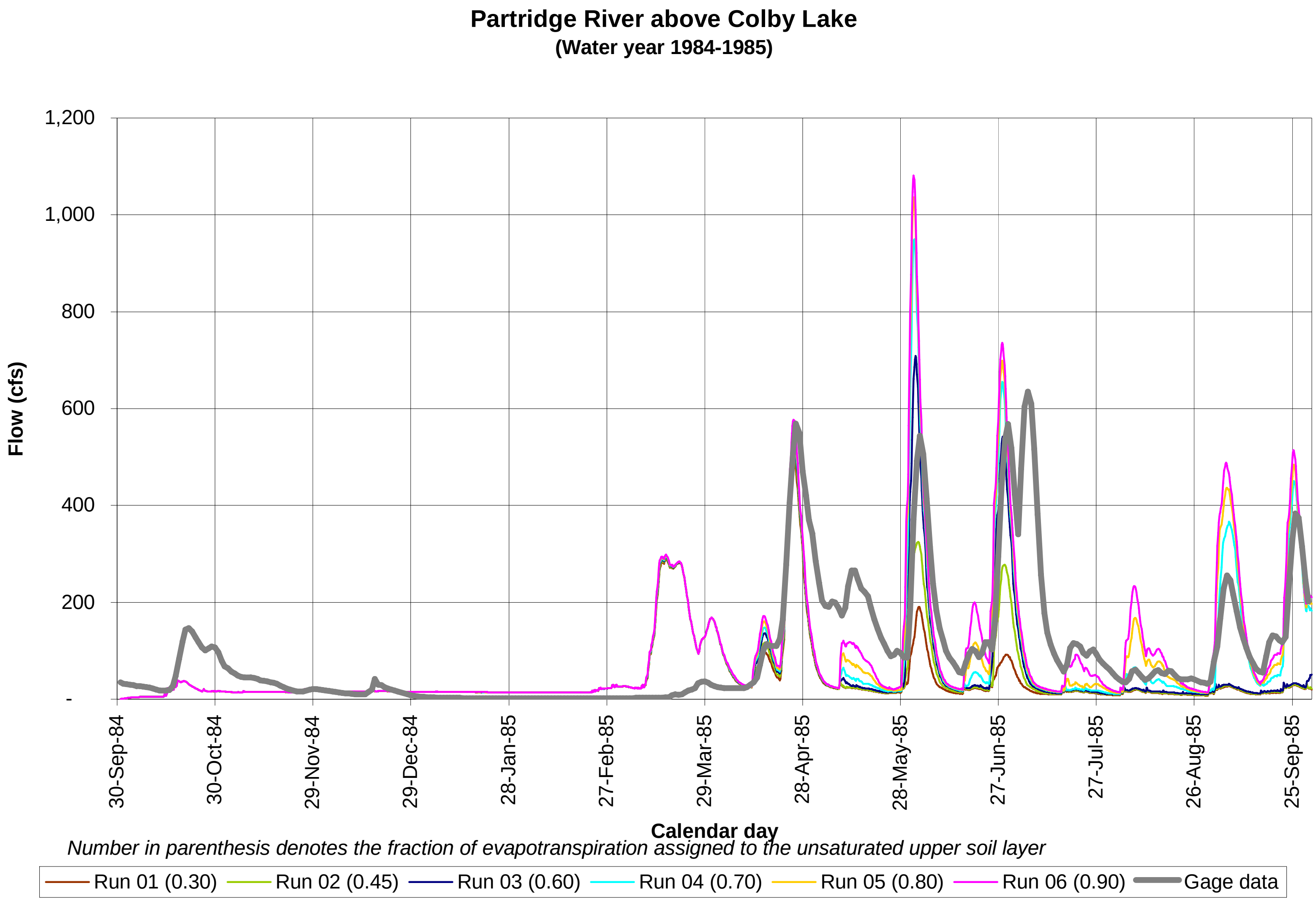
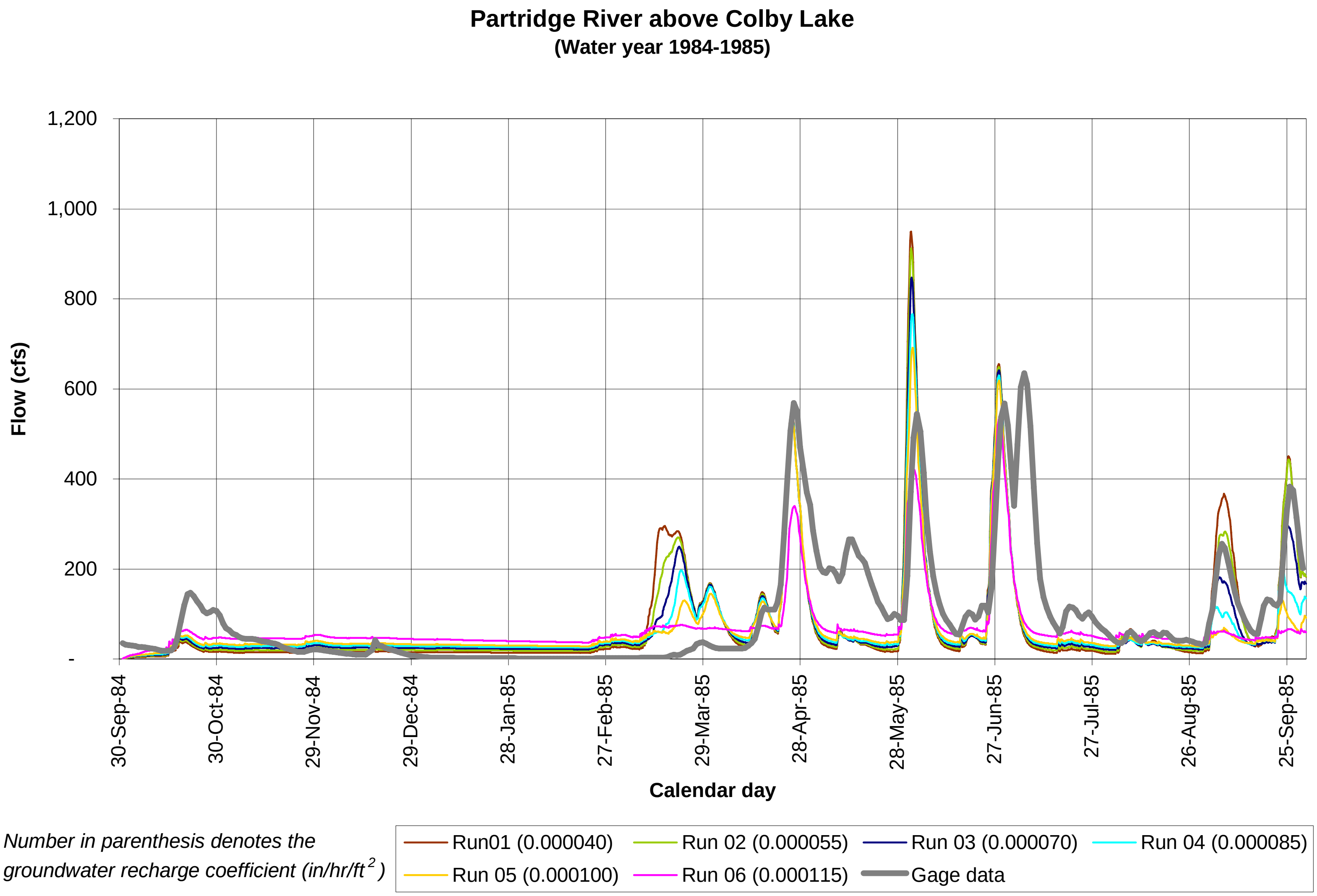


Figure 26: Sensitivity analysis of groundwater parameter “Groundwater Outflow Coefficient”



Number in parenthesis denotes the groundwater recharge coefficient (in/hr/ft^2)

Figure 27: Sensitivity analysis of snowmelt parameter “Snowmelt Base Temperature”

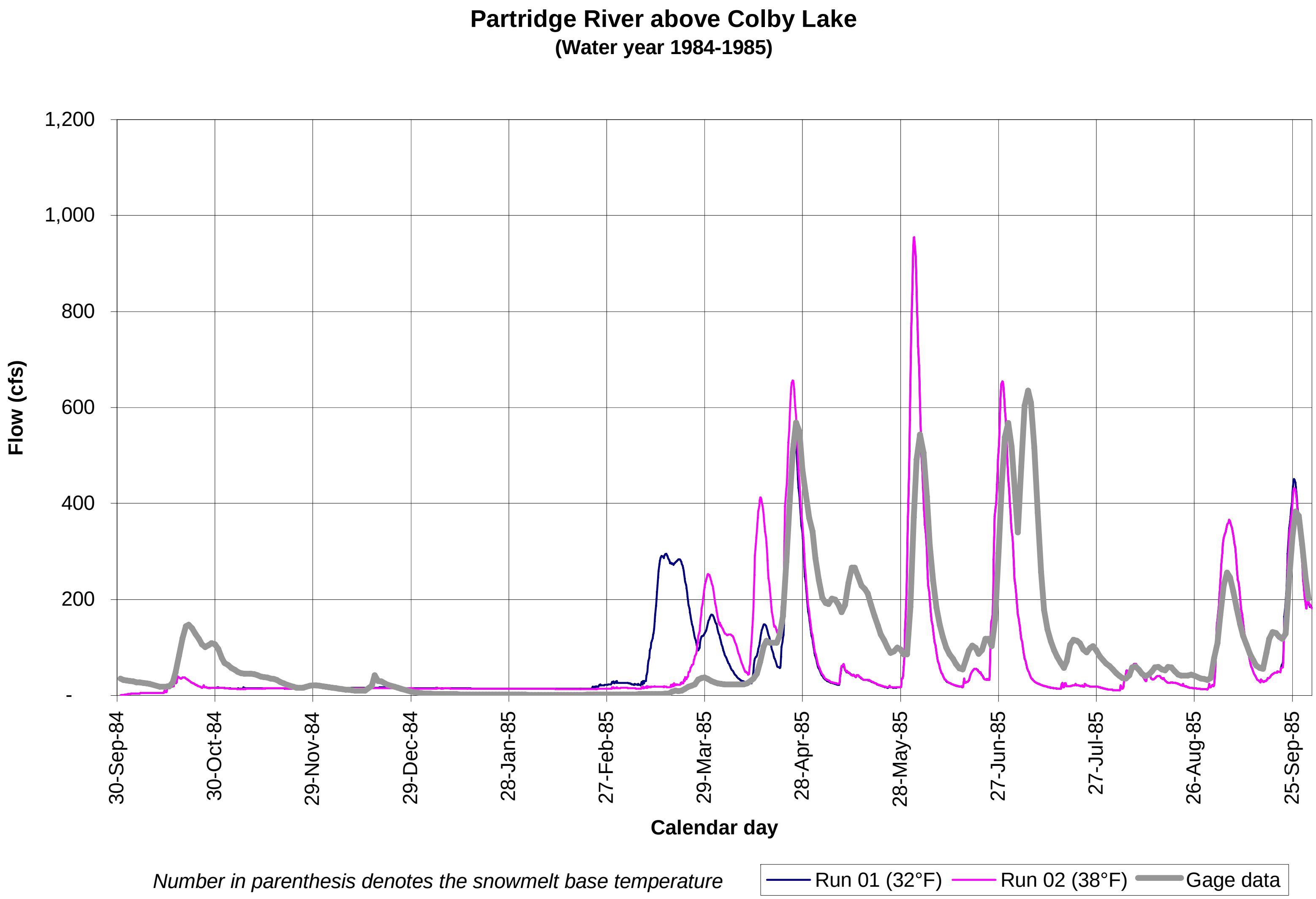
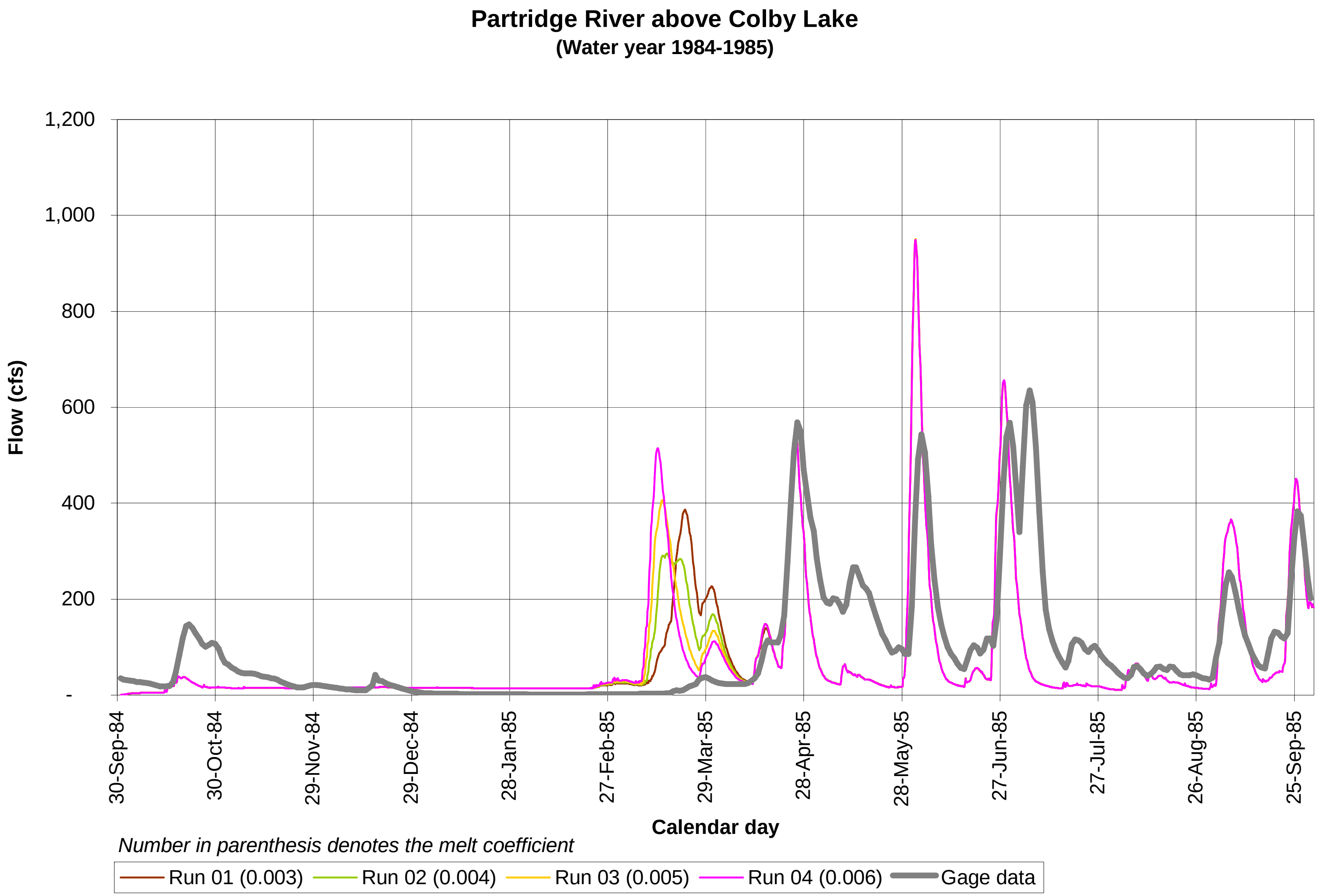


Figure 28: Sensitivity analysis of snowmelt parameter "June Melt Coefficient"



Number in parenthesis denotes the melt coefficient

Figure 29: Sensitivity analysis of snowmelt parameter “Snow Gage Correction Factor”

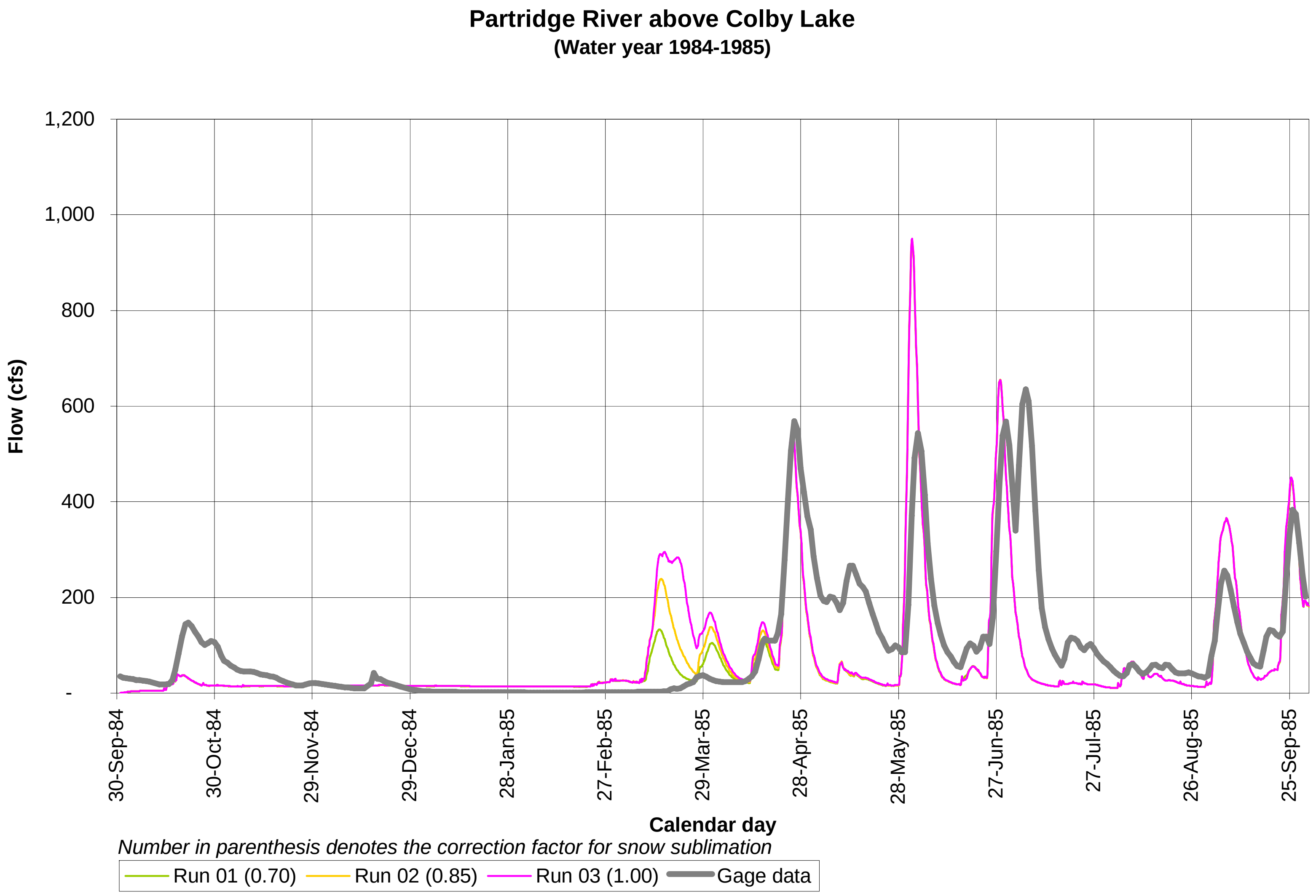


Figure 30: Comparison of modeled flows with calibrated XP-SWMM and gage data for watershed of Partridge River above Colby Lake (outlet of study area), water year 1984-1985

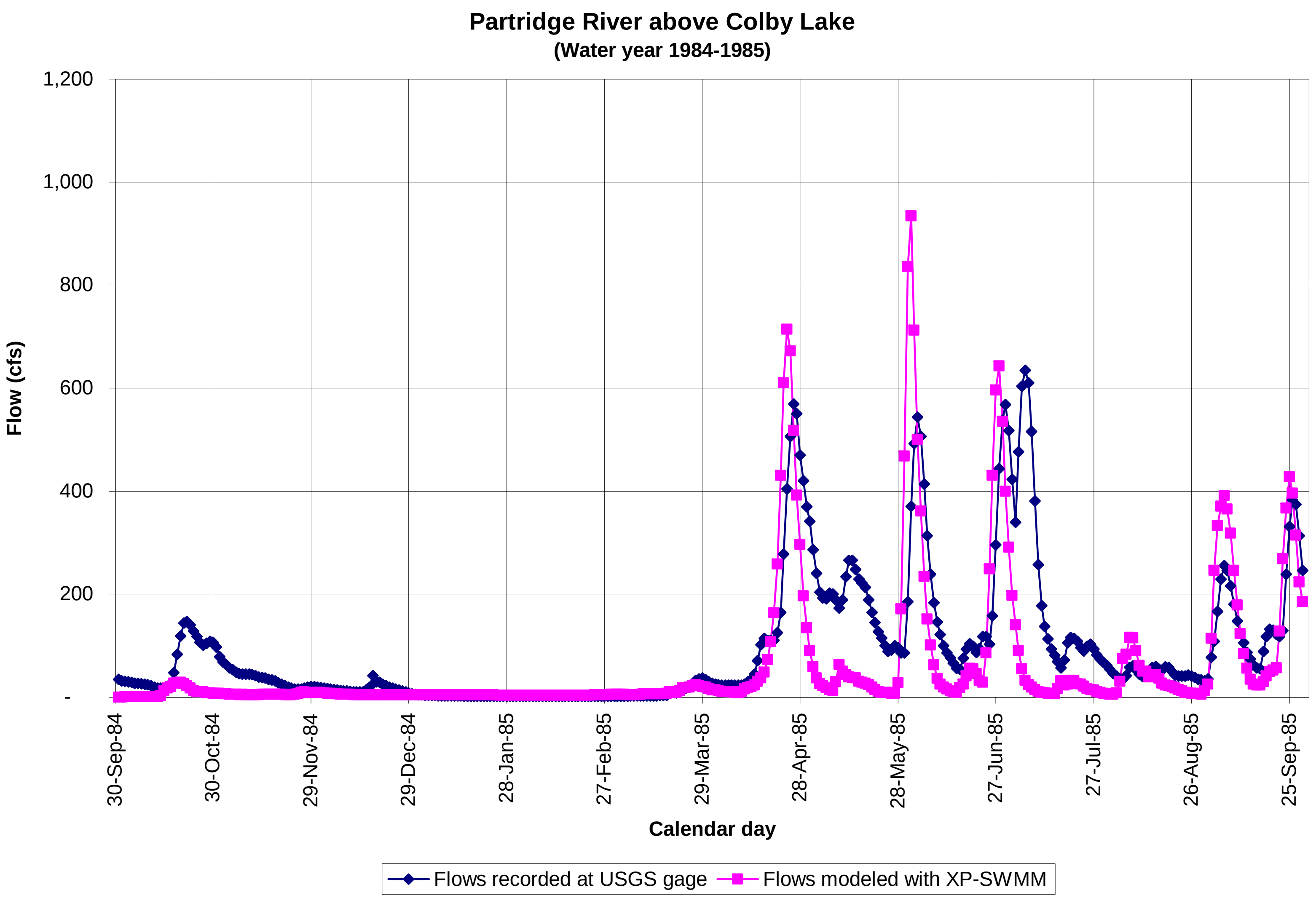


Figure 31: Comparison of model flows with XP-SWMM and gage data for watershed of Partridge River above Colby Lake (outlet of study area), validation period 1978-1988

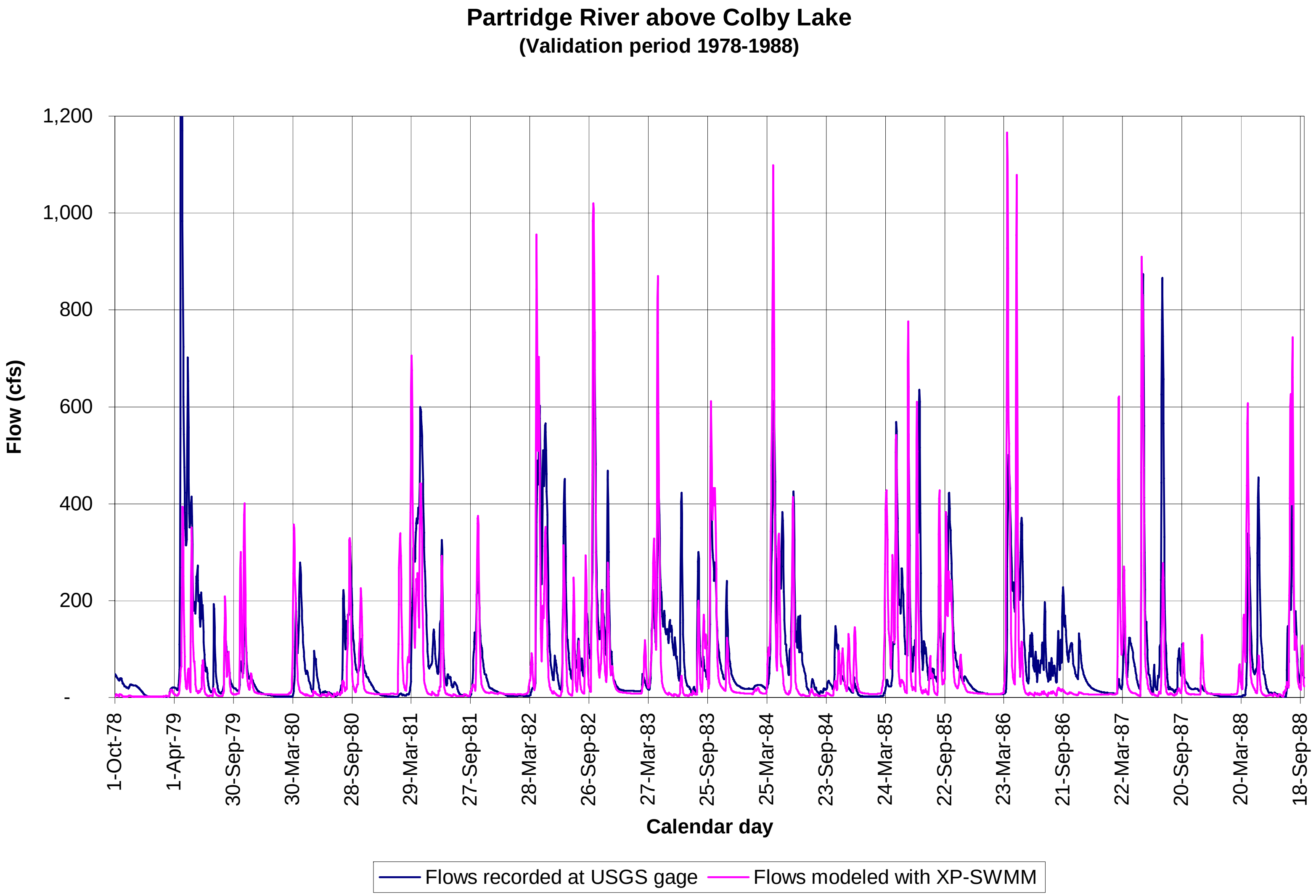
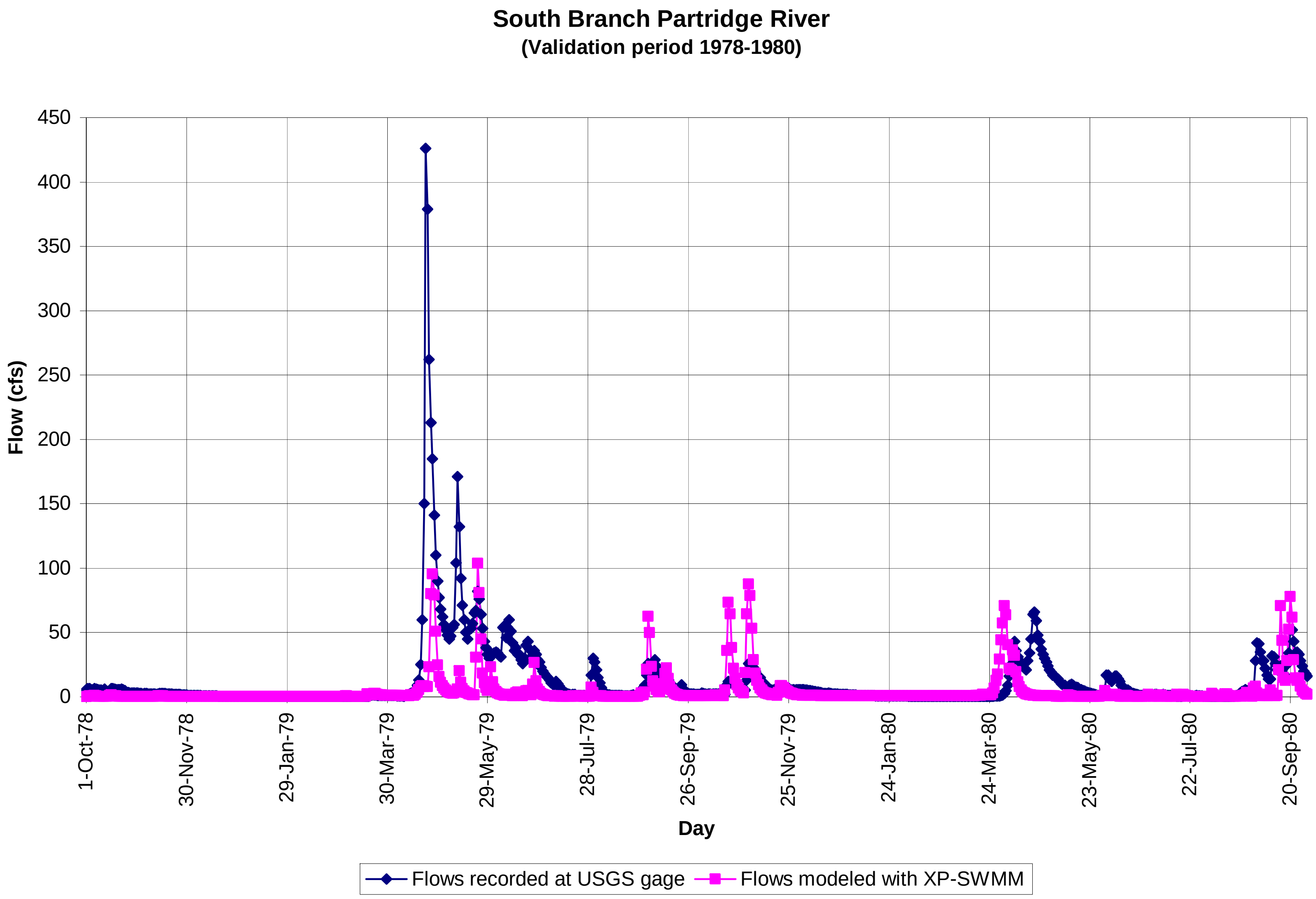


Figure 32: Comparison of model flows with XP-SWMM and gage data for watershed of South Branch Partridge River, validation period 1978-1980



Appendices

Appendix A

Barr Memorandum on “USGS gaging station #04015475 – Watershed area”

BARR

External Memorandum

To: Mike Liljegren, John Adams; MnDNR – Jim Kunkel; Knight Piesold

From: Miguel Wong, Nancy Johnson Dent

Subject: USGS gaging station # 04015475 – Watershed area

Date: July 19, 2007

This memo has been prepared in response to a request by the Minnesota Department of Natural Resources (MnDNR) and Knight Piesold during the meeting held in Barr's Minneapolis Office on July 9, 2007.

A telephone inquiry on July 13, 2007 with Chris Sanocki, a hydrologist with the Minnesota office of the USGS resulted in the following (for more details, see Telephone Memo attached):

- The watershed area of USGS gaging station # 04015475 – Partridge River above Colby Lake was estimated at 104.5 square miles based on watersheds delineated from 1:24,000 scale USGS quad maps. Mr. Sanocki is not certain why a drainage area of 106 square miles and a contributing area of 100 square miles are listed at the USGS National Water Information System (NWIS).
- The catchment area used in the hydrologic/hydraulic model of the Partridge River watershed (which corresponds to the watershed area of USGS gaging station # 04015475 – Partridge River above Colby Lake) is 103.4 square miles, based on watersheds delineated as part of the Lakeshed Project completed by MnDNR staff using custom software developed in ESRI ArcGIS on a hydrologically corrected Digital Elevation Model (DEM) of the St. Louis River watershed. (For more details, see RS73A report.)

Barr recommends using a catchment area of 103.4 square miles, as this value is supported by the input topographic data used to build the hydrologic/hydraulic model.

Telephone Memo

Date: 7/13/07
Time: 1:20 PM

Greg Williams of Barr Engineering Company

☒ placed a call to ☐ received a call from ☐ received a voice mail from ☐ left message/voice mail to

Name	**Position**	**Company**	**Telephone**
Chris Sanocki	Hydrologist	US Geological Survey	763-783-3151
			- -
			- -
			- -

Re: **Project Name:** PolyMet NorthMet Project **Project Number:** 23 / 69 - 862 015 022R

Notes: This inquiry arose from a discrepancy regarding the size of the watershed of the Partridge River watershed above Colby Lake. The XP-SWMM model used a drainage area of 103.4 square miles. The historical flow record provided by the USGS National Water Information System (NWIS) lists a drainage area of 106 square miles and a contributing drainage area of 100 square miles for the same watershed (USGS gage 04015475 – Partridge River above Colby Lake).

Chris Sanocki, a hydrologist with the Minnesota office of the USGS, took a look at the watershed in question. Mr. Sanocki calculated a drainage area of ~104.5 square miles, based on watersheds delineated from 1:24,000 scale USGS quad maps. Mr. Sanocki was not certain where the 106 square mile area listed at the NWIS came from, suggesting that the calculation may have been performed prior to the development of automated methods. Mr. Sanocki was confident that the area of ~104.5 square miles is accurate, and he did not believe that the contributing area should be any less than the drainage area. Mr. Sanocki was unsure why the NWIS record lists a smaller value for the contributing area or how that area had been calculated.

The Partridge River watershed used in the XP-SWMM model differs slightly from that calculated by Mr. Sanocki. The area used in the XP-SWMM model is based on a preliminary version of the Minnesota Lake Watershed Delineation (Lakeshed) Project delineation of the St. Louis River watershed by MnDNR staff, which represents an improved version of the watershed boundaries (see RS73A report).

Appendix B

Barr Memorandum on “PolyMet Mine Site – Partridge River Model Low Flow Statistics”

External
Memorandum

To: Mike Liljegren, John Adams, Jim Solstad; MnDNR

From: Miguel Wong, Nancy Johnson Dent

Subject: PolyMet Mine Site – Partridge River Model Low Flow Statistics

Date: July 5, 2007

The purpose of this memo is to provide a measure of the "goodness-of-fit" during periods of low flows from the XP-SWMM model results that was developed and calibrated for the Partridge River watershed (see RS73A and RS73B documents). The analysis was requested by the Minnesota Department of Natural Resources (MnDNR) during the meeting held in Barr's Duluth Office on June 7, 2007.

The emphasis on low flows is because it has become apparent that low flows are critical for several of the issues under investigation for PolyMet's Environmental Impact Study. For instance, periods of low flow may portray the conditions under which potential impacts of the NorthMet Project on the water quality of the Partridge River and aquatic environment supported by this water course are more significant.

While measures of goodness-of-fit were provided in RS73A, the statistics presented were computed for the entire period of simulation 1978-1988 for which gaged flow data in the Partridge River was available. These statistics were computed as well for each water year between 1978 and 1988. However, the measures of goodness-of-fit that were reported in RS73A (the deviation of volume runoff, and the coefficient of efficiency) tend to be dominated by peak flow events. Therefore, these statistics would not be an appropriate measure of the model performance during periods of low flows. The following paragraphs explain this in more detail.

The deviation of volume runoff D_v, which is defined as

$$D_v = 100 \times \left(\frac{V_{obs} - V_{mod}}{V_{obs}} \right) \tag{1}$$

becomes overly sensitive to small deviations in flow under low flow conditions. The observed average annual 30-day low flow in the Partridge River is 5 cubic feet per second, which is equivalent to a total runoff over the 30-day period of 0.05 inches over a 67,000 acre watershed. With flows this small, a discrepancy between modeled and observed runoff of just 0.0018 inches per day would result in a 100% volume deviation.

The coefficient of efficiency *E*, which is defined as

$$E = 1 - \left[\frac{\sum_{i=1}^{N} \left(Q_{obs}^{i} - Q_{mod}^{i} \right)^2}{\sum_{i=1}^{N} \left(Q_{obs}^{i} - \overline{Q} \right)^2} \right] \tag{2}$$

becomes a very large negative number under low flow conditions when the variance of the observed flows (the denominator on the right side of equation (2)) is very small; such small variance is expected during periods of low flows in the Partridge River as low flows are dominated by groundwater recharge that is anticipated to have small variability.

We propose to use a dimensionless version of the Root Mean Square Error *RMSE'*, which is defined as

$$RMSE' = \frac{1}{\overline{Q}} \sqrt{\frac{\sum_{i=1}^{N} \left(Q_{obs}^{i} - Q_{mod}^{i} \right)^2}{N}} \tag{3}$$

to measure the goodness-of-fit for the 30-day low flow period. A 30-day period was selected as an appropriate time scale to represent low flow conditions because it is long enough to avoid comparing instantaneous observed and modeled flows (as indicated in RS73A, the XP-SWMM model of the Partridge River watershed “is not intended to predict instantaneous flow values, but to provide estimates of overall trends in the flow pattern as the mining project is implemented”) and it is short enough to avoid bias due to flow increases associated with unexpected, seldom events in the middle of the winter. Evaluation of the entire winter low flow period of 90-150 days would likely correspond to greater watershed flows because of the chance of intermittent snowmelt and occasional rainfall-over-snow events, therefore would not depict the greatest impact of the Mine Site on reduction of Partridge River low flows.

The *RMSE'* is a direct measure of the error in modeled daily mean flows and is scaled by the mean observed 30-day low flow for a given water year. No references on typical ranges could be found, therefore values in the order of 1.62 or less were considered acceptable as this would represent a discrepancy between observed and modeled flows of less than 0.10 inches in runoff over the 30-day period. The resulting values for the *RMSE'* are given in the table below.

Period	30-day low flow (cfs)[1]	*RMSE'*
1978-1979	1.3	0.68
1979-1980	1.6	3.05
1980-1981	1.6	6.65
1981-1982	2.9	0.32
1982-1983	13.3	0.45
1983-1984	12.5	0.75
1984-1985	1.6	3.08
1985-1986	6.5	0.04
1986-1987	8.7	0.27
1987-1988	1.2	1.55

[1] This is the mean flow over the 30-day low flow period for each water year.

The dimensionless Root Mean Squared Error for the entire period of simulation is 1.15, which is approximately equivalent to the mean 30-day flow, or more specifically denotes a discrepancy between observed and modeled flows of 0.06 inches in runoff over the average 30-day period of low flows. The table above shows that 7 out of the 10 years of simulations comply with the criterion indicated above.

Appendix C

Copy of SWMM Manual:

Appendix II – SWMM Snowmelt Routines

Appendix X – Subsurface Flow Routing in Runoff Block

Appendix II
SWMM Snowmelt Routines

Introduction

Snowmelt is an additional mechanism by which urban runoff may be generated. Although flow rates are typically low, they may be sustained over several days and remove a significant fraction of pollutants deposited during the winter. Rainfall events superimposed upon snowmelt baseflow may produce higher runoff peaks and volumes as well as add to the melt rate of the snow.

In the context of long term continuous simulation, runoff and pollutant loads are distributed quite differently in time between the cases when snowmelt is and is not simulated. The water and pollutant storage that occurs during winter months in colder estimates cannot be simulated without including snowmelt.

Several hydrologic models include snowmelt computations, e.g., Stanford Watershed Model (Crawford and Linsley, 1966), HSPF (Johanson et al., 1980), NWS (Anderson, 1973, 1976), STORM (Hydrologic Engineering Center, 1977; Roesner et al., 1974) and SSARR (Corps of Engineers, 1971). Of these examples, only HSPF and STORM include pollutant routing options. Useful summaries of snowmelt modeling techniques are available in texts by Fleming (1975), Eagleson (1970), Linsley et al. (1975), Viessman et al. (1977), and Gray (1970). All of these draw upon the classic work, *Snow Hydrology*, of the Corps of Engineers (1956).

As part of a broad program of testing and adaptation to Canadian conditions, a snowmelt routine was placed in SWMM for single event simulation by Proctor and Redfern, Ltd. and James F. MacLaren, Ltd., abbreviated PR-JFM (1976a, 1976b, 1977), during 1974-1976. The basic melt computations were based on routines developed by the U.S. National Weather Service, NWS (Anderson, 1973). The work herein has utilized the Canadian SWMM snowmelt routines as a starting point and has considerably augmented their capabilities as well as added the facility for snowmelt computations while running continuous SWMM. In addition, features have been added which aid in adapting the snowmelt process to urban conditions since most efforts in the past, except for STORM, have been aimed at simulation of spring melt in large river basins. The work of the National Weather Service (Anderson, 1973) has also been heavily utilized, especially for the extension to continuous simulation and the resulting inclusion of cold content, variable melt coefficients and areal depletion.

The following sections describe the methodology presently programmed in the SWMM Runoff Block. It is intended to aid in understanding the various input parameters required, computations performed, and the output produced.

Overview

Snow Depth

Throughout the program, all snow depths are treated as “depth of water equivalent” to avoid specification of the specific gravity of the snow pack which is highly variable with time. The specific gravity of new snow is of the order of 0.09; an 11:1 or 10:1 ratio of snow pack depth to water equivalent depth is often used as a rule of thumb. With time, the pack compresses until the specific gravity can be considerably greater, to 0.5 and above. In urban areas, lingering snow piles may resemble ice more than snow with specific gravities approaching 1.0. Although snow pack heat conduction and storage depend on specific gravity, sufficient accuracy may be obtained without using it. It is adequate to maintain continuity through the use of depth of water equivalent.

Most input parameters are in units of inches of water equivalent (in. w.e.). For all computations, conversions are made to feet of water equivalent.

Single Event Simulation

For most SWMM calculations, there is no functional distinction between single event and continuous simulation. However, for snowmelt calculations, the user can specify (through parameter ISNOW in Runoff Block data group B1) whether melt is to be treated in a single event or continuous form. For single event simulation, it is unnecessary to generate a long record of precipitation and temperature data. Snow quantities are input as initial depths (water equivalent) on subcatchments and as negative rainfall intensities on rainfall input data groups. Snowfall is generally keyed as negative precipitation on input files. Temperature data are read for each time step from line input. The air temperature time step is defined by parameter DTAIR on data group C5. (Other parameters are explained subsequently.)

During the simulation, melt is generated at each time step using a degree-day type equation during dry weather and Anderson’s NWS equation (1973) during rainfall periods. Specified, constant areas of each subcatchment are designated as snow covered. Melt, after routing through the remaining snow pack, is combined with rainfall to form the spatially weighted “effective rainfall” for overland flow routing.

Continuous Simulation

For continuous simulation, hourly precipitation depths from NWS magnetic tapes are utilized along with daily max-min temperatures from other NWS tapes. The latter are interpolated sinusoidally to produce the temperature value at the beginning of a time step, as explained in detail in the next subsection. If temperatures are below a dividing value (e.g., 32◇F), precipitation values are treated as snow and keyed with a negative sign. The interpolated temperatures are also used in the melt computations.

Melt is again generated using a degree-day type equation during dry weather and Anderson’s NWS equation during rainfall periods. In addition, a record of the cold content of the snow is maintained. Thus, before melt can occur, the pack must be “ripened,” that is, heated to a specified base temperature.

One partition of the urban subcatchment is the “normally bare impervious area.” This is intended to represent surfaces such as streets, parking lots and sidewalks which are subject to plowing or snow “redistribution”. The program includes this feature.

Following the practice of melt computations in natural basins, “areal depletion curves” describe the spatial extent of snow cover as the pack melts. For instance, shaded areas would be expected to retain a snow cover longer than exposed areas. Thus, the snow covered area of each subcatchment changes with time during continuous simulation.

Melt computations themselves proceed as in the single event simulation, except that the degree-day melt coefficients vary sinusoidally, from a maximum on June 21 to a minimum on December 21.

Pollutant Simulation

Pollutant washoff is simulated using combined runoff from snowmelt and/or rainfall. For continuous SWMM, regeneration of any pollutant may depend upon whether snow cover is present if, for example, chlorides are to be simulated.

Snow and Temperature Generation from NWS Tapes

National Weather Service (NWS) Data

Continuous SWMM utilizes long-term precipitation and temperature data obtained from the National Climatic Data Center (NCDC) at Asheville, North Carolina, for the nearest NWS or airport weather station of record. (Similar data, but with a different format, are available in Canada from the Atmospheric Environment Service.) If snowmelt is not simulated only the precipitation tape is needed; hourly precipitation totals are included on it for every day with measurable precipitation. For continuous SWMM without snowmelt, all such hourly values are treated as rainfall.

Maximum and minimum temperatures as well as several other meteorological parameters are given for every day of the year on the NCDC’s “Surface Land Daily Cooperative, Summary of Day, TD-3200.” For snowmelt, only the ID number, date and max-min temperatures are used although other data (e.g., evaporation) may be used for other purposes. Note that the ID number for TD-3200 is not necessarily the same as for the hourly precipitation data. The data are accessed in the Rain and Temp Blocks, usually directly from the magnetic tape. The unit number of the tape is input in the Executive Block as NSCRAT(1). As explained in the description of continuous SWMM, a magnetic tape containing card images of hourly precipitation values is accessed similarly using unit number JIN(1).

Temperature data are input and processed for every day of the year, including summer months. Should an entry (date) be missing, the max-min values for the previous day are used.

Creation of Hourly Temperatures

The “Summary of Day” or temperature tape does not list the time of day at which the minimum and maximum temperatures occur. Hence, the minimum temperature is assumed to occur at sunrise each day, and the maxi-mum is assumed to occur three hours prior to sunset. All times are rounded to the nearest hour. This scheme obviously cannot account for many meteorological phenomena that would create other temperature-time distributions but is apparently the most appropriate one under the circumstances. Given the max-min temperatures and their assumed hours of occurrence, the other 22 hourly temperatures are readily created by sinusoidal interpolation, as sketched in Figure II-1. The interpolation is performed, using three different periods: 1) between the maximum of the previous day and the minimum of the present, 2) between the minimum and maximum of the present, and 3) between the maximum of the present and minimum of the following.

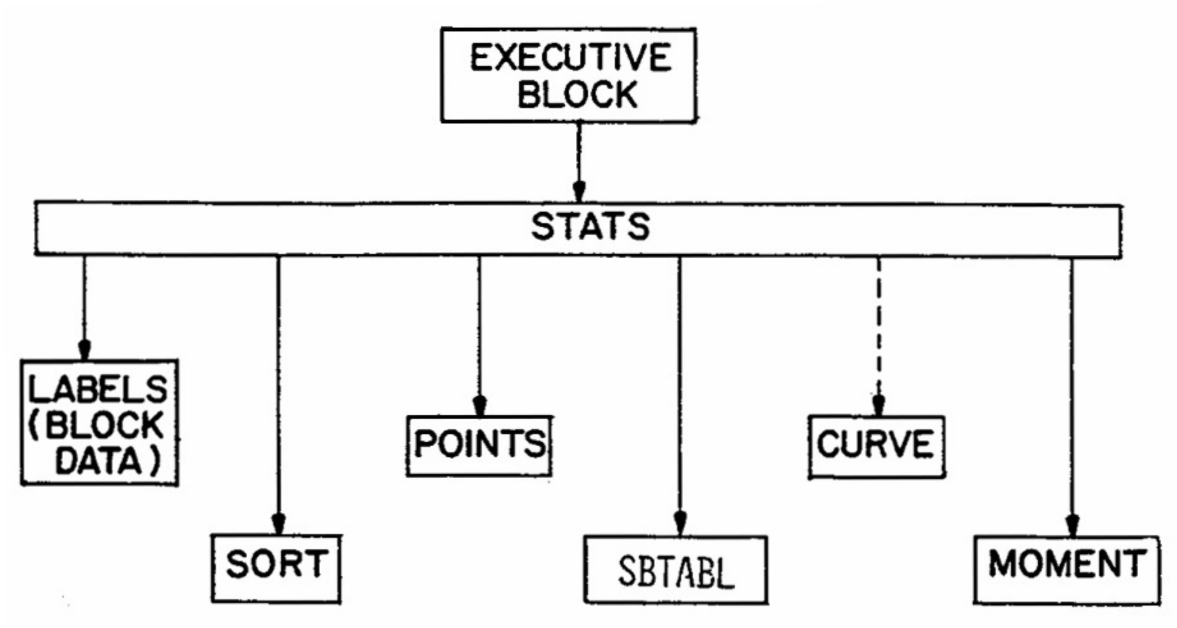


Figure II-1. Sinusoidal interpolation of hourly temperatures.

The time of day of sunrise and sunset are easily obtained as a function of latitude and longitude of the catchment and the date. Techniques for these computations are explained, for example, by List (1966) and by the TVA (1972). The Runoff Block utilizes approximate (but sufficiently accurate) formulas given in the latter reference. Their use is explained briefly below.

The hour angle of the sun, h, is the angular distance between the instantaneous meridian of the sun (i.e., the meridian through which passes a line from the center of the earth to the sun) and the meridian of the observer (i.e., the meridian of the catchment). It may be measured in degrees or radians or readily converted to hours, since 24 hours is equivalent to 360 degrees or 2 pi radians. The hour angle is a function of latitude, declination of the earth and time of day and is zero at noon, true solar time, and positive in the afternoon. However, at sunrise and sunset, the solar altitude of the sun (vertical angle of the sun measured from the earth’s surface) is zero, and the hour angle is computed only as a function of latitude and declination,

$$\cos h = -\tan \delta \text{ ⟐ } \tan \phi \tag{II-1}$$

where

h = hour angle, radians,
δ = earth’s declination, a function of season (date), radians, and
ϕ = latitude of observer, radians.

The earth’s declination is provided in tables (e.g., List, 1966), but for programming purposes an approximate formula is used (TVA, 1972):

$$\delta = \left(\frac{23.45 \text{ pi}}{180} \right) \cos \left[\frac{2 \text{ pi}}{365} (172 - \text{D}) \right] \tag{II-2}$$

where D is number of the day of the year (no leap year correction is warranted) and d is in radians. Having the latitude as an input parameter, the hour angle is thus computed in <u>hours</u>, positive for sunset, negative for sunrise, as

$$h = (12/\text{pi}) \cos^{-1} (-\tan \delta \text{ ⟐ } \tan \phi) \tag{II-3}$$

The computation is valid for any latitude between the arctic and Antarctic circles, and no correction is made for obstruction of the horizon.

The hour of sunrise and sunset is symmetric about noon, true solar time. True solar noon occurs when the sun is at its highest elevation for the day. It differs from standard zone time, i.e., the time on clocks) because of a longitude effect and because of the “equation of time”. The latter is of astronomical origin and causes a correction that varies seasonally between approximately ± 15 minutes.; it is neglected here. The longitude correction accounts for the time difference due to the separation of the meridian of the observer and the meridian of the standard time zone. These are listed in Table II-1. It is readily computed as

Table II-1. Time Zones and Standard Meridians

Time Zone	Example Cities	Standard Meridian
Newfoundland Std. Time	St. Johns's, Newfoundland	52.5[a]
Atlantic Std. Time	Halifax, Nova Scotia San Juan, Puerto Rico	60
Eastern Std. Time	New York, New York	75
Central Std. Time	Chicago, Illinois	90
Mountain Std. Time	Denver, Colorado	105
Pacific Std. Time	San Francisco, California	120
Yukon Std. Time	Yakutat, Alaska[b]	135
Alaska Std. Time Hawaiian Std. Time	Anchorage, Alaska Honolulu, Hawaii	150
Bering Std. Time	Nome, Alaska	165

[a] The time zone of the island of Newfoundland is offset one half hour from other zones.
[b] All of the Yukon Territory is on Pacific Standard Time.

$$\mathrm{DTLONG} = 4\,\frac{\mathrm{minutes}}{\mathrm{degree}} \times (\Theta - \mathrm{SM}) \tag{II-4}$$

where

DTLONG = longitude correction, minutes (of time),
Θ = longitude of the observer, degrees, and
SM = standard meridian of the time zone, degrees, from Table II-1.

Note that DTLONG can be either positive or negative, and the sign should be retained. For instance, Boston at approximately 71°W has DTLONG = -16 minutes, meaning that mean solar noon precedes EST noon by 16 minutes. (Mean solar time differs from true solar time by the neglected "equation of time.")

The time of day of sunrise is then

$$\mathrm{HSR} = 12 - h + \mathrm{DTLONG}/60 \tag{II-5}$$

and the time of day of sunset is

$$\mathrm{HSS} = 12 + h + \mathrm{DTLONG}/60 \tag{II-6}$$

These times are rounded to the nearest hour for use in continuous SWMM. As stated earlier, the maximum temperature is assumed to occur at hour HSS - 3.

Standard time is used in all calculations and in NWS tapes. There is no input or output that includes allowance for daylight savings time.

Generation of Snowfall Intensities

The estimated hourly temperatures, T, in °F, are compared to a dividing temperature, SNOTMP, for each hour with precipitation. Then if

$$\begin{aligned} &T > \text{SNOTMP, precipitation = rain;} \\ &T \geq \text{SNOTMP, precipitation = snow.} \end{aligned} \qquad \text{(II-7)}$$

Snowfall depths are tagged as negative quantities for identification by later components of the program.

Gage Catch Deficiency Correction

Precipitation gages tend to produce inaccurate snowfall measurements because of the complicated aerodynamics of snow flakes falling into the gage. Snowfall totals are generally underestimated as a result, by a factor that varies considerably depending upon gage exposure, wind velocity and whether or not the gage has a wind shield. The program includes a parameter, SCF, which multiplies snow depths only.

Although it will vary considerable from storm to storm, SCF acts as a mean correction factor over a season in the model. Anderson (1973) provides typical values of SCF as a function of wind speed, as shown in Figure II-2, that may be helpful in establishing an initial estimate. The value of SCF can also be used to account for other factors, such as losses of snow due to interception and sublimation not accounted for in the model. Anderson (1973) states that both losses are usually small compared to the gage catch deficiency.

Structure of Precipitation - Temperature Data Set

The Rain and Temp Blocks create output files from the NWS precipitation and temperature data tapes that are subsequently read as input by the Runoff Block. The interested user can find descriptions of the output file format used by the Rain and Temp Blocks in Sections 10 and 11, respectively.

Single Event SWMM

NWS tapes are not used in single event simulation. Precipitation is entered on Runoff Block data group E1-E3. However, snowfall can be included, if desired, as a negative precipitation value at any time step.

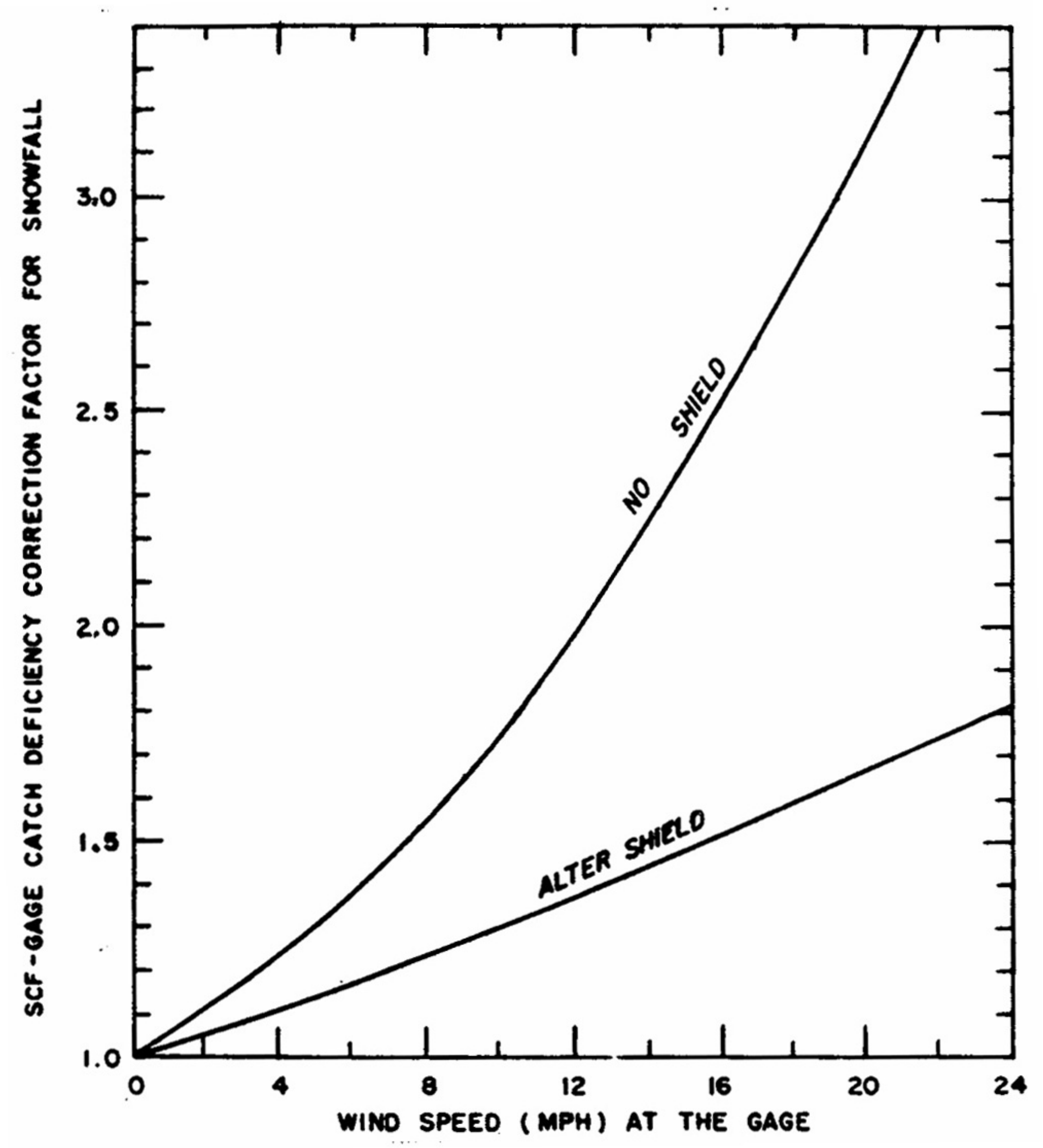


Figure II-2. Typical gage catch deficiency correction (Anderson, 1973, p. 5-20).

Subcatchment Schematization

Land Surface - Snow Cover Combinations

In order to have flexibility in treating different combinations of snow cover and ground surface types, four such combinations are provided, as described in Table II-2 and illustrated in Figure II-3. When snowmelt is not simulated, only the first three are used, as in the past. (Type 3, impervious area with no depression storage, is specified in Runoff by the parameter PCTZER, percent of impervious area with immediate runoff.) Snow cover is treated identically on types 1 and 3 since these surfaces are likely to be of similar nature, e.g., streets, sidewalks, parking lots, etc. For continuous simulations, these surfaces are considered "normally bare" because of probable plowing, salting or other rapid snow removal, but are subject to snow cover also, as described subsequently. For single event simulation, these surfaces are always bare; all snow on impervious areas is handled in type 4.

In Runoff, especially subroutine WSHED, the "types" are subscripts for the parameter WDEPTH, the water depth on each surface type. Since snow cover is the same for types 1 and 3, snow depths, WSNOW, are only triply subscripted.

For single event simulation, the fraction of snow-covered pervious area is constant; for continuous simulation the fraction varies according to an areal depletion curve (as for type 4 impervious). The depletion curves are explained later.

Apportionment of impervious area is different when simulating with and without snowmelt. For the latter situation, the area with zero depression storage (type 3) is taken to be a percentage, PCTZER, of the total impervious area. For the former situation (with snowmelt), it is taken as a percentage, the "normally bare" impervious area (continuous simulation). Thus, the type 3 area will vary according to whether snowmelt is simulated or not, as shown in Figure II-3. The effect on outflow is very minor. The fraction of impervious area with 100 percent snow cover (single event) or subject to an areal depletion curve (continuous) is an input parameter, SNN1, for each subcatchment.

Table II-2. Subcatchment Surface Classification

Type	Perviousness	Depression Storage	Snow Cover and Extent	
			Single Event*	Continuous*
1	Impervious	Yes	Bare	Normally bare, but may have snow cover over 100% of type 1 plus type 3 area.
2	Pervious	Yes	Constant fraction, SNCP, of area is snow covered.	Snow covered subject to areal depletion curve.
3	Impervious	No	Bare	Same as type 1.
4	Impervious	Yes	100% covered.	Snow covered subject to areal depletion curve.

*Single event or continuous is determined by parameter ISNOW in Runoff Block input.

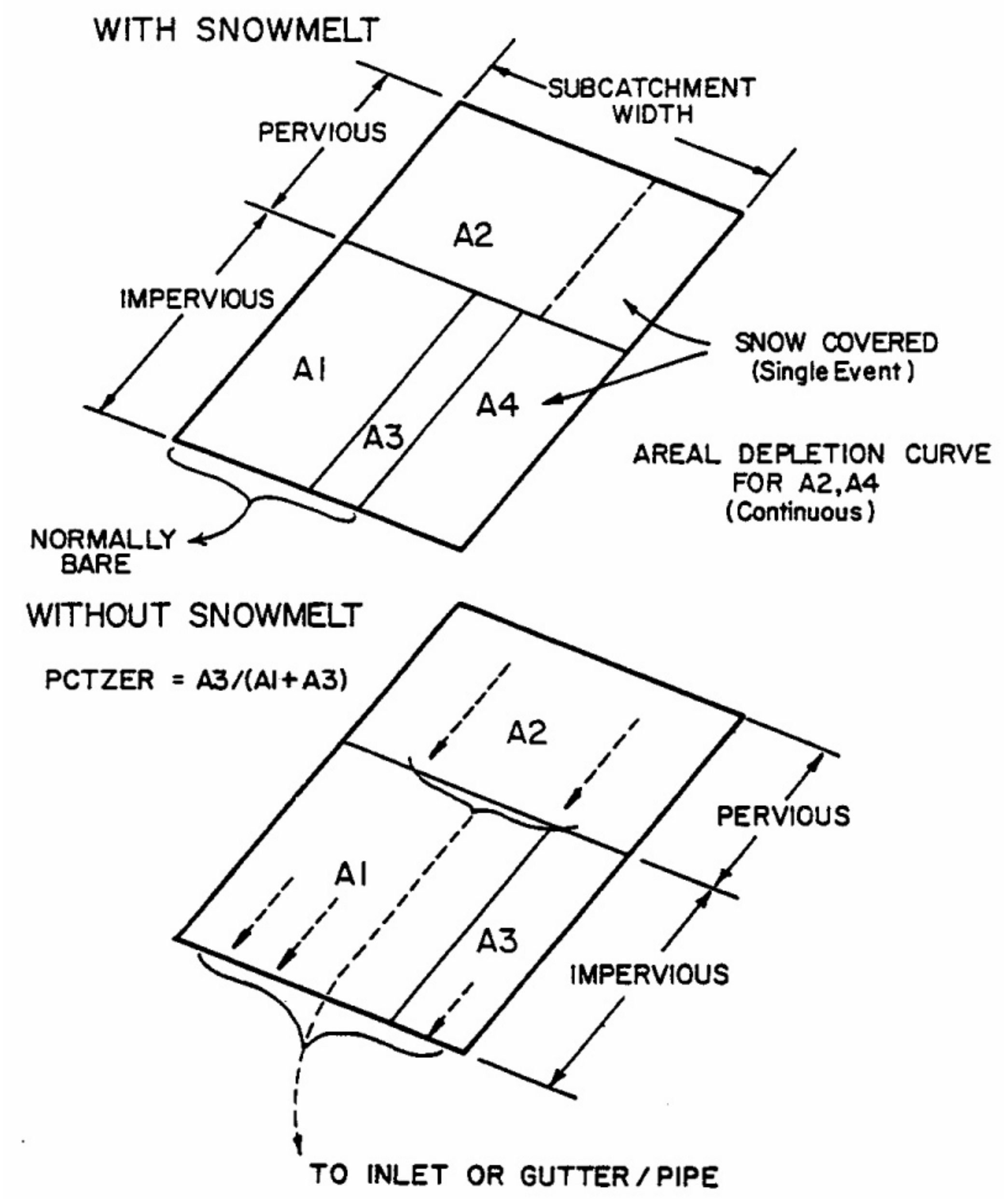


Figure II-3. Subcatchment schematization with and without snowmelt simulation. See also Table II-2.

Redistribution and Simulation of Snow Removal

Snow removal practices form a major difference between the snow hydrology of urban and rural areas. Much of the snow cover may be completely removed from heavily urbanized areas, or plowed into windrows or piles, with melt characteristics that differ markedly from those of undisturbed snow. Management practices in cities vary according to location, climate, topography and the storm itself; they are summarized in a study by APWA (1974). It is probably not possible to treat them all in a simulation model. See Table R-20. However, in continuous SWMM, provision is made to approximate simulation of some practices.

It is assumed that all snow subject to “redistribution”, (e.g. plowing) resides on the “normally bare” category, type 1 plus 3 above, (see Figure II-3), that might consist of streets, sidewalks, parking lots, etc. (The desired degree of definition may be obtained by using several subcatchments, although a coarse schematization, e.g., one or two subcatchments, may be sufficient for some continuous simulations.) For each subcatchment, a depth of snow, WEPLOW, is input for this area, above which redistribution occurs as indicated in Figure II-4. All snow in excess of this depth, say 0.1 - 0.2 in. water equivalent (2.5 - 5.1 mm), is redistributed to other areas according to five fractions, SFRAC, input for each subcatchment. These are described on Figure II-4. For instance, if snow is usually windrowed onto adjacent impervious or pervious areas, SFRAC(1) or SFRAC(2) may be used. If it is trucked to another subcatchment (the last one input is used for this purpose), a fraction SFRAC(3) will so indicate, or SFRAC(4) if the snow is removed entirely from the simulated watershed. In the latter case, such removals are tabulated and included in the final continuity check. Finally, excess snow may be immediately “melted” (i.e., treated as rainfall), using SFRAC(5). The transfers are area weighted, of course, and the five fractions should sum to 1.0. A depth of snow WEPLOW remains on the normally bare area and is subject to melting as on the other areas. See Table II-3 for guidelines as to typical levels of service for snow and ice control (Richardson et al., 1974).

No pollutants are transferred with the snow. The transfers are assumed to have no effect on pollutant washoff and regeneration. In addition, all the parameters of this process remain constant throughout the simulation and can only represent averages over a snow season.

The redistribution simulation does not account for snow management practices using chemicals, e.g., roadway salting. This is handled using the melt equations, as described subsequently.

Array Restrictions

Continuous snowmelt and single event snowmelt are limited to the number of subcatchments defined by the variable NW in the parameter statement of the /Tapes/ Common. The NOW parameter is 100 in the default version of SWMM. This should be more than adequate for continuous simulation, with or without snowmelt, since only a coarse catchment discretization should be sufficient.

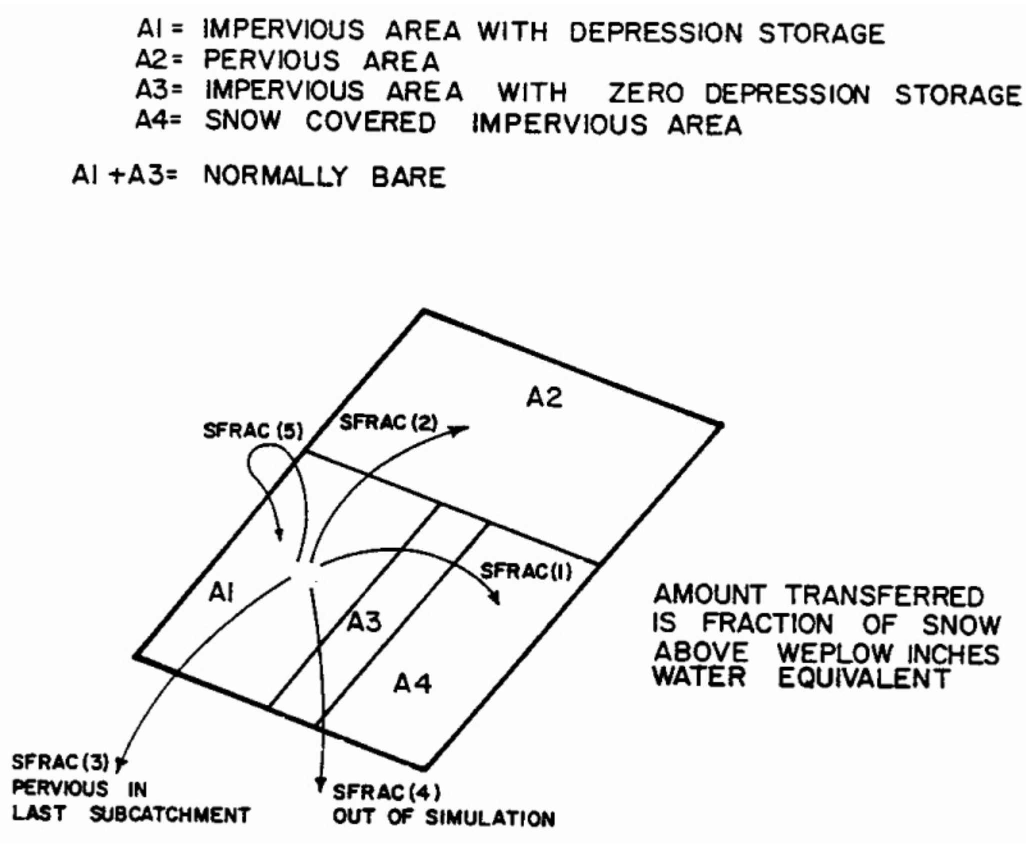


Figure II-4. Redistribution of snow during continuous simulation.

Table II-3. Guidelines for Levels of Service in Snow and Ice Control (Richardson et al., 1974)

Road Classification	Level of Service	Snow Depth to Start Plowing (Inches)	Max. Snow Depth on Pavement (Inches)	Full Pavement Clear of Snow After Storm (Hours)	Full Pavement Clear of Ice After Storm (Hours)
1. Low-Speed Multilane Urban Expressway	• Roadway routinely patrolled during storms • All traffic lanes treated with chemicals • All lanes (including breakdown lanes) operable at all times but at reduced speeds • Occasional patches of well sanded snow pack • Roadway repeatedly cleared by echelons of plows to minimize traffic disruption • Clear pavement obtained as soon as possible	0.5 to 1	1	1	12
2. High Speed 4-Lane Divided Highways Interstate System ADT greater than 10,000[a]	• Roadway routinely patrolled during storms • Driving and passing lanes treated with chemicals • Driving lane operable at all times at reduced speeds • Passing lane operable depending on equipment availability • Clear pavement obtained as soon as possible	1	2	1.5	12
3. Primary Highways Undivided 2 and 3 lanes ADT 500-5000[a]	• Roadway is routinely patrolled during storms • Mostly clear pavement after storm stops • Hazardous areas receive treatment of chemicals or abrasive • Remaining snow and ice removed when thawing occurs	1	2.5	2	24
4. Secondary Roads ADT less than 500[a]	• Roadway is patrolled at least once during a storm • Bare left-wheel track with intermittent snow cover • Hazardous areas are plowed and treated with chemicals or abrasives as a first order of work • Full width of road is cleared as equipment becomes available	2	3	3	48

[a] ADT – average daily traffic

Melt Calculations

Theory of Snowmelt

Introduction

Excellent descriptions of the processes of snowmelt and accumulation are available in several texts and simulation model reports and in the well known 1956 Snow Hydrology report by the Corps of Engineers (1956). The important heat budget and melt components are first mentioned briefly here; any of the above sources may be consulted for detailed explanations. A brief justification for the techniques adopted for snowmelt calculations in SWMM is presented below.

Snowpack Heat Budget

Heat may be added or removed from a snowpack by the following processes:

1. Absorbed solar radiation (addition).
2. Net longwave radiation exchange with the surrounding environment (addition or removal).
3. Convective transfer of sensible heat from air (addition or removal).
4. Release of latent heat of vaporization by condensate (addition) or, the opposite, its removal by sublimation (removing the latent heat of vaporization plus the latent heat of fusion).
5. Advection of heat by rain (addition) plus addition of the heat of fusion if the rain freezes.
6. Conduction of heat from underlying ground (removal or addition).

The terms may be summed, with appropriate signs, and equated to the change of heat stored in the snowpack to form a conservation of heat equation. All of the processes listed above vary in relative importance with topography, season, climate, local meteorological conditions, etc., but items 1-4 are the most important. Item 5 is of less importance on a seasonal basis, and item 6 is often neglected.

A snow pack is termed “ripe” when any additional heat will produce liquid runoff. Rainfall (item 5) will rapidly ripen a snowpack by release of its latent heat of fusion as it freezes in subfreezing snow, followed by quickly filling the free water holding capacity of the snow.

Melt Prediction Techniques

Prediction of melt follows from prediction of the heat storage of the snow pack. Energy budget techniques are the most exact formulation since they evaluate each of the heat budget terms individually, requiring as meteorologic input quantities such as solar radiation, air temperature, dew point or relative humidity, wind speed, and precipitation. Assumptions must be made about the density, surface roughness and heat and water storage (mass balance) of the snow pack as well as on related topographical and vegetative parameters. Further complications arise in dealing with heat conduction and roughness of the underlying ground and whether or not it is permeable.

Several models individually treat some or all of these effects. One of the more recent was developed for the NWS river forecast system by Anderson (1976). Interestingly, under many conditions he found that results obtained using his energy balance model were not significantly better than those obtained using simpler (e.g., degree-day or temperature-index) techniques in his earlier model (1973). The more open and variable the conditions, the better is the energy balance technique. Closest agreement between his two models was for heavily forested watersheds.

Minimal data needed to apply an energy balance model are a good estimate of incoming solar radiation, plus measurements of air temperature, vapor pressure (or dew point or relative humidity) and wind speed. All of these data, except possibly solar radiation, are available at at least one location (e.g., the airport) for almost all reasonably sized cities. Even solar radiation measurements are taken at several locations in most states. Predictive techniques are also available, for solar radiation and other parameters, based on available measurements (TVA, 1972; Franz, 1974).

Choice of Predictive Method

Two major reasons suggest that simpler, e.g., temperature-index, techniques should be used for simulation of snowmelt and accumulation in urban areas. First, even though required meteorologic data for energy balance models are likely to be available, there is a large local variation in the magnitude of these parameters due to the urbanization itself. For example, radiation melt will be influenced heavily by shading of buildings and albedo reduced by urban pollutants. In view of the many unknown properties of the snowpack itself in urban areas, it may be overly ambitious to attempt to predict melt at all! But at the least, simpler techniques are probably all that are warranted. They have the added advantage of considerably reducing the already extensive input data to a model such as SWMM.

Second, the objective of the modeling should be examined. Although it may contribute, snowmelt seldom causes flooding or hydrologic extremes in an urban area itself. Hence, exact prediction of flow magnitudes does not assume nearly the importance it has in the models of, say, the NWS, in which river flood forecasting for large mountainous catchments is of paramount importance. For planning purposes in urban areas, exact quantity (or quality) prediction is not the objective in any event; rather, these efforts produce a statistical evaluation of a complex system and help identify critical time periods for more detailed analysis.

For these and other reasons, simple snowmelt prediction techniques have been incorporated into SWMM. Anderson's NWS (1973) temperature-index method is also well documented and tested, and has been incorporated into SWMM. As described subsequently, the snowmelt modeling follows Anderson's work in several areas, not just in the melt equations. The energy budget technique is illustrated later to show how it reduces to a temperature-index equation under certain assumptions. It may be noted that the STORM model (Hydrologic Engineering Center, 1977; Roesner et al., 1974) also uses the temperature-index method for snowmelt prediction, in a considerably less complex manner than is now programmed in SWMM.

SWMM Melt Equations

Anderson's NWS model (1973) treats two different melt situations: with and without rainfall. When there is rainfall (greater than 0.1 in./6 hr or 2.5 mm/6 hr in the NWS model; greater than 0.02 in./hr or 0.51 mm/hr in SWMM), accurate assumptions may be made about several energy budget terms. These are: zero solar radiation, incoming longwave radiation equals blackbody radiation at the ambient air temperature, the snow surface temperature is 32°F (0°C), and the dew point and rain water temperatures equal the ambient air temperature. Anderson combines the appropriate terms for each heat budget component into one equation for the melt rate. As used in subroutine MELT in SWMM, it is:

$$\text{SMELT} = (\text{TA} - 32) \cdot (0.001167 + \text{SGAMMA} \cdot \text{UADJ} + 0.007 \cdot \text{PREC}) + 8.5 \cdot \text{UADJ} \cdot (\text{EA} - 0.18) \tag{II-8}$$

where

SMELT = melt rate, in./hr,
TA = air temperature, °F,
SGAMMA = 7.5 $\cdot$ GAMMA, in. Hg/°F,
GAMMA = psychometric constant, in. Hg/°F,
UADJ = wind speed function, in. /in. Hg - hr,
PREC = rainfall intensity, in./hr, and
EA = saturation vapor pressure at air temperature, in. Hg.

The psychometric constant, GAMMA, is calculated as:

$$\text{GAMMA} = 0.000359 \cdot \text{PA} \tag{II-9}$$

where PA = atmospheric pressure, in. Hg.

Average atmospheric pressure is in turn calculated as a function of elevation, z:

$$\text{PA} = 29.9 - 1.02\ (z/1000) + 0.0032 \cdot (z/1000)^{2.4} \tag{II-10}$$

where z = average catchment elevation, ft.

The elevation, z, is an input parameter, ELEV. The wind function, UADJ, accounts for turbulent transport of sensible heat and water vapor. Anderson (1973) gives:

$$\text{UADJ} = 0.006 \cdot u \tag{II-11}$$

where

UADJ = wind speed function, in./in. Hg - hr, and
u = average wind speed 1.64 ft (0.5 m) above the snow surface, mi/hr.

In practice, available wind data are used and are seldom corrected for the actual elevation of the anemometer. For SWMM, average wind speeds are input for each month. Finally, the saturation vapor pressure, EA, is given accurately by the convenient exponential approximation,

$$\text{EA} = 8.1175 \times 10^6 \exp[-7701.544/(\text{TA} + 405.0265)] \tag{II-12}$$

where

EA = saturation vapor pressure, in. Hg, and
TA = air temperature, °F.

The origin of numerical constants found in equation II-8 for SMELT is given by Anderson (1973), and reflects units conversions as well as U.S. customary units for physical properties. Note that equation II-13 of Appendix III may be reduced to equation II-8.

During non-rain periods, melt is calculated as a linear function of the difference between the air temperature, TA, and a base temperature, TBASE, using a degree-day or temperature-index type equation:

$$\mathrm{SMELT} = \mathrm{DHM} \diamond (\mathrm{TA} - \mathrm{TBASE}) \tag{II-13}$$

where

SMELT = snowmelt, in./hr (internally as ft/sec,)
TA = air temperature, °F,
TBASE = base melt temperature, °F, and
DHM = melt factor, in./hr-°F (internally ft/sec-°F).

Different values of TBASE and DHM may be input for three area classifications for each subcatchment (see Table II-2 and Figure II-3). For instance, these parameters may be used to account for street salting which lowers the base melt temperature. If desired, rooftops could be simulated as a separate subcatchment using a lower value of TBASE to reflect heat transfer vertically through the roof. Values of TBASE will probably range between 25 and 32 °F (-4 and 0 °C). Unfortunately, few urban area data exist to define adequately appropriate modified values for TBASE and DHM, and they may be considered calibration parameters.

In rural areas, the melt coefficient ranges from 0.03 - 0.15 in./day-°F (1.4 - 6.9 mm/day-°C) or from 0.001 - 0.006 in./hr-°F (0.057 - 0.29 mm/hr-°C). In urban areas, values may tend toward the higher part of the range due to compression of the pack by vehicles, pedestrians, etc. Again there appear to be few data available to produce accurate estimates. However, Bengtsson (1981) and Westerstrom (1981) do describe preliminary results of urban snowmelt studies in Sweden, including degree-day coefficients which range from 3 to 8 mm/°C-day (0.07 - 0.17 in./°F-day). Additional data for snowmelt on an asphalt surface (Westerstrom, 1984) gave degree-day coefficients of 1.7 - 6.5 mm/°C-day (0.04 - 0.14 in./°F-day).

It is important to realize that a degree-day equation may be derived from the complete energy budget equation if parameters other than air temperature are held constant. The equation is simply linearized about a desired air temperature range, and numerical values for DHM and TBASE computed. The values are accurate for the assumed values of other parameters, but may not appear to make sense physically, e.g., it is not difficult to use parameters that produce negative values of TBASE. An example of this procedure is given in Appendix III. It also serves to illustrate the energy budget computation method.

For single event SWMM, parameters DHM and TBASE are constant throughout the simulation. For continuous SWMM, TBASE remains constant, but DHM is allowed a seasonal variation, as illustrated in Figure II-5. Following Anderson (1973), the minimum melt coefficient is assumed to occur on December 21 and the maximum of June 21. Parameters

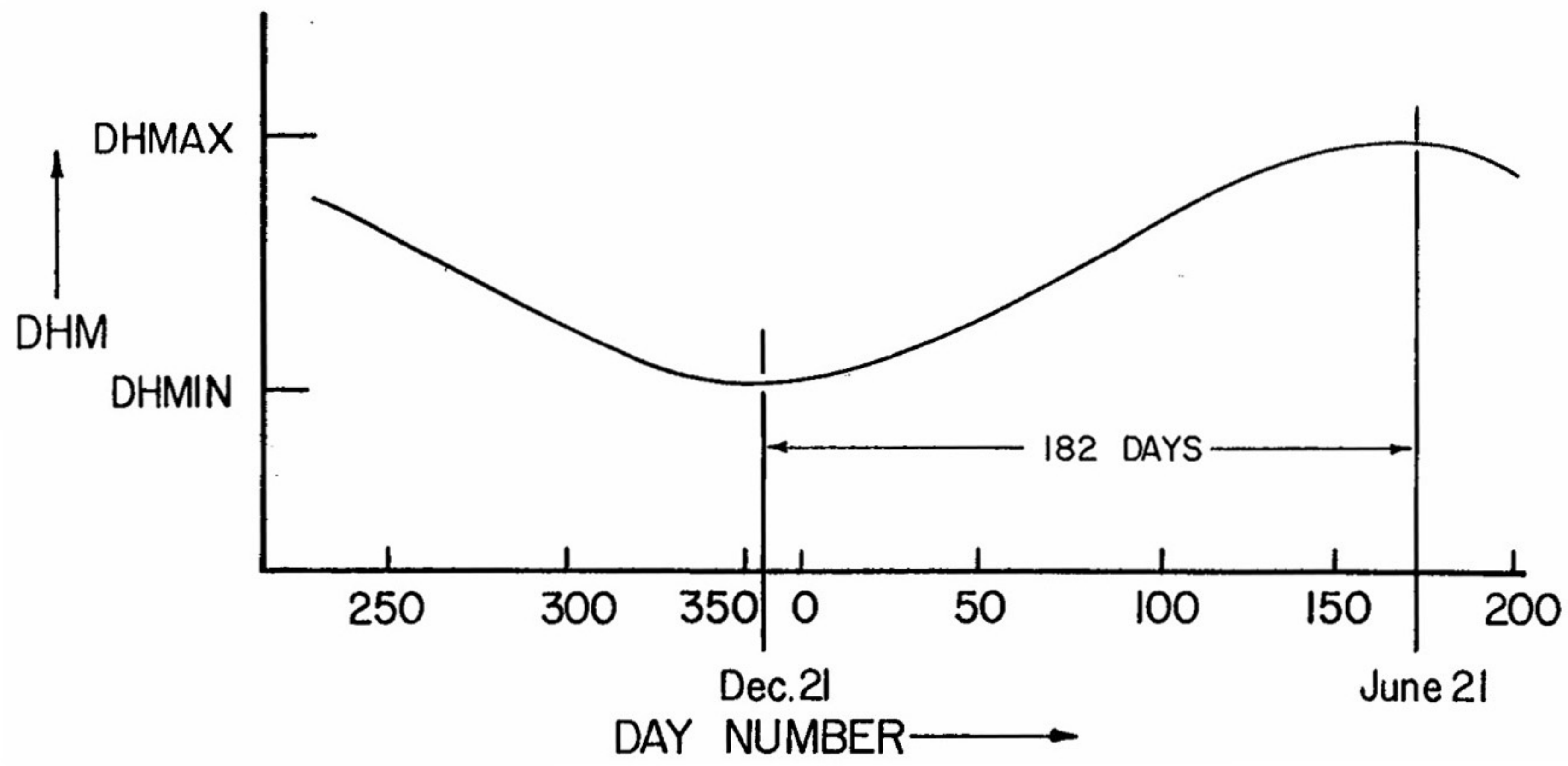


Figure II-5. Seasonal variation of melt coefficients

DHMIN and DHMAX are input for the three areas of each subcatchment, and sinusoidal interpolation is used to produce a value of DHM, constant over each day,

$$\mathrm{DHM} = (\mathrm{DHMAX} + \mathrm{DHMIN})/2 + (\mathrm{DHMAX} - \mathrm{DHMIN}) \Big/ 2 \cdot \sin\left[\frac{\mathrm{pi}}{182}(\mathrm{D} - 81)\right] \qquad \text{(II-14)}$$

where

DHMIN = minimum melt coefficient, occurring Dec. 21, in./hr-°F,
DHMAX = maximum melt coefficient, occurring June 21, in./hr-°F, and
D = number of the day of the year.

No special allowance is made for leap year. However, the correct date (and day number, D) is maintained.

Heat Exchange During Non-Melt Periods

During subfreezing weather, the snow pack does not melt, and heat exchange with the atmosphere can either warm or cool the pack. The difference between the heat content of the subfreezing pack and the (higher) base melt temperature is taken as positive and termed the “cold content” of the pack. No melt will occur until this quantity, COLDC, is reduced to zero. It is maintained in inches (or feet) of water equivalent. That is, a cold content of 0.1 in. (2.5 mm) is equivalent to the heat required to melt 0.1 in. (2.5 m) of snow. Following Anderson (1973), the heat exchange altering the cold content is proportional to the difference between the air temperature, TA, and an antecedent temperature index, ATI, indicative of the temperature of the surface layer of the snow pack. The revised value of ATI at time step 2 is calculated as

$$\mathrm{ATI}_2 = \mathrm{ATI}_1 + \mathrm{TIPM} \cdot (\mathrm{TA}_2 - \mathrm{ATI}_1) \qquad \text{(II-15)}$$

where

ATI = antecedent temperature index, °F,
TA = air temperature, °F,
TIPM = antecedent temperature index parameter, $0 \le \mathrm{TIPM} \le 1.0$, and

subscripts 1 and 2 refer to time steps 1 and 2, respectively. The value of ATI is not allowed to exceed TBASE, and when snowfall is occurring, ATI takes on the current air temperature.

The weighting factor, TIPM, is an indication of the thickness of the “surface” layer of snow. Values of TIPM less than 0.1 give significant weight to temperatures over the past week or more and would thus indicate a deeper layer than TIPM values greater than, say, 0.5, which would essentially only give weight to temperatures during the past day. In other words, the pack will both warm and cool more slowly with low values of TIPM. Anderson states that TIPM = 0.5 has given reasonable results in natural watersheds, although there is some evidence that a lower value may be more appropriate. No calibration has been attempted on urban water-sheds.

Following computation of the antecedent temperature index, the cold content is changed by an amount

$$\mathrm{DCOLDC} = \mathrm{RNM} \diamond \mathrm{DHM} \diamond (\mathrm{ATI} - \mathrm{TA}) \diamond \mathrm{DELT}$$

(II-16)

where

DCOLDC = change in cold content, ft water equivalent,
RNM = ratio of negative melt coefficient to melt coefficient,
DHM = melt coefficient, ft/sec-°F,
TA = air temperature, °F,
ATI = antecedent temperature index, °F, and
DELT = time step, sec.

Note that the cold content is increased, (DCOLDC is positive) when the air temperature is less (colder) than the antecedent temperature index. Since heat transfer during non-melt periods is less than during melt periods, Anderson uses a “negative melt coefficient” in the heat exchange computation. SWMM computes this simply as a fraction, RNM, of the melt coefficient, DHM. Hence, the negative melt coefficient, i.e., the product RNM × DHM, also varies seasonally. A typical value of RNM is 0.6.

When heat is added to a snow pack with zero cold content, liquid melt is produced, but runoff does not occur, until the “free water holding capacity” of the snow pack is filled. This is discussed subsequently. For single event SWMM no cold content calculations are performed; values of COLDC are assumed to equal zero throughout the simulation. The value of COLDC is in units of feet of water equivalent over the area in question. The cold content “volume,” equivalent to calories or BTUs, is obtained by multiplying by the area. Finally, an adjustment is made to equation II-16 depending on the areal extent of snow cover. This is discussed below.

Areal Extent of Snow Cover

Introduction

The snow pack on a catchment rarely melts uniformly over the total area. Rather, due to shading, drifting, topography, etc., certain portions of the catchment will become bare before others, and only a fraction, ASC, will be snow covered. This fraction must be known in order to compute the snow covered area available for heat exchange and melt, and to know how much rain falls on bare ground. Because of year to year similarities in topography, vegetation, drift patterns, etc., the fraction, ASC, is primarily only a function of the amount of snow on the catchment at a given time; this function, called an “areal depletion curve”, is discussed below. These functions are used only for continuous SWMM to describe the seasonal growth and recession of the snow pack. For single event simulation, fractions of snow covered area are fixed for the pervious and impervious areas of each subcatchment.

Areal Depletion Curves

As used in most snowmelt models, it is assumed that there is a depth, SI, above which there will always be 100 percent cover. In some models, the value of SI is adjusted during the simulation; in SWMM it remains constant. The amount of snow present at any time is indicated by the parameter WSNOW, which is the depth (water equivalent) over each of the three possible snow covered areas of each subcatchment (see Figure II-3). This depth is nondimensionalized by SI for use in calculating ASC. Thus, an areal depletion curve is a plot of WSNOW/SI versus ASC; a typical ADC for a natural catchment is shown in Figure II-6. For values of the ratio AWESI = WSNOW/SI greater than 1.0, ASC = 1.0, that is, the area is 100 percent snow covered.

Some of the implications of different functional forms of the ADC may be seen in Figure II-7. Since the program maintains snow quantities, WSNOW, as the depth over the total area, AT, the actual snow depth, WS, and actual area covered, AS, are related by continuity:

WSNOW ⟐ AT = WS ⟐ AS
(II-17)

where

WSNOW = depth of snow over total area AT, ft water equivalent,
AT = total area, ft^2
WS = actual snow depth, ft water equivalent, and
AS = snow covered area, ft^2.

In terms of parameters shown on the ADC, this equation may be rearranged to read

AWESI = WSNOW/SI = (WS/SI) ⟐ (AS/AT) = (WS/SI) ⟐ ASC (II-18)

This equation can be used to compute the actual snow depth, WS, from known ADC parameters, if desired. It is unnecessary to do this in the program, but it is helpful in understanding the curves of Figure II-7. Thus:

WS = (AWESI/ASC) ⟐ SI (II-19)

Consider the three ADC curves B, C and D. For curve B, AWESI is always less than ASC; hence WS is always less than SI as shown in Figure II-7d. For curve C, AWESI = ASC, hence WS = SI, as shown in Figure II-7e. Finally, for curve D, AWESI is always greater than ASC; hence, WS is always greater than SI, as shown in Figure II-7f. Constant values of ASC at 100 percent cover and 40 percent cover are illustrated in Figure II-7c, curve A, and Figure II-7g, curve E, respectively. At a given time (e.g., t_1 in Figure II-7), the area of each snow depth versus area curve is the same and equal to AWESI ⟐ SI, (e.g., 0.¸ SI for time t_1).

Curve B on Figure II-7a is the most common type of ADC occurring in nature, as shown in Figure II-6. The convex curve D requires some mechanism for raising snow levels above their original depth, SI. In nature, drifting might provide such a mechanism; in urban areas, plowing and windrowing could cause a similar effect. A complex curve could be generated to represent specific snow removal practices in a city. However, the program utilizes only on ADC curve for all impervious areas (e.g., area A4 of Figure II-3 for all subcatchments) and only one ADC curve for all per-vious areas (e.g., area A2 of Figure II-3 for all subcatchments). This limitation should not hinder an adequate simulation since the effects of variations in individual locations are averaged out in the city-wide scope of most continuous simulations.

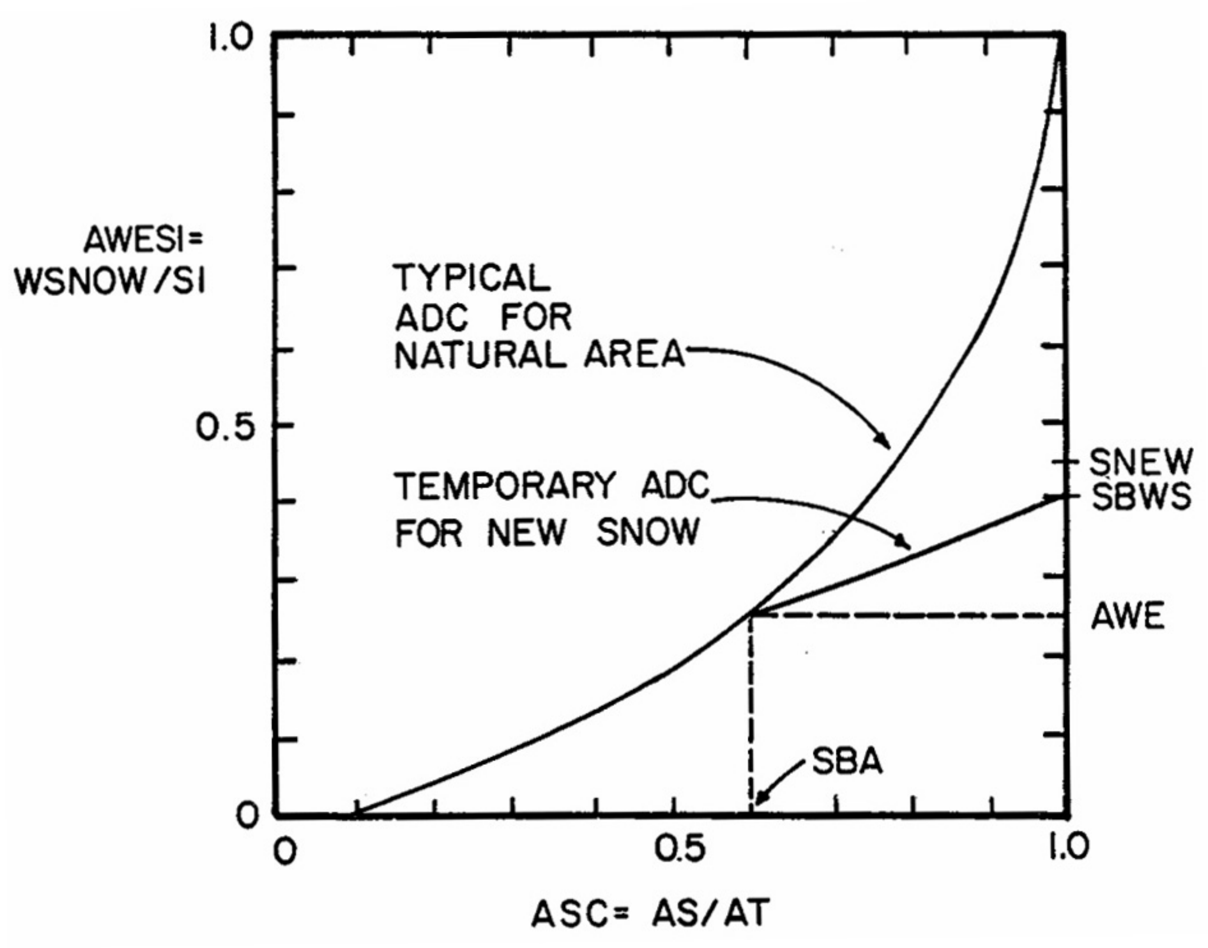


Figure II-6. Typical areal depletion curve for natural area (Anderson, 1973, p. 3-15) and temporary curve for new snow.

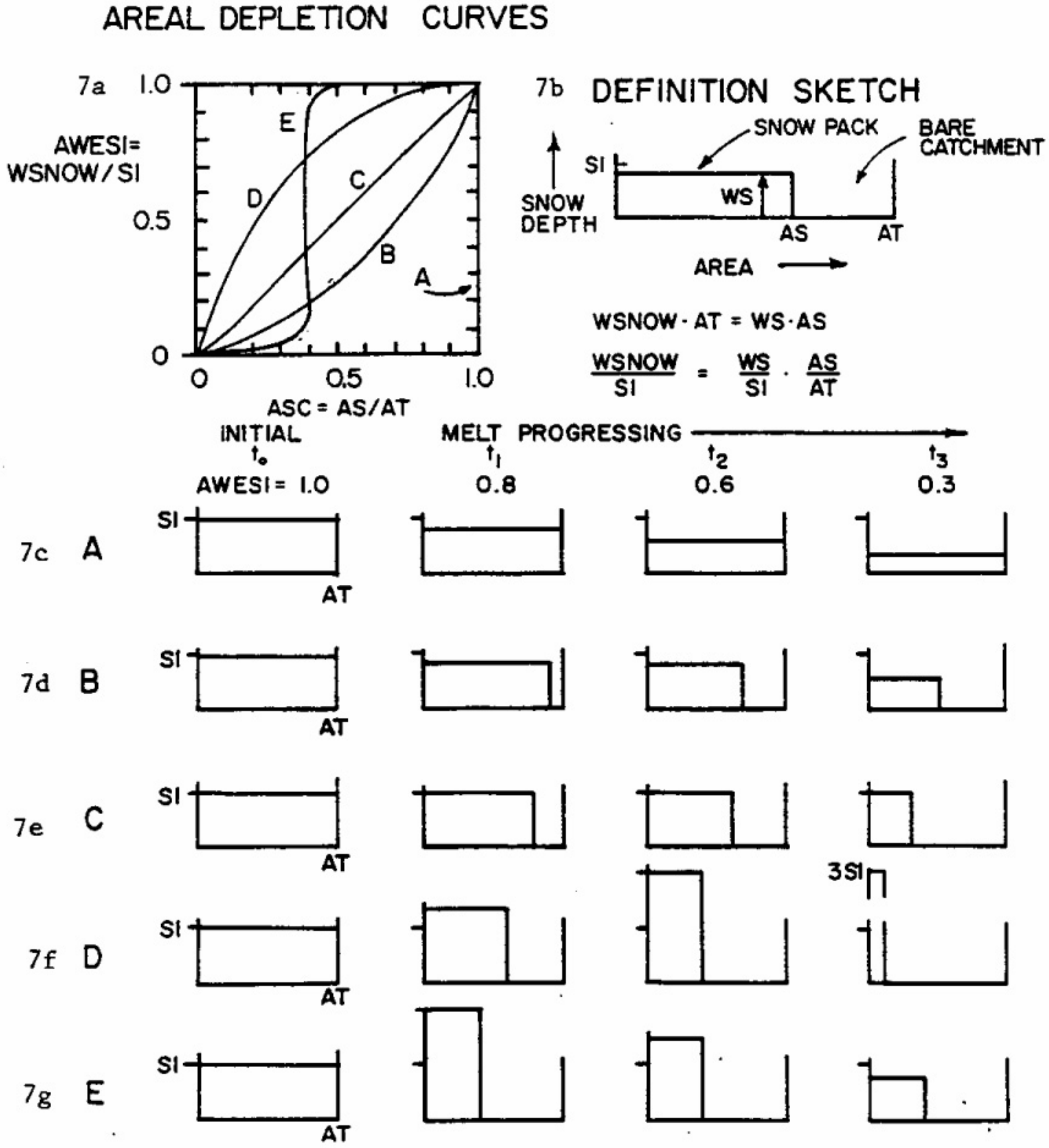


Figure II-7. Effect of snow cover on areal depletion curves.

The two ADC curves for pervious and impervious areas are input by the user, as are values of SI for each subcatchment. The program does not require the ADC curves to pass through the origin, AWESI = ASC = 0; they may intersect the abscissa at a value of ASC > 0 in order to maintain some snow covered area up until the instant that all snow disappears (see Figure II-6). However, the curves may not intersect the ordinate, AWESI > 0 when ASC = 0.

The preceding paragraphs have centered on the situation where a depth of snow greater than or equal to SI has fallen and is melting. (The ADC curves are not employed until WSNOW becomes less than SI.) The situation when there is new snow needs to be discussed, starting from both zero or non-zero initial cover. The SWMM procedure again follows Anderson's NWS method (1973).

When there is new snow and WSNOW is already greater than or equal to SI, them ASC remains unchanged at 1.0. However, when there is new snow on bare or partially bare ground, it is assumed that the total area is 100 percent covered for a period of time, and a "temporary" ADC is established as shown in Figure II-6. This temporary curve returns to the same point on the ADC as the snow melts. Let the depth of new snow be SNO, measured in equivalent feet of water. Then the value of AWESI will be changed from an initial value of AWE to a new value of SNEW by:

$$\text{SNEW} = \text{AWE} + \text{SNO/SI} \tag{II-20}$$

It is assumed that the areal snow cover remains at 100 percent until 25 percent of the new snow melts. This defines the value of SBWS of Figure II-6 as:

$$\text{SBWS} = \text{AWE} + 0.75 \Diamonddot \text{SNO/SI} \tag{II-21}$$

Anderson (1973) reports low sensitivity of model results to the arbitrary 25 percent assumption. When melt produces a value of AWESI between SBWS and AWE, linear interpolation of the temporary curve is used to find ASC until the actual ADC curve is again reached. When new snow has fallen, the program thus maintains values of AWE, SBA and SBWS (Figure II-6).

The interactive nature of melt and fraction of snow cover is not accounted for during each time step. It is sufficient to use the value of ASC at the beginning of each time step, especially with a short (e.g., one-hour) time step for the simulation.

Use of Value of ASC

The fraction of area that is snow covered, ASC, is used to adjust 1) the volume of melt that occurs, and 2) the "volume" of cold content change, since it is assumed that heat transfer occurs only over the snow covered area. The melt rate is computed from either of the two equations for SMELT. The snow depth is then reduced from its value at time step 1 to time step 2 as:

$$\text{WSNOW}_2 = \text{WSNOW}_1 - \text{SMELT} \Diamonddot \text{ASC} \tag{II-22}$$

with variables as defined previously and including appropriate continuity checks in the program to avoid melting more snow than is there, etc.

Cold content changes are also adjusted by the value of ASC. Thus, using equation II-16, cold content at time step 2 is computed from the value at time step 1 by:

$$\mathrm{COLDC}_2 = \mathrm{COLDC}_1 + \mathrm{RNM} \diamond \mathrm{DHM} \diamond (\mathrm{ATI}-\mathrm{TA}) \diamond \mathrm{DELT} \diamond \mathrm{ASC} \tag{II-23}$$

where variables are as previously defined. Again there are program checks for negative values of COLDC, etc.

Liquid Water Routing in Snow Pack

Production of melt does not necessarily mean that there will be liquid runoff at a given time step since a snow pack, acting as a porous medium with a “porosity,” has a certain “free water holding capacity” at a given instant in time. Following PR-JFM (1976a, 1976b), this capacity is taken to be a constant fraction, FWFRAC, of the variable snow depth, WSNOW, at each time step. This volume (depth) must be filled before runoff from the snow pack occurs. The program maintains the depth of free water, FW, ft of water, for use in these computations. When FW = FWFRAC × WSNOW, the snow pack is fully ripe. The procedure is sketched in Figure II-8.

The inclusion of the free water holding capacity via this simple reservoir-type routing delays and somewhat attenuates the appearance of liquid runoff. The value of FWFRAC will normally be less than 0.10 and usually between 0.02 - 0.05 for deep snow packs (SWNOW > 10 in. or 254 mm water equivalent). However, Anderson (1973) reports that a value of 0.25 is not unreasonable for shallow snow packs that may form a slush layer. When rainfall occurs, it is added to the melt rate entering storage as free water. No free water is released when melt does not occur, but remains in storage, available for release when the pack is again ripe. This re-frozen free water is not included in subsequent cold content or melt computations.

Net Runoff

Melt from snow covered areas and rainfall on bare surfaces are area weighted and combined to produce net runoff onto the surface as follows:

$$\mathrm{RI} = \mathrm{ASC} \diamond \mathrm{SMELT} + (1.0 - \mathrm{ASC}) \diamond \mathrm{RINE} \tag{II-24}$$

where

RI = net runoff onto surface, ft/sec,
ASC = fraction of area that is snow covered,
SMELT = melt rate, including effect of attenuation due to free water holding capacity, ft/sec, and
RINE = rainfall intensity, ft/sec.

Thus, the net runoff acts just as rainfall would act alone in subsequent overland flow and infiltration calculations.

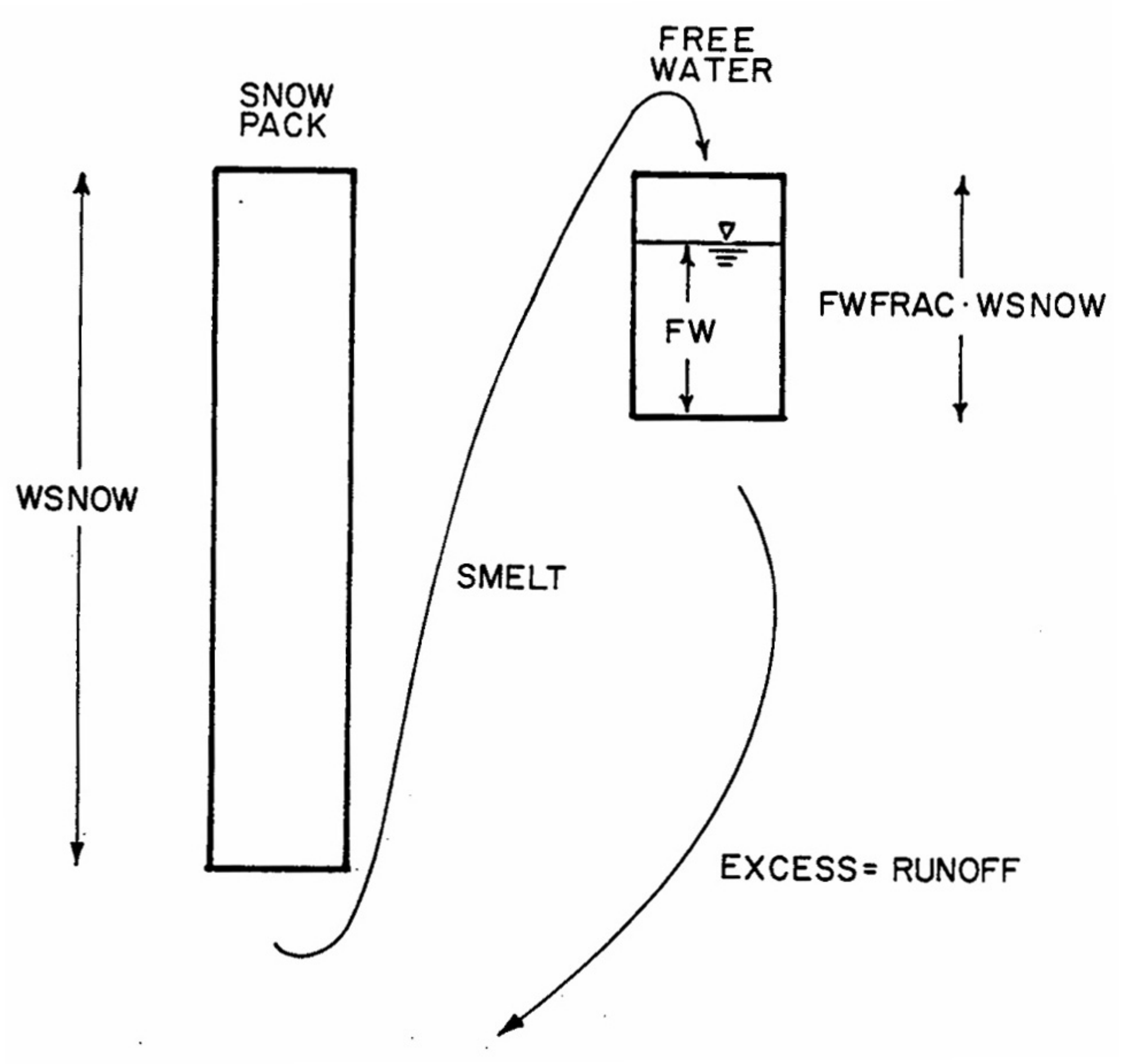


Figure II-8. Schematic of liquid water routing through snow pack.

If immediate melt is produced through the use of the snow redistribution fraction SFRAC(5) (see Figure II-4), it is added to the last equation. Furthermore, all melt calculations are ended when the depth of snow water equivalent becomes less than 0.001 in. (0.025 mm), and remaining snow and free water are converted to immediate melt and added to equation II-24.

Effect of Snow on Infiltration and Surface Parameters

A snow pack tends to insulate the surface beneath it. If ground has frozen prior to snowfall, it will tend to remain so, even as the snow begins to melt. Conversely, unfrozen ground is generally not frozen by subsequent snowfall. The infiltration characteristics of frozen versus unfrozen ground are not well understood and depend upon the moisture content at the time of freezing. For these and other reasons, SWMM assumes that snow has no effect on infiltration or other parameters, such as surface roughness or detention storage (although the latter is altered in a sense through the use of the free water holding capacity of the snow). In addition, all heat transfer calculations cease when the water becomes "net runoff." Thus, water in temporary surface storage during the overland flow routing will not refreeze as the temperature drops and is also subject to evaporation beneath the snow pack.

Quality Interactions

Pollutant Accumulation

Snowmelt Quality

A detailed review of literature related to snowmelt quality is given by PR-JFM (1976a, 1976b). Among the various contaminants found in deposited snow and melt water, chlorides and lead appear to be the most serious and potentially hazardous. Chloride concentrations in runoff along major highways can be higher than 20,000 mg/l, with typical values of from 1,000 to 10,000 mg/l. Several other studies also document chloride contamination and discuss street salting practices (Field et al., 1973; Richardson et al., 1974; Ontario Ministry of the Environment, 1974). Lead concentrations in snow windrows have been as high as 100 mg/l with typical values of from 1 to 10 mg/l. However, most deposited lead results from automobile combustion and is insoluble. Hence, melt runoff concentrations are lower than snow pack values and are mostly associated with suspended solids.

Pollutant Loadings

Mechanisms and modeling alternatives for pollutant buildup and washoff are described extensively in Section 4 (Runoff Block). Any parameter related to snowmelt may be generated using linear or non-linear buildup, or else a rating curve (load proportional to flow). Specifically, street salting chemicals may be simulated, such as sodium chloride or calcium chloride.

Adjustments for Presence of Snow

As a user option, regeneration of any quality constituent may be performed only when snow is present. This option is indicated by parameter LINKUP. Thus, if chlorides are simulated, for example, they will not be regenerated from bare ground, during the summer months for instance. However, regeneration when it does occur is a function only of snow <u>presence</u>, not the actual <u>amount</u> (depth).

Possible Loading Rates

Pollutant loading rates are best determined from local data. The literature review of PR-JFM (1976a, 1976b) may also be consulted for tables that may be used to estimate loading rate parameters for snow-associated pollutants. Other references will also be useful (e.g., Field et al., 1973; Richardson et al., 1974; Ontario Ministry of the Environment, 1974).

Table II-4 (Richardson et al., 1974) lists recommended deicing chemical application rates for roadways. In general, PR-JFM show that observed loading rates are functions of population density with suburban rates lower than arterial highway rates, as indicated in Table II-5. This is also true for other pollutants.

Street Sweeping

The effect of snow is included in two minor ways. First, beginning and ending dates, parameters KLNBGN and KLNEND respectively, may be input for continuous SWMM to indicate the interval during the year subject to street sweeping. If sweeping normally is not done between, say, December 1 and March 1, because of high snow volumes, this may be so indicated.

Second, the presence of snow can alter the street sweeping interval. These intervals are specified for each of the five land uses. Each subcatchment is swept when the number of dry time steps for that subcatchment exceeds the interval for the given land use. A dry time step, in subroutine QSHED, is one in which there is no precipitation and no water or snow on areas A1 and A3 (Figure II-3). Thus, subcatchments will not be swept until there is no snow or water on “normally bare” impervious areas.

Other Considerations

The snow itself is assumed to be “pure” and contain no pollutants. Thus, the redistribution or transfers of snow described earlier (Figure II-4) will not remove accumulated pollutants. This is partially justified on the basis of the assumption that such transfers would occur soon after fresh snow has fallen. They occur during the same time step in the model.

Although not well tested, it is assumed that the principal effect of inclusion of snowmelt upon runoff quality predictions of continuous SWMM will be to shift the season and magnitude of pollutant washoff. There will tend to be fewer periods of washoff during the winter. As snowmelt, equivalent melt rates are likely to be less than the usual magnitude of rainfall intensities experienced. Hence, concentrations may tend to be more uniform during the melt washoff events.

Data Requirements

Input Parameters

For single event simulation, input parameters include watershed elevation, free water holding capacities, air temperatures and wind speeds, and for each subcatchment, snow covered fractions, initial snow and free water, melt coefficients and base temperatures. Continuous simulation requires the same data as above, except that air temperatures are computed using other input parameters. In addition, it requires the snow gage correction factor, negative heat exchange parameter, areal depletion curves, and, for each subcatchment, the redistribution parameters. Of course, for continuous simulation, the required parameters can be kept to a minimum by keeping the number of subcatchments used to a minimum. Also required are pollutant loading data that may or may not be related to snow.

Table II-4. Guidelines for Chemical Application Rates (Richardson et al., 1974)

Weather Conditions			Application Rate (pounds of material per mile of 2-lane road or 2 lanes of divided)			
Temperature	Pavement Conditions	Precipitation	Low- and High-Speed Multilane Divided	Two- and Three-Lane Primary	Two-Lane Secondary	Instructions
30°F and above	Wet	Snow	300 salt	300 salt	300 salt	Wait at least 0.5 hour before plowing
		Sleet or Freezing Rain	200 salt	200 salt	200 salt	Reapply as necessary
25-30°F	Wet	Snow or Sleet	Initial at 400 salt Repeat at 200 salt	Initial at 400 salt Repeat at 200 salt	Initial at 400 salt Repeat at 200 salt	Wait at least 0.5 hour before plowing; repeat
		Freezing Rain	Initial at 300 salt Repeat at 200 salt	Initial at 300 salt Repeat at 200 salt	Initial at 300 salt Repeat at 200 salt	Repeat as necessary
20-25°F	Wet	Snow or Sleet	Initial at 500 salt Repeat at 250 salt	Initial at 500 salt Repeat at 250 salt	1200 of 5:1 sand/salt; repeat same	Wait about 0.75 hour before plowing; repeat
		Freezing Rain	Initial at 400 salt Repeat at 300 salt	Initial at 400 salt Repeat at 300 salt		Repeat as necessary
15-20°F	Dry	Dry Snow	Plow	Plow	Plow	Treat hazardous areas with 1200 of 20:1 sand/salt
	Wet	Wet Snow or Sleet	500 of 3:1 salt/ calcium chloride	500 of 3:1 salt/ calcium chloride	1200 of 5:1 sand	Wait about one hour before plowing; continue plowing until storm ends; then repeat application
Below 15°F	Dry	Dry Snow	Plow	Plow	Plow	Treat hazardous areas with 1200 of 20:1 sand/salt

Table II-5. Salting Rates Used in Ontario (Proctor and Redfern Ltd. and James F. MacLaren, Ltd., Vol. II, 1976b)

Population Density (person per sq mile)	Salting Rate per Application (lb per lane-mile)
Less than 1,000	75 - 800
1,000 to 5,000	350 - 1,800
More than 5,000	400 - 1,200

Sensitivity

The melt routines have not been sufficiently tested to date to quantify the sensitivity of results to various input parameters. It is expected that melt volumes will be most related to the precipitation record, of course, and to the gage correction factor, which influences the amount of snow that falls. Melt rates will be influenced by the melt coefficients and base temperatures, and, to some degree, by the areal depletion curves which simulate the relative "piling" or "stacking" of the snow.

Output

Temperature and Snowfall Generation

Output consists of temperatures synthesized from daily max-min values, and hourly precipitation totals, in which snowfall is tagged as a negative value.

Runoff Simulation Output

Snowmelt events are not indicated in a special manner for output by either continuous or single event SWMM. If daily output is used, snowmelt may be discerned to some degree by observing whether precipitation accompanies the runoff for that day. Snowfall and initial snow depths are identified as separate items in the final continuity check for the total watershed.

Appendix X

Subsurface Flow Routing in Runoff Block

Introduction

Because SWMM was originally developed to simulate combined sewer overflows in urban catchments, the fate of infiltrated water was considered insignificant. Since its development, however, SWMM has been used on areas ranging from highly urban to relatively undeveloped. Many of the undeveloped and even some of the developed areas, especially in areas like south Florida, are very flat with high water tables, and their primary drainage pathway is through the surficial groundwater aquifer and the unsaturated zone above it, rather than by overland flow. In these areas a storm will cause a rise in the water table and subsequent slow release of groundwater back to the receiving water (Capece et al., 1984). For this case, the fate of the infiltrated water is highly significant. By assuming that the infiltration is lost from the system, an important part of the high-water-table system is not being properly described (Gagliardo, 1986).

It is known that groundwater discharge accounts for the time-delayed recession curve that is prevalent in certain watersheds (Fetter, 1980). This process has not, however, been satisfactorily modeled by surface runoff methods alone. By modifying infiltration parameters to account for subsurface storage, attempts have been made to overcome the fact that SWMM assumes infiltration is lost from the system (Downs et al., 1986). Although the modeled and measured peak flows matched well, the volumes did not match well, and the values of the infiltration parameters were unrealistic. Some research on the nature of the soil storage capacity has been done in south Florida (SFWMD, 1984). However, it was directed towards determining an initial storage capacity for the start of a storm. There remains no standard, widely-used method for combining the groundwater discharge hydrograph with the surface runoff hydrograph and determining when the water table will rise to the surface. For instance, HSPF (Johansen et al., 1980) performs extensive subsurface moisture accounting and works well during average conditions. However, the model never permits the soil to become saturated so that no more infiltration is permitted, limiting its usefulness during times of surface saturation and flooding. Another difficulty with HSPF occurs during drought conditions, since there is no threshold saturated zone water storage (corresponding to the bottom of a stream channel) below which no saturated zone outflow will occur. These difficulties have limited HSPF usefulness for application to extreme hydrologic conditions in Florida (Heaney et al., 1986).

In order to incorporate subsurface processes into the simulation of a watershed and overcome previously mentioned shortcomings, SWMM has been equipped with a simple groundwater subroutine. The remainder of this appendix will describe the theory, use, and some limitations of the subroutine.

Theory

Introduction

An effort was made to utilize existing theoretical formulations for as many processes as possible. The purpose was to maintain semblance to the real world while enabling the user to determine parameter values that have meaning to the soil scientist. Also, in the following discussion the term “flow” will refer to water that is passed on to another part of the system, and the term “loss” will refer to water that is passed out of the system. In addition, in the groundwater subroutines, flows and losses have internal units of velocity (flow per unit area).

The groundwater subroutine, GROUND, simulates two zones – an upper (unsaturated) zone and a lower (saturated) zone. This configuration is similar to the work done by Dawdy and O’Donnell (1965) for the USGS. The flow from the unsaturated to the saturated zone is controlled by a percolation equation for which parameters may either be estimated or calibrated, depending on the availability of the necessary soil data. Upper zone evapotranspiration is the only loss from the unsaturated zone. The only inflow to subroutine GROUND is the calculated infiltration from subroutine WSHED. Losses and outflow from the lower zone can be via deep percolation, saturated zone evapotranspiration, and groundwater flow. Groundwater flow is a user-defined power function of water table stage and, if chosen, depth of water in the discharge channel.

Continuity

The physical processes occurring within each zone are accounted for by individual mass balances in order to determine end-of-time-step stage, groundwater flow, deep percolation, and upper zone moisture. Parameters are shown in Figure X-1 and defined below. Mass balance in the upper (unsaturated) zone is given by,

$$\mathrm{TH2} = \left\{ \begin{matrix} \left[(\mathrm{ENFIL} - \mathrm{ETU}) \cdot \mathrm{PAREA} - \mathrm{PERC} \right] \\ \cdot\, \mathrm{DELT} + (\mathrm{D1} - \mathrm{D2}) \cdot \mathrm{TH2} + \mathrm{TH} \cdot \mathrm{DWT1} \end{matrix} \right\} \Big/ (\mathrm{DTOT} - \mathrm{D2}) \qquad \text{(X-1)}$$

In the lower (saturated) zone, for rising water tables,

$$\mathrm{D2} = \left\{ \begin{matrix} \left[\begin{matrix} \mathrm{PERC} - \mathrm{ETD} \cdot \mathrm{PAREA} - .5 \\ \cdot \left(\begin{matrix} \mathrm{GWFKW} + \mathrm{A1} \cdot (\mathrm{D2} - \mathrm{BC})^{\mathrm{B1}} + \mathrm{A3} \\ \cdot\, \mathrm{D2} \cdot \mathrm{TW} + \mathrm{DEPRC} + \mathrm{DP} \cdot \mathrm{D2} / \mathrm{DTOT} \end{matrix} \right) - \mathrm{TWFFLW} \end{matrix} \right] \\ \mathrm{DELT} + (\mathrm{D2} - \mathrm{D1}) \cdot (\mathrm{TH} - \mathrm{TH2}) \end{matrix} \right\} \Big/ (\mathrm{PR} - \mathrm{TH2}) + \mathrm{D1} \qquad \text{(X-2)}$$

and for falling water tables,

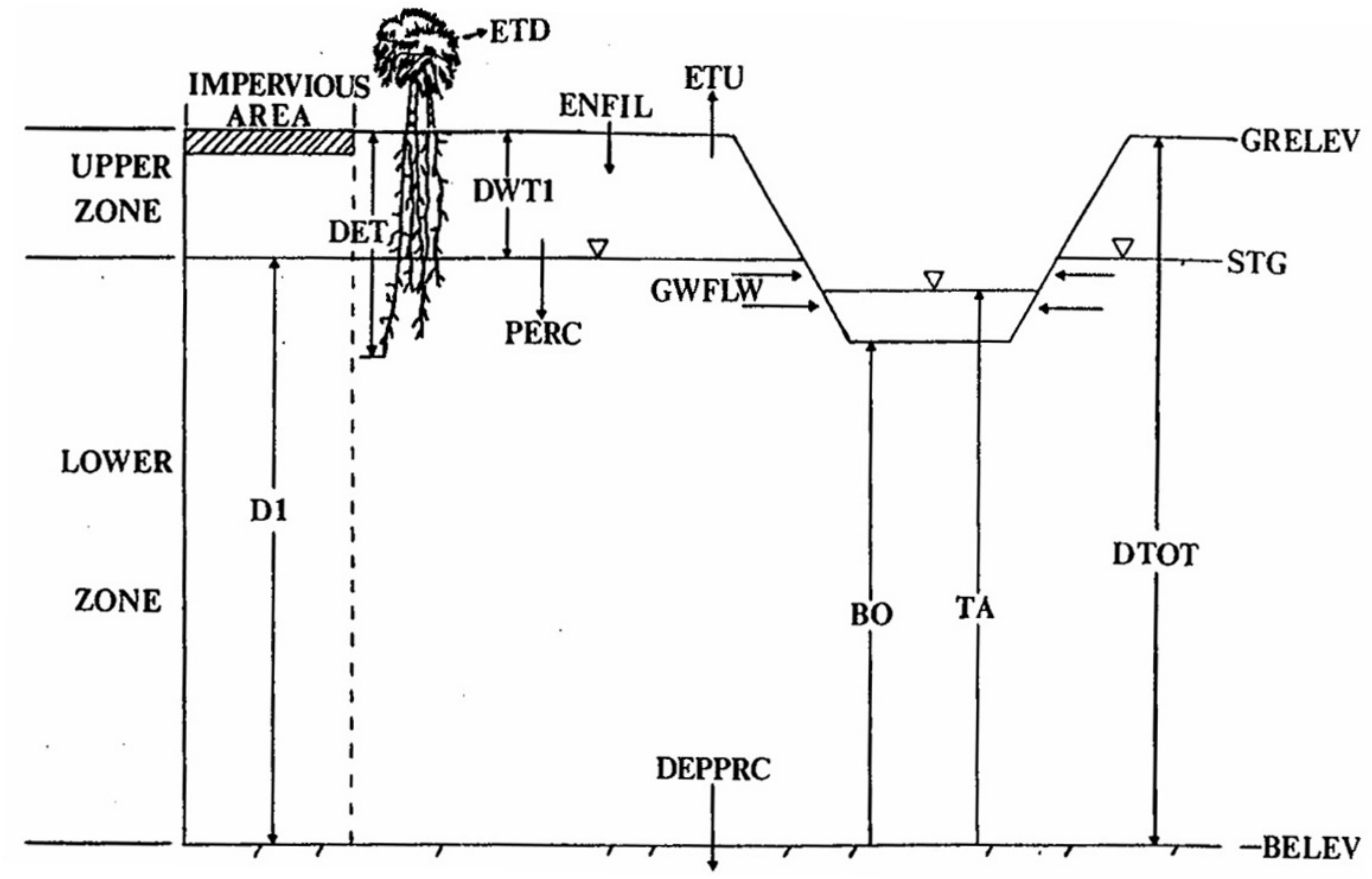


Figure X-1. GROUND parameters and conceptualization.

$$\left\{\left[\left[\begin{matrix}\text{PERC} - \text{ETD}\cdot\text{PAREA} - .5 \\ \left(\begin{matrix}\text{GWFLW} + \text{A1}\cdot(\text{D2}-\text{BC})^{\text{B1}} + \text{A3} \\ \cdot\,\text{D2}\cdot\text{TW} + \text{DEPPRC} + \text{DP}\cdot\text{D2}/\text{DTOT}\end{matrix}\right) - \text{TWFLW}\end{matrix}\right]\cdot\text{DELT}\right]\right\} \Big/ (\text{PR}-\text{TH2}) + \text{D1} \tag{X-3}$$

where

TH2 = end-of-time-step upper zone moisture content (fraction),
ENFIL = infiltration rate calculated in subroutine WSHED,
ETU = upper zone evapotranspiration rate,
PERC = percolation rate,
PAREA = pervious area divided by total area,
DELT = time step value,
D = beginning-of-time-step lower zone depth (elevation above a datum),
D2 = end-of-time-step lower zone depth,
TH = beginning-of-time-step upper zone moisture content,
DWT1 = beginning-of-time-step upper zone depth,
DTOT = total depth of upper and lower zone = D1+DWT1,
ETD = lower zone evapotranspiration rate,
GWFLW = beginning-of-time-step groundwater flow rate,
A1 = groundwater flow coefficient,
BC = bottom of channel depth (elevation above datum),
B1 = groundwater flow exponent,
DEPPRC = beginning-of-time-step deep percolation rate,
DP = a recession coefficient derived from interevent declines in the water table,
PR = porosity, and
TWFLW = channel water influence rate,
A3 = groundwater flow coefficient, and
TW = depth of water in channel (elevation above datum).

Moisture content (a fraction) is defined as the volume of moisture divided by the volume of solids plus voids. The maximum possible moisture content is the porosity; the minimum is the wilting point (discussed below). Solving equation X-1 for TH2 and using DWT1 = DTOT-D1, yields a much simpler form which is not a function of the unknown D2,

$$\text{TH2} = \left[(\text{ENFIL} - \text{ETU})\cdot\text{PAREA} - \text{PERC}\right]\cdot\text{DELT}/\text{DWT1} + \text{TH} \tag{X-4}$$

Equation X-4 is solved first, followed by a Newton-Raphson solution of equation X-2 or X-3. The sequencing will be described in more detail in a subsequent section, following a description of the various simulated processes.

Infiltration

Infiltration enters subroutine GROUND as the calculated infiltration from subroutine WSHED. As before in SWMM, either the Horton or Green-Ampt equation can be used to describe infiltration. For time steps where the water table has risen to the surface, the amount of infiltration that cannot be accepted is subtracted from RLOSS (infiltration plus surface evaporation) in subroutine WSHED. In the event that the infiltrated water is greater than the amount of storage available for that time step, the following equation is used to calculate the amount of infiltration that is not able to be accepted by the soil.

$$\mathrm{XSINFL} = \mathrm{ENVIL} \cdot \mathrm{DELT} - \mathrm{AVLVOL}/\mathrm{PAREA} \qquad \text{(X-5)}$$

where

XSINFL = excess infiltration over pervious area, and
AVLVOL = initial void volume in the upper zone plus total losses and outflows from the system for the time step.

The second condition exists because of the algebra in equations X-2, X-3 and X-4. As the water table approaches the surface, the end-of-time-step moisture value, TH2, approaches the value of porosity, which makes the denominator in equations X-2 and X-3 go towards zero. Since a denominator close to zero could result in an unrealistic value of D2, a different way of handling the calculations had to be implemented. When the initial available volume in the upper zone plus the volume of total outflows and losses from the system minus the infiltration volume is between zero and an arbitrary value of 0.0001 ft, several assumptions are made. First, end-of-time-step groundwater flow and deep percolation, which are normally found by iteration, are assumed to be equal to their respective beginning-of-time-step values. This step is taken to ensure that the final available volume remains in the previously mentioned range. Second, TH2 is set equal to an arbitrary value of 90% of porosity. It is believed that this will allow the TH2 value in this special case to be reasonably consistent with the TH^2 values juxtaposed to it in the time series. Third, D2 is set close to the total depth – the actual value of D2 depends on the value of porosity. Fourth, the amount of infiltration that causes the final available volume to exceed 0.0001 ft is calculated in the following equation and sent back to the surface in the form of a reduction in the term RLOSS in subroutine WSHED.

$$\mathrm{XSINFL} = \mathrm{ENFIL} \cdot \mathrm{DELT} + (.0001 - \mathrm{AVLVOL})/\mathrm{PAREA} \qquad \text{(X-6)}$$

Because of the way this special case is handled, it is possible for a falling water table to have the calculated excess infiltration be greater than the actual amount of infiltration. It is not desirable for the ground to pump water back onto the surface! Hence, the difference between the calculated excess infiltration and the actual infiltration is added to the infiltration value of the next time step. The number of occurrences of this situation in a typical run is very small, as is the computed difference that is passed to the next time step, so no problems should occur because of this solution.

Upper Zone Evapotranspiration

Evapotranspiration from the upper zone (ETU) represents soil moisture lost via cover vegetation and by direct evaporation from the pervious area of the subcatchment. No effort was made to derive a complex formulation of this process. The hierarchy of losses by evapotranspiration is as follows: 1) surface evaporation, 2) upper zone evapotranspiration, and 3) lower zone transpiration. Upper zone evapotranspiration is represented by the following equations,

ETMAX = VAP(MONTH) (X-7)

ETAVLB = ETMAX-EVAPO (X-8)

ETU = CET*ETMAX (X-9)

IF(TH.LT.WP.OR.ENFIL.GT.O.) ETU = 0. (X-10)

IF(ETU.GT.ETAVLB) ETU = ETAVLB (X-11)

where

ETMAX = maximum total evapotranspiration rate (input on card F1),
VAP(MONTH) = input maximum evapotranspiration rate for month MONTH,
ETAVLB = maximum upper zone evapotranspiration rate,
EVAPO = portion of ETMAX used by surface water evaporation,
CET = fraction of evapotranspiration apportioned to upper zone, and
WP = wilting point of soil.

The two conditions that make ETU equal to zero in equation X-10 are believed to simulate the processes actually occurring in the natural system. The first condition (moisture content less than wilting point) relates to the soil science interpretation of wilting point – the point at which plants can no longer extract moisture from the soil. The second condition (infiltration greater than zero) assumes that vapor pressure will be high enough to prevent additional evapotranspiration from the unsaturated zone.

Lower Zone Evapotranspiration

Lower zone evapotranspiration, ETD, represents evapotranspiration from the saturated zone over the pervious area. ETD is the last evapotranspiration removed, and is determined by the following depth-dependent equation and conditions.

ETD = (DET-DWT1)*ETMAX*(1-CET)/DET (X-12)

IF(ETD.GT.(ETAVLB-ETU)) ETD = ETAVLB-ETU (X-13)

IF(ETD.LT.0.) ETD = 0. (X-14)

where

ETD = lower zone evapotranspiration rate, and
DET = depth over which evapotranspiration can occur.

Since ETD is typically very small compared to other terms and has to be checked for certain conditions, it is assumed constant over the time step and not solved for in the iterative process.

Percolation

Percolation (PERC) represents the flow of water from the unsaturated zone to the saturated zone, and is the only inflow for the saturated zone. The percolation equation in the subroutine was formulated from Darcy's Law for unsaturated flow, in which the hydraulic conductivity, K, is a function of the moisture content, TH. For one-dimensional, vertical flow, Darcy's Law may be written

v = -K(TH) ⟐ dh/dz (X-15)

where

v = velocity (specific discharge) in the direction of z,
z = vertical coordinate, positive upward,
K(TH) = hydraulic conductivity,
TH = moisture content, and
h = hydraulic potential.

The hydraulic potential is the sum of the elevation (gravity) and pressure heads,

h = z + PSI (X-16)

where PSI = soil water tension (negative pressure head) in the unsaturated zone.

Equating vertical velocity to percolation, and differentiating the hydraulic potential, h, yields

Percolation = -K(TH) ⟐ (1+ dPSI/dz)
(X-17)

A choice is customarily made between using the tension, PSI, or the moisture content, TH, as parameters in equations for unsaturated zone water flow. Since the quantity of water in the unsaturated zone is identified by TH in previous equations, it is the choice here. PSI can be related to TH if the characteristics of the unsaturated soil are known. Thus, for use in equation X-17, the derivative is

dPSI/dz = dPSI/dTH ⟐ dTH/dz (X-18)

The slope of the PSI versus TH curve should be obtained from data for the particular soil under consideration. Relationships for a sand, sandy loam and silty loam are shown in Figures X-2, X-3 and X-4 (Laliberte et al., 1966). The data are based on laboratory tests of disturbed soil samples and illustrate only the desaturation (draining) characteristics of the soil. The relationship during the saturation (wetting) phase will ordinarily be different; when both the wetting and draining relationships are shown the curves usually illustrate a hysteresis effect. The figures also show the relationship between the hydraulic conductivity of the unsaturated soils and the moisture content. In some cases (e.g., sand), K(TH) may range through several orders of magnitude. Soils data of this type are becoming more readily available; for example, soil science departments at universities often publish such information (e.g., Carlisle et al., 1981). The data illustrated in Figures X-2, X-3 and X-4 are also useful for extraction of parameters for the Green-Ampt infiltration equations.

Equation X-17 may be approximated by finite differences as

$$\text{Percolation} = -K(TH) ⟐ [1+(\Delta TH/\Delta z)⟐(\Delta PSI/\Delta TH)]$$

(X-19)

For calculation of percolation, it is assumed that the gradient, $\Delta TH/\Delta z$, is the difference between moisture content TH in the upper zone and field capacity at the boundary with the lower zone, divided by the average depth of the upper zone, DWT1/2. Thus,

$$\text{Percolation} = -K(TH) ⟐ \{1+[(TH-FD)⟐2/DWT1] ⟐ PCO\}$$

(X-20)

where

FD = field capacity, and
PCO = $\Delta PSI/\Delta TH$ in the region between TH and FD.

PCO is obtained from data of the type of Figures X-2, X-3 and X-4.

Finally, the hydraulic conductivity as a function of moisture content is approximated functionally in the moisture zone of interest as

$$K(TH) = HKTH = HKSAT ⟐ EXP[(TH-PR)⟐HCO]$$

(X-21)

where

HKTH = hydraulic conductivity as a function of moisture content,
HKSAT = saturated hydraulic conductivity, and
HCO = calibration parameter.

HCO can be estimated by fitting the HKTH versus TH curve to the hydraulic conductivity versus moisture content curve, if such data are available (e.g., Figures X-2, X-3, X-4); three fits are shown in Figure X-5. The fits are not optimal over the entire data range because the fit is only performed for the high moisture content region between field capacity and porosity. If soils data are not available, HCÏ can be estimated by model calibration.

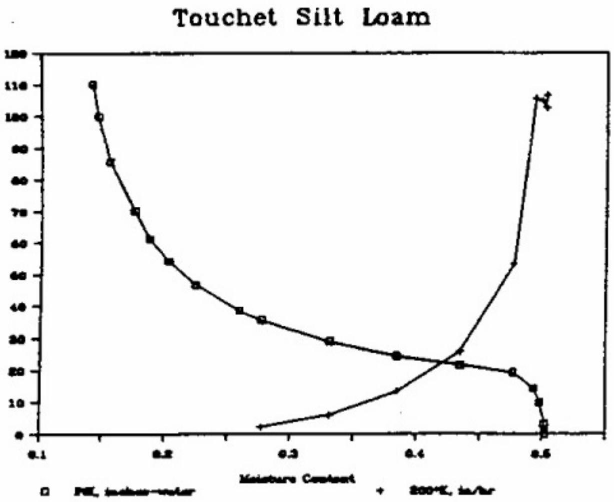


Figure X-2. Tension, PSI (squares, in. of water) and hydraulic conductivity, K (crosses, in./hr, K multiplied by 200) versus moisture content. Derived from data of Laliberte et al. (1966), Tables B-5 and C-3. Porosity = 0.503, temp. = 26.5° C, saturated hyd. conductivity = 0.53 in./hr.

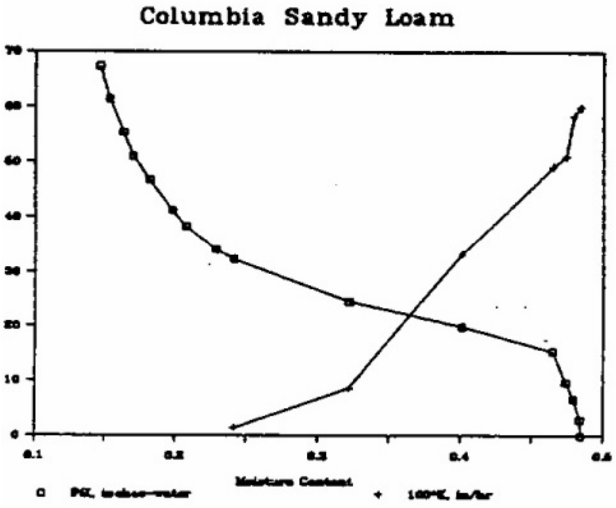


Figure X-3. Tension, PSI (squares, in. of water) and hydraulic conductivity, K (crosses, in./hr, K multiplied by 100) versus moisture content. Derived from data of Laliberte et al. (1966), Tables B-8 and C-5. Porosity = 0.485, temp. = 25.1 °C, saturated hyd. conductivity = 0.60 in./hr.

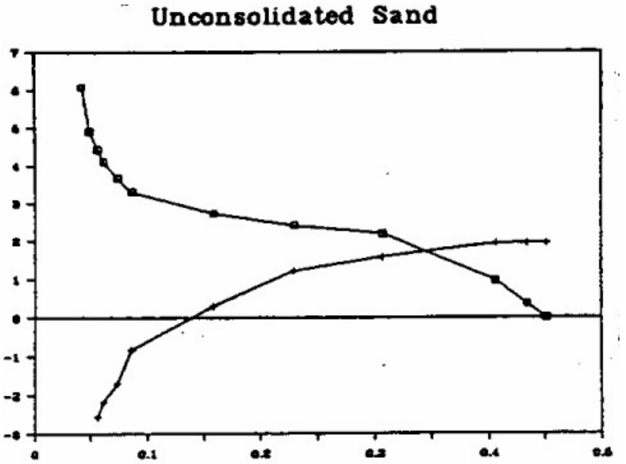


Figure X-4. Tension, PSI (squares, in. of water) and log-10 of hydraulic conductivity, K (crosses, K in in./hr) versus moisture content. Derived from data of Laliberte et al. (1966), Tables B-14 and C-11. Porosity = 0.452, temp. = 25.1°C, saturated hyd. conductivity = 91.5 in./hr.

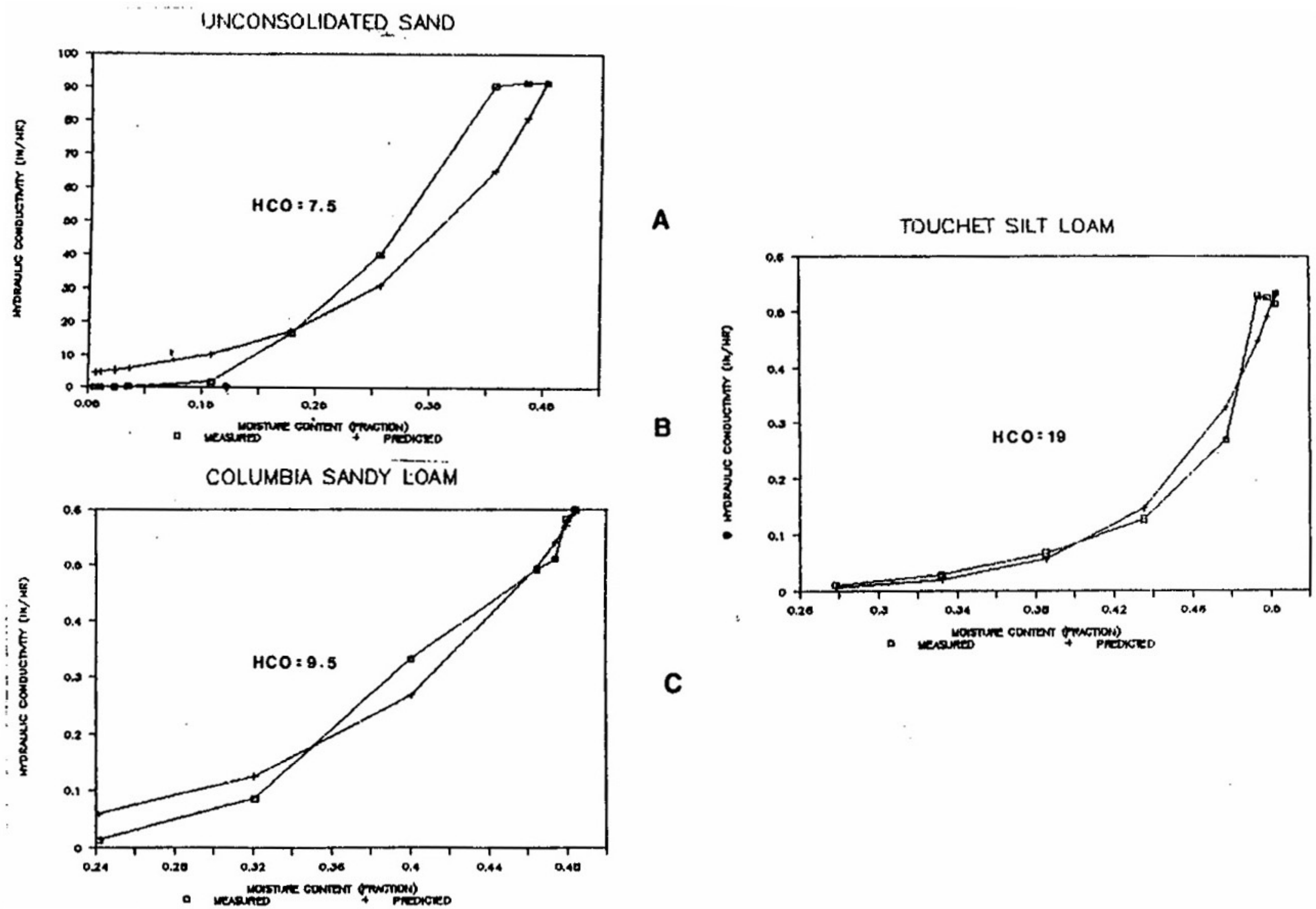


Figure X-5. Model representation and measured hydraulic conductivity curves for three types of soil.

Combining equations X-20 and X-21 gives the resulting percolation equation for the model,

$$\mathrm{PERC} = \mathrm{HKTH} \diamond [1+\mathrm{PCO}\diamond(\mathrm{TH}-\mathrm{FD})/(\mathrm{DWT1}/2)] \tag{X-22}$$

where PERC = percolation rate (positive downward) and is only nonzero when TH is greater than FD.

If data sources for parameters PCO and HCO are lacking, they may be estimated through the calibration process. On the basis of preliminary runs, the groundwater subroutine is relatively insensitive to changes in PCO and HCO, so a lack of extensive soils data should not discourage one from using the model.

If moisture content is less than or equal to field capacity, percolation becomes zero. This limit is in accordance with the concept of field capacity as the drainable soil water that cannot be removed by gravity alone (Hillel, 1982, p. 243). Once TH drops below field capacity, it can only be further reduced by upper zone evapotranspiration (to a lower bound of the wilting point).

The percolation rate calculated by equation X-22 will be reduced by the program if it is high enough to drain the upper zone below field capacity or make the iterations for D2 converge to an unallowable value. Also, since checks must be made on PERC, it is assumed to be constant over the time step and therefore not determined through an iterative process.

Field Capacity and Wilting Point

These parameters are used for demarcations for percolation and ET. Field capacity, FC, is usually considered to be the amount of water a well-drained soil holds after free water has drained off, or the maximum amount it can hold against gravity (SCS, 1964; Linsley et al., 1982). This occurs at soil moisture tensions (see further discussion below) of from 0.1 to 0.7 atmospheres, depending on soil texture. Moisture content at a tension of 1/3 atmosphere is often used. The wilting point (or permanent wilting point), WP, is the soil moisture content at which plants can no longer obtain enough moisture to meet transpiration requirements; they wilt and die unless water is added to the soil. The moisture content at a tension of 15 atmospheres is accepted as a good estimate of the wilting point (SCS, 1964; Linsley et al., 1982). The general relationship among soil moisture parameters is shown in Figure X-6 (SCS, 1964).

Data for FC and WP are available from the SCS, agricultural extension offices and university soil science departments. Generalized data are shown in Table X-1, as derived from Linsley et al. (1982, p. 179).

Deep Percolation

Deep percolation represents a lumped sink term for unquantified losses from the saturated zone. The two primary losses are assumed to be percolation through the confining layer and lateral outflow to somewhere other than the receiving water. The arbitrarily chosen equation for deep percolation is

$$\mathrm{DEPPRC} = \mathrm{DP} \diamond \mathrm{D1}/\mathrm{DTOT}$$

(X-23)

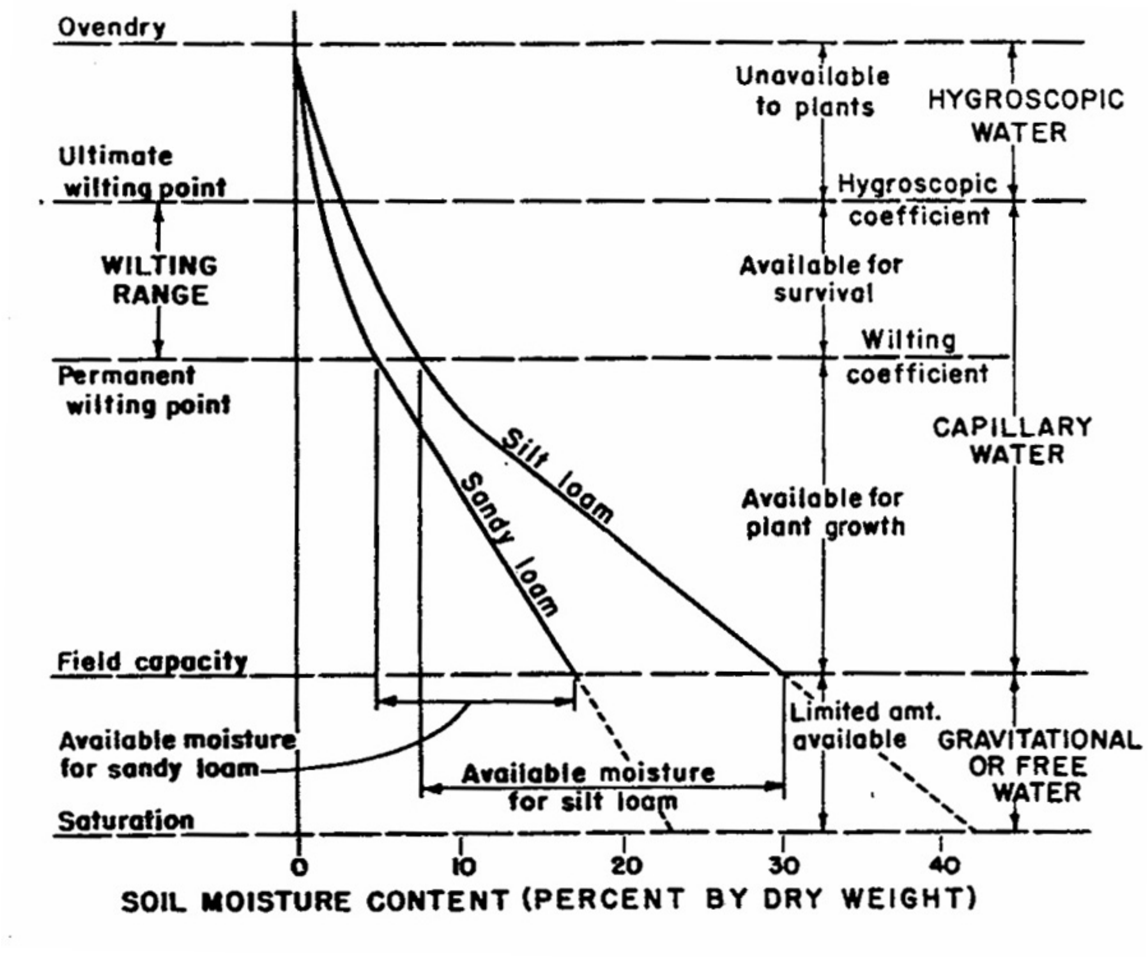


Figure X-6. Kinds of water in soil (SCS, 1964). Note that silt loam contains more than twice as much readily available water than sandy loam.

Table X-1. Volumetric Moisture Content at Field Capacity and Wilting Point (derived* from Linsley et al., 1982, Table 6-1.)

Soil Type	Field Capacity	Wilting Point
Sand	0.08	0.03
Sandy loam	0.17	0.07
Loam	0.26	0.14
Silt Loam	0.28	0.17
Clay loam	0.31	0.19
Clay	0.36	0.26
Peat	0.56	0.30

*Fraction moisture content = fraction dry weight × dry density / density of water.

where

DEPPRC = beginning-of-time-step deep percolation rate, and
DP = a recession coefficient derived from interevent water table recession curves.

The ratio of D1 to DTOT allows DEPPRC to be a function of the static pressure head above the confining layer. Although DEPPRC will be very small in most cases, it is included in the iterative process so that an average over the time step can be used. By doing this, large continuity errors will be avoided should DEPPRC be set at a larger value.

Groundwater Discharge

Functional Form

Groundwater discharge represents lateral flow from the saturated zone to the receiving water. The flow equation takes on the following general form:

$$GWFLW = A1 ⟐ (D1-BC)^{B1} - TWFLW + A3 ⟐ D1 ⟐ TW \tag{X-24}$$

and

$$TWFLW = A2 ⟐ (TW-BC)^{B2} \tag{X-25}$$

where

GWFLW = beginning-of-time-step groundwater flow rate (per subcatchment area,
TWFLW = channel water influence flow rate (per subcatchment area),
A1,A2 = groundwater and channel water influence flow coefficients,

A3 = coefficient for cross-product,
B1,B2 = groundwater and tailwater influence flow exponents,
BC = elevation of bottom of channel, and
TW = elevation of water in channel.

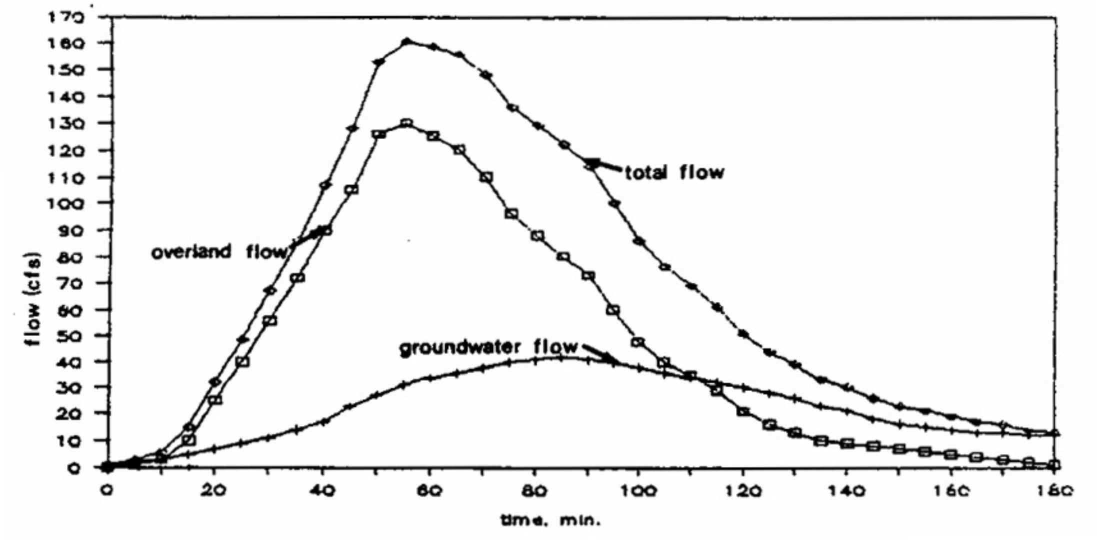
If D1 is less than BC or TW, GWFLW is set equal to zero. In addition, if TW = BC and B2 = 0, then the indeterminant form of zero raised to the zero power in equation X-25 is set equal to 1.0 by the program. The functional form of equations X-24 and X-25 was selected in order to be able to approximate various horizontal flow conditions, as will be illustrated below.

Since groundwater flow can be a significant volume, an average flow each time step is found by iteration using equation X-2 or X-3. Groundwater flows can be routed to any previously defined inlet, trapezoidal channel, or pipe, al-lowing the user to isolate the various components of the total hydrograph, as shown in Figure X-7. That is, the groundwater flow does not have to be routed to the same destination as the overland flow from the subcatchment.

The effects of channel water on groundwater flow can be dealt with in two different manners. The first option entails setting TW (elevation of water surface in the channel) to a constant value greater than or equal to BC (bottom-of-channel elevation) and A2, B2 and/or A3 to values greater than zero. If this method is chosen, then the user is specifying an average tailwater influence over the entire run to be used at each time step.

The second option makes the channel water elevation, TW, equal to the elevation of water in an actual channel (trapezoidal channel or circular pipe). For this option, the groundwater must be routed to a trapezoidal channel or pipe – not an inlet. The depth of water in the channel (TW - BC) at each time step is then determined as the depth in the channel or pipe from the previous time step. (It is assumed that the bottom of the channel is at the elevation BC.) The beginning-of-time-step depth must be used to avoid complex and time-consuming iterations with the coupled channel discharge equations in subroutine GUTTER. Unfortunately, because of this compromise the groundwater flow may pulsate as D1 oscillates between just above and just below elevation TW. This pulsing may introduce errors in continuity and is, of course,

Figure X-7. Hydrograph of total flow and its two major components.



unrepresentative of the actual system. Shorter time steps and larger or less steep channels (reducing the response of the channel) can be used to reduce the pulses. Also, caution must be taken when selecting A1, B1, A2, B2 and A3 so that GWFLW cannot be negative. Although this may occur in the actual system and represent recharge from the channel, there is currently no means of representing this reverse flow and subtracting it from the channel. One way of assuring that this cannot happen is to make A1 greater than or equal to A2 and B1 greater than or equal to B2, and A3 equal to zero.

Because of the general nature of the equation, it can take on a variety of functional forms. For example, a linear reservoir can be selected by setting B1 equal to one and A2 and A3 equal to zero. Two drainage examples are illustrated below.

Example: Infiltration and Drainage to Adjacent Channel

Under the assumption of uniform infiltration and horizontal flow by the Dupuit-Forcheimer approximation, the relationship between water table elevation and infiltration for the configuration shown in Figure X-8 is (Bouwer, 1978, p. 51)

$$K\left(h_1^2 - h_2^2\right) = L^2 f \tag{X-26}$$

where

f = infiltration rate,
K = hydraulic conductivity, and other parameters are as shown on Figure X-8.

Before matching coefficients of equations X-24 and X-25 to equation X-26, it should be recognized that the water table elevation in SWMM, D1, represents an average over the catchment, not the maximum at the "upstream" end that is needed for h_1 in equation X-26. Let D1 be the average head,

$$D1 = \left(h_1 + h_2\right)/2 \tag{X-27}$$

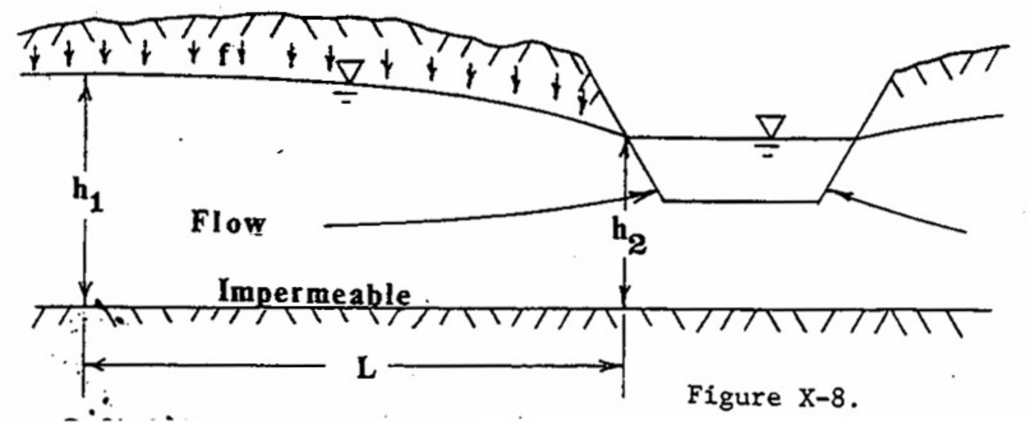


Figure X-8. Definition sketch for Dupuit-Forcheimer approximation for drainage to adjacent channel.

Substituting $h_1 = 2\ D1 - h_2$ into equation X-26 gives, after algebra

$$\left(D1^2 - D1\ h_2\right) 4K/L^2 = f \tag{X-28}$$

from which a comparison with equations X-24 and X-25 yields $A1 = A3 = 4K/L^2$, $A2 = 0$, and $B1 = 2$. Note that GWFLW has units of flow per unit area, or length per time, which are the units of infiltration, f, in equation X-28.

Example: Hooghoudt's Equation for Tile Drainage

The geometry of a tile drainage installation is illustrated in Figure X-9. Hooghoudt's relationship (Bouwer, 1978, p. 295) among the indicated parameters is

$$f = \left(2D_e + m\right) 4Km/L^2 \tag{X-29}$$

where D_e = effective depth of impermeable layer below drain center, and other parameters are defined in Figure X-9. D_e is less than or equal to b_o in Figure X-9 and is a function of b_o, drain diameter, and drain spacing, L; the complicated relationship is given by Bear (1972, p. 412) and graphed by Bouwer (1978, p. 296). The maximum rise of the water table, $M = h_1 - b_o$. Once again approximating the average water table depth above the impermeable layer by $D1 = 2h_1 - b_o$, equation X-29 can be manipulated to

$$\begin{aligned} f &= \left[\left(h_1 - b_o\right)^2 + 2D_e\left(h_1 - b_o\right)\right] 4K/L^2 = \\ &= \left[\left(D1 - b_o\right)^2 + D_e D1 - D_e b_o\right] 16K/L^2 \end{aligned} \tag{X-30}$$

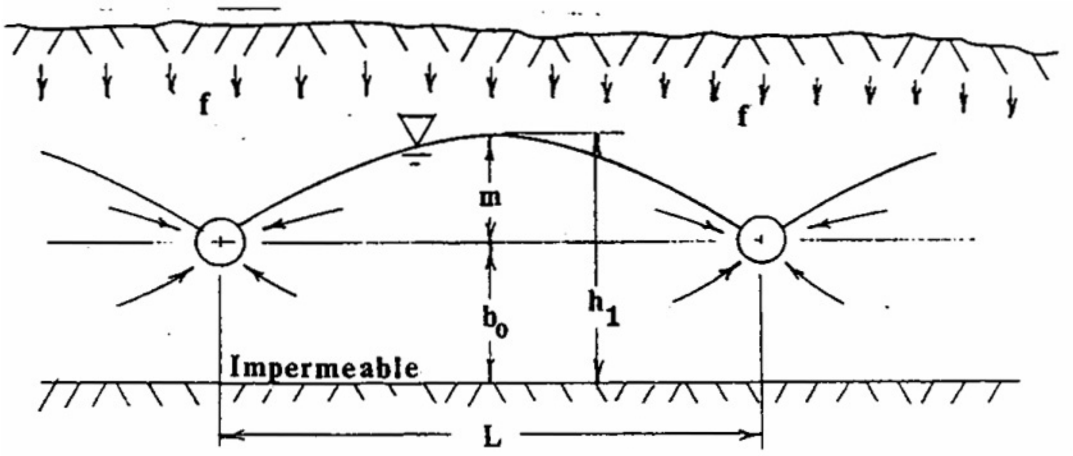


Figure X-9. Definition sketch for Hooghoudt's method for flow to circular drains.

Comparing equation X-30 with equations X-24 and X-25 yields

$A1 = 16K/L^2$,

$B1 = 2$

$A2 = 16KD_e b_o/L^2$

$B2 = 0$

$A3 = 16KD_e/TW\ L^2$

and TW = BC = b_o = constant during the simulation. The equivalent depth, D_e, must be obtained from the sources indicated above. The mathematics of drainage to ditches or circular drains is complex» several alternative formulations are described by van Schilfgaarde (1974).

Limitations

Since the moisture content of the unsaturated zone is taken as an average over the entire zone, the shape of the moisture profile is totally obscured. Therefore, infiltrated water cannot be modeled as a diffusing slug moving down the unsaturated zone, as is the case in the real system. Furthermore, water from the capillary fringe of the saturated zone cannot move upward by diffusion or "suction" into the unsaturated zone.

The simplistic representation of subsurface storage by one unsaturated "tank" and one saturated "tank" limits the ability of the user to match non-uniform soil columns. Another limitation is the assumption that the infiltrated water is spread uniformly over the entire catchment area, not just over the pervious area. In addition, just as for surface flow, groundwater may not be routed from one subcatchment to another. The tendency of the tailwater influence to cause pulses if TW-BC is equated to the dynamic water depth in the adjacent channel is a limitation that will remain until the channel flow and subsurface flow are solved simultaneously using a set of coupled equations. Such a solution would also permit reverse flow or recharge from the channel to be simulated.

Finally, water quality is not simulated in any of the subsurface routines. If water quality is simulated in RUNOFF and the subsurface flow routines activated, any loads entering the soil will "disappear," as if the soil provides 100 percent treatment.

Subroutine Configuration

A flowchart of the subroutine configuration is presented in Figure X-10. Initial values and constants used in subroutine GROUND come mostly from subroutine GRIN, designed specifically to read in these values. Subroutine GRIN is called by RHYDRO. Other necessary values are transferred during the CALL statement and from previously calculated values stored in COMMON.

Subroutine GROUND first initializes pertinent parameters, then calculates fluxes that are constant over the time step. Beginning-of-time-step fluxes are calculated next, and the value of percolation is checked to ensure that it will not raise the water table above the ground surface.

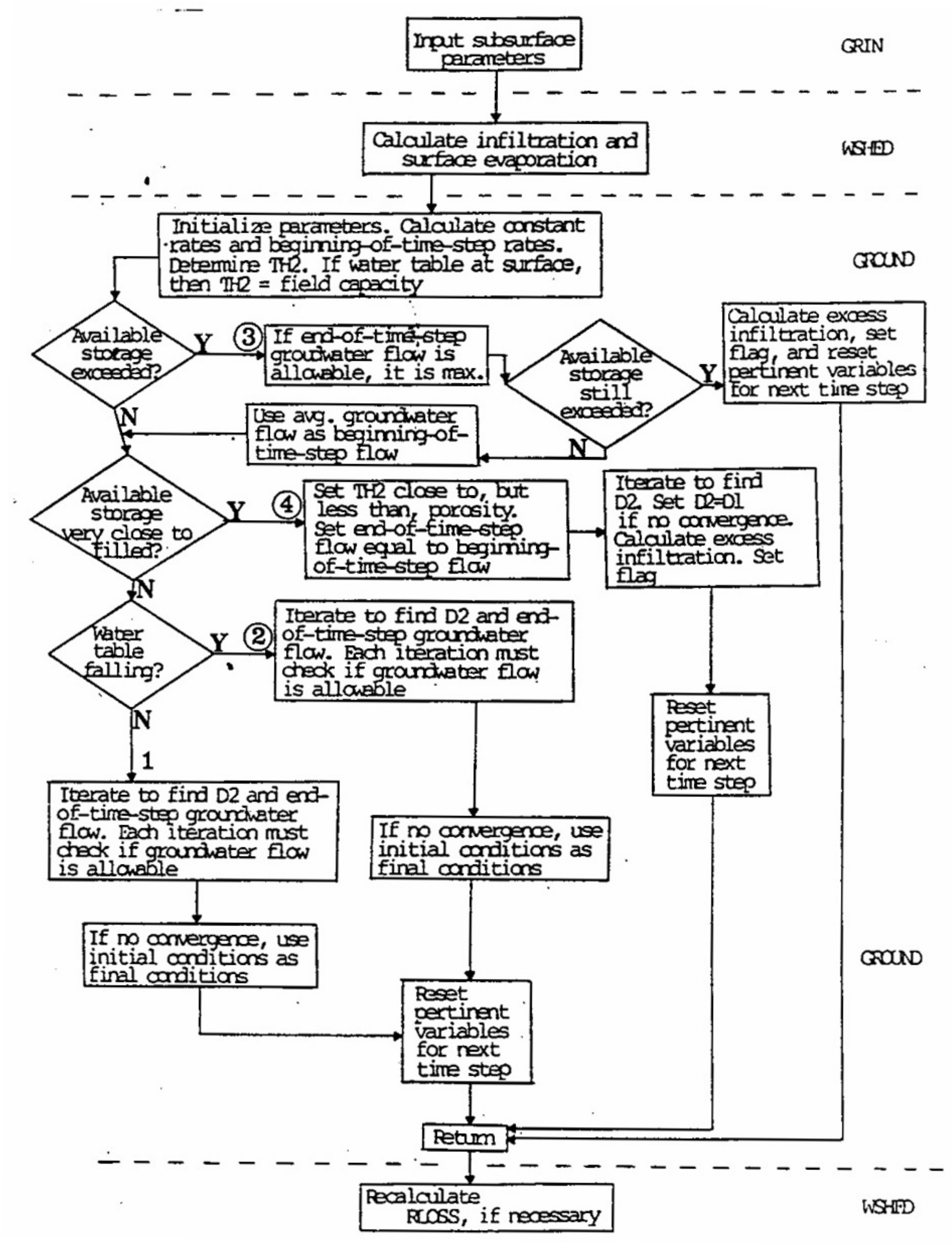


Figure X-10. Flowchart of subsurface and directly connected surface calculations.

After other constants are calculated and TH² is determined from equation X-4, the program branches to one of four areas. The first and second areas are for rising and falling water tables, equations X-2 and X-3, respectively. In both cases, Newton-Raphson iteration is used to solve simultaneously for the final groundwater flow, depth of lower zone, and deep percolation. Each iteration checks whether or not groundwater flow is possible (D1 greater than or equal to TW and BC). After the iterations converge, final conditions are set as the next time step's initial conditions.

In the event of saturation (D1 = DTOT), the third area sets D2 equal to DTOT, sets final ground-water flow equal to the maximum possible (D2 = DTOT), and assumes DEPPRC remains constant over the time step. Any excess infiltration is then routed back to the surface for overland flow calculations, and final conditions are set for the next initial conditions. However, if the maximum groundwater flow and DEPPRC rates permit some infiltration into the subsurface zone, the initial and final groundwater flow are averaged to be used as the new initial ground-water flow, and the program branches back to iterate for the solution. This pathway will rarely, if ever, be taken, but must be included to minimize possible continuity errors.

In the event the available storage in the unsaturated zone is less than 0.0001 ft, the fourth area sets TH2 equal to 90% of porosity and D2 close to DTOT, and returns any infiltration to the surface that causes the final unfilled upper zone volume to be greater than 0.0001 ft. This is to avoid oscillations as the water table hovers near the ground surface. Again, final conditions are then set as the next time step's initial conditions.

Examples

Cypress Creek Calibration and Verification

Two examples will illustrate the use of the new subroutine. The first example is a year-long simulation of a 47 mi^2 portion of the 117 mi^2 Cypress Creek Watershed in Pasco County, Florida, about 30 miles north of Tampa (Figure X-11). The region has been studied in relation to the interaction of surface water and ground water under the stress of heavy pumping and drainage activities in the area (Heaney et al., 1986). The watershed is characterized by sandy soils in which most water movement follows subsurface pathways. For this example, only a single 47 mi^2 area above State Road 52 (Figure X-11) and tributary to the USGS gage at San Antonio has been simulated.

Twenty-four parameters on three additional H-cards are required for each subsurface subcatchment. (Many of these can be ignored or set to zero during most runs» not all parameters are required for all runs.) Input parameters are echoed on two new pages of output that immediately follow the surface subcatchment information. Figure X-12 is an example of these two new pages; the values in Figure X-12 are from the calibration run on Cypress Creek. In addition to the new output just mentioned, a subsurface continuity check is provided in addition to the existing surface continuity check. An example of this amended page is shown in Figure X-13.

The simulation is divided into two six-month runs: the first six months for calibration, and the second six months for verification. Since Cypress Creek is a very flat, pervious area with well-drained soils and very little surface flow, it was modeled in a manner that would allow groundwater flow to account for most of the flow in the channel. In other words, the groundwater parameters represented by far the most critical part of the calibration. The only complete rainfall data for the calibration period are for the gage at St. Leo, out of the catchment to the east. Although these data are in daily increments, the calibration process was relatively

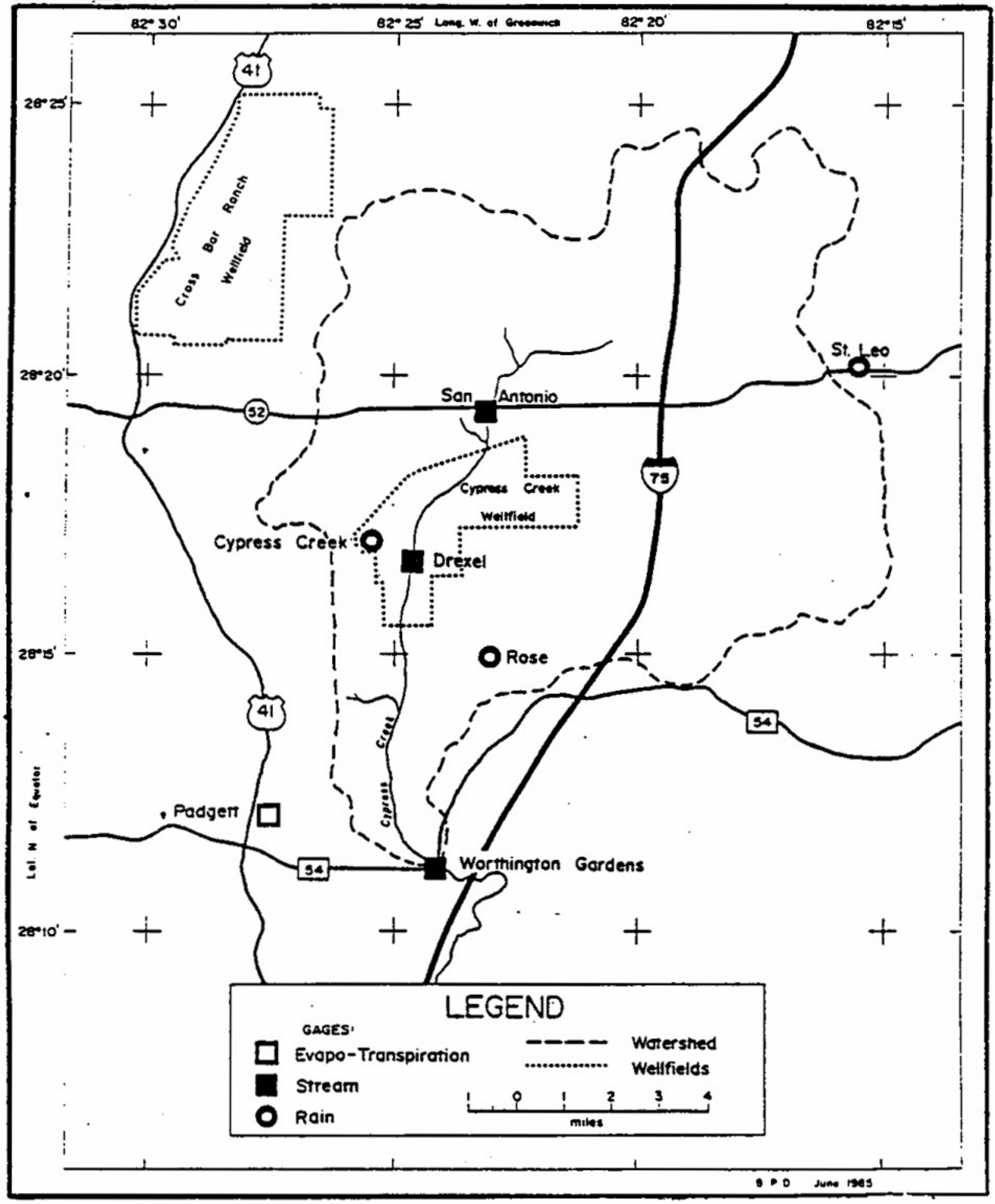
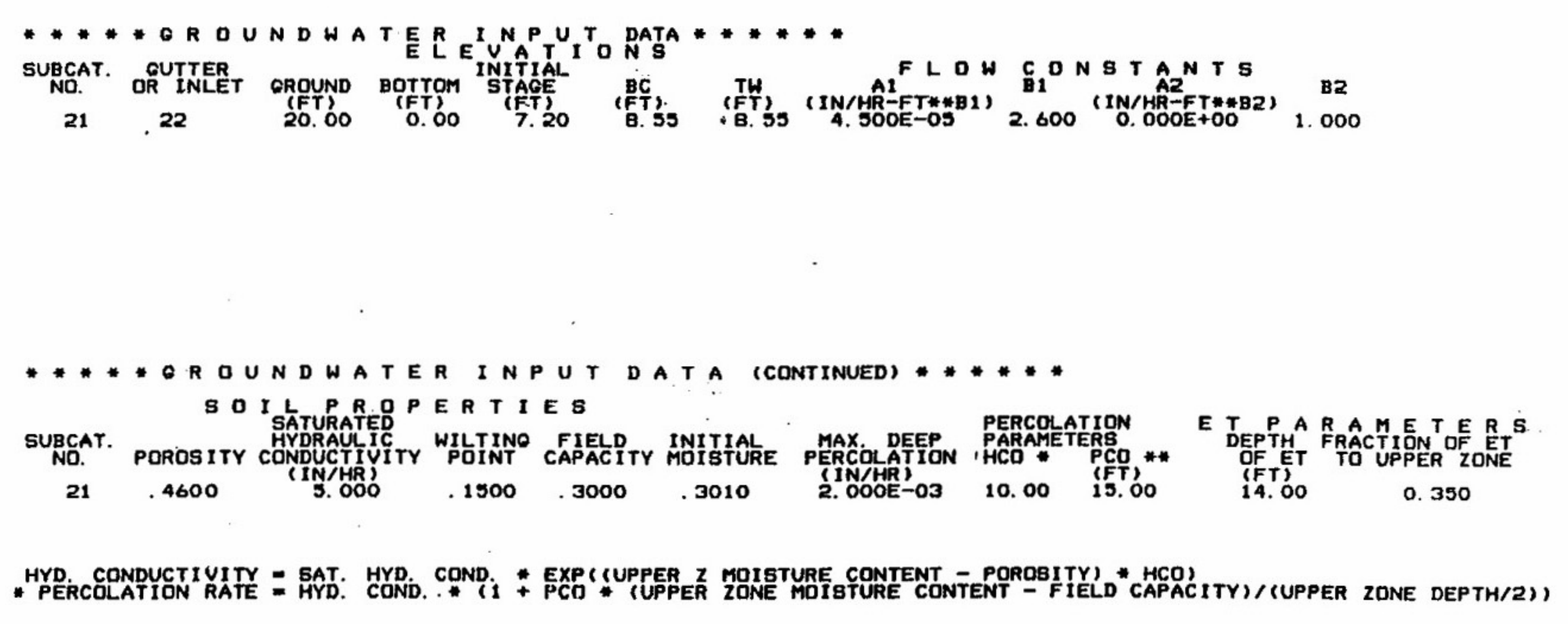
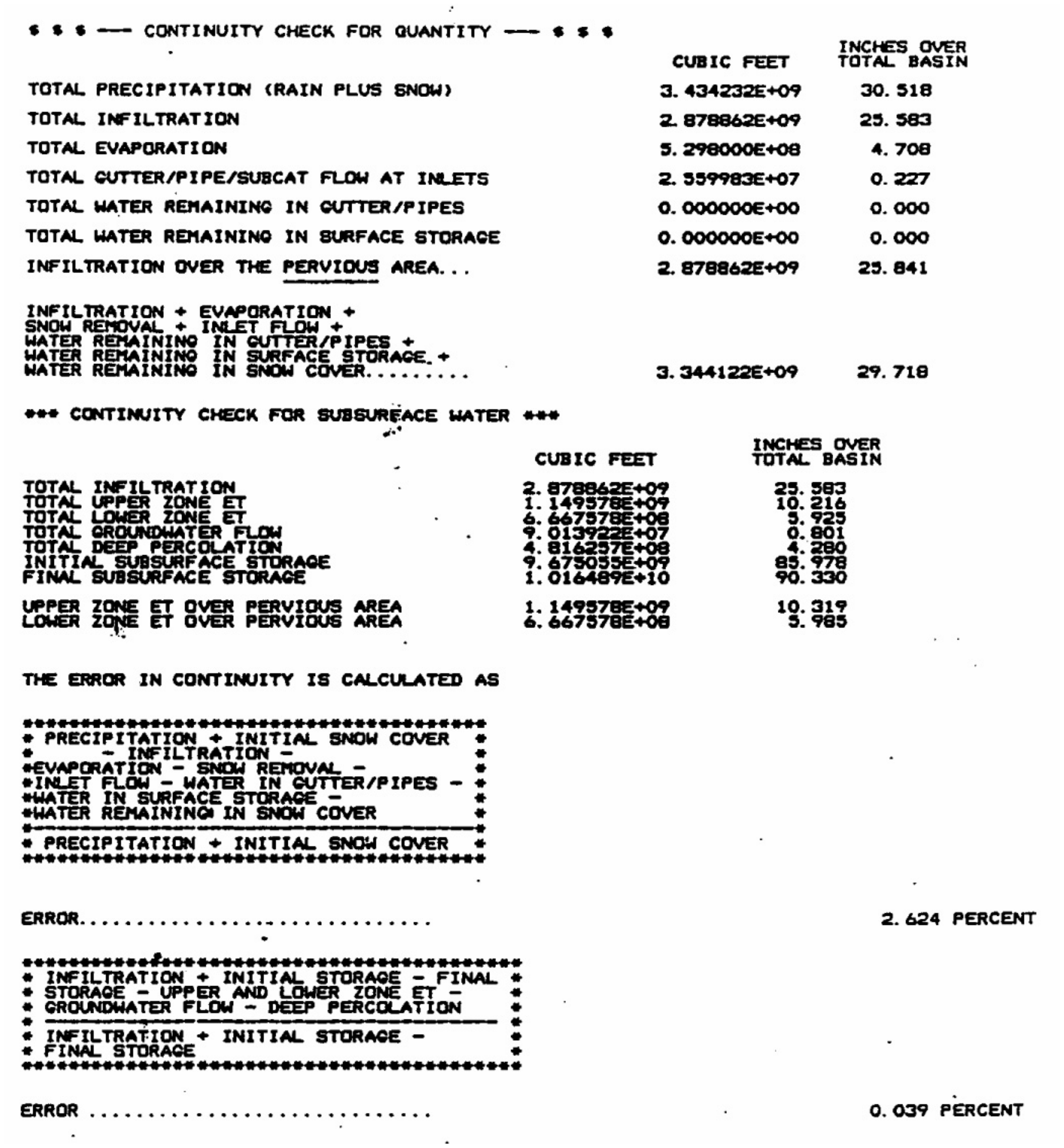


Figure X-11. Map of Cypress Creek watershed in Pasco County, Florida (Heaney et al., 1986).



```
* * * * * G R O U N D W A T E R   I N P U T   DATA * * * * * *
                                   E L E V A T I O N S
SUBCAT.   GUTTER                      INITIAL                                      F L O W   C O N S T A N T S
  NO.    OR INLET   GROUND   BOTTOM   STAGE       BC        TW            A1            B1           A2             B2
                     (FT)     (FT)    (FT)       (FT)      (FT)   (IN/HR-FT**B1)             (IN/HR-FT**B2)
   21       22      20.00     0.00     7.20      8.55      8.55     4.500E-05       2.600     0.000E+00       1.000




* * * * * G R O U N D W A T E R   I N P U T   D A T A   (CONTINUED) * * * * * *

              S O I L   P R O P E R T I E S
                         SATURATED                                                       PERCOLATION       E T   P A R A M E T E R S
SUBCAT.                  HYDRAULIC     WILTING   FIELD      INITIAL      MAX. DEEP       PARAMETERS          DEPTH   FRACTION OF ET
  NO.      POROSITY   CONDUCTIVITY     POINT    CAPACITY   MOISTURE     PERCOLATION     HCO *    PCO **       OF ET   TO UPPER ZONE
                         (IN/HR)                                          (IN/HR)                  (FT)         (FT)
   21        .4600        5.000         .1500     .3000      .3010       2.000E-03      10.00    15.00        14.00        0.350


HYD. CONDUCTIVITY = SAT. HYD. COND. * EXP((UPPER Z MOISTURE CONTENT - POROSITY) * HCO)
* PERCOLATION RATE = HYD. COND. * (1 + PCO * (UPPER ZONE MOISTURE CONTENT - FIELD CAPACITY)/(UPPER ZONE DEPTH/2))
```

Figure X-12. Subsurface input data for Cypress Creek calibration.



```
$ $ $ ---- CONTINUITY CHECK FOR QUANTITY ---- $ $ $
                                                                 INCHES OVER
                                                  CUBIC FEET     TOTAL BASIN

TOTAL PRECIPITATION (RAIN PLUS SNOW)             3.434232E+09      30.518

TOTAL INFILTRATION                               2.878862E+09      25.583

TOTAL EVAPORATION                                5.298000E+08       4.708

TOTAL GUTTER/PIPE/SUBCAT FLOW AT INLETS          2.559983E+07       0.227

TOTAL WATER REMAINING IN GUTTER/PIPES            0.000000E+00       0.000

TOTAL WATER REMAINING IN SURFACE STORAGE         0.000000E+00       0.000

INFILTRATION OVER THE PERVIOUS AREA...           2.878862E+09      25.841

INFILTRATION + EVAPORATION +
SNOW REMOVAL + INLET FLOW +
WATER REMAINING IN GUTTER/PIPES +
WATER REMAINING IN SURFACE STORAGE +
WATER REMAINING IN SNOW COVER.........           3.344122E+09      29.718

*** CONTINUITY CHECK FOR SUBSURFACE WATER ***
                                                          INCHES OVER
                                        CUBIC FEET        TOTAL BASIN

TOTAL INFILTRATION                      2.878862E+09         25.583
TOTAL UPPER ZONE ET                     1.149578E+09         10.216
TOTAL LOWER ZONE ET                     6.667578E+08          5.925
TOTAL GROUNDWATER FLOW                  9.013922E+07          0.801
TOTAL DEEP PERCOLATION                  4.816257E+08          4.280
INITIAL SUBSURFACE STORAGE              9.675055E+09         85.978
FINAL SUBSURFACE STORAGE                1.016489E+10         90.330

UPPER ZONE ET OVER PERVIOUS AREA        1.149578E+09         10.319
LOWER ZONE ET OVER PERVIOUS AREA        6.667578E+08          5.985

THE ERROR IN CONTINUITY IS CALCULATED AS

*****************************************
* PRECIPITATION + INITIAL SNOW COVER    *
*        - INFILTRATION -               *
*EVAPORATION - SNOW REMOVAL -           *
*INLET FLOW - WATER IN GUTTER/PIPES -   *
*WATER IN SURFACE STORAGE -             *
*WATER REMAINING IN SNOW COVER          *
*---------------------------------------*
* PRECIPITATION + INITIAL SNOW COVER    *
*****************************************

ERROR..............................                          2.624 PERCENT

*******************************************
* INFILTRATION + INITIAL STORAGE - FINAL  *
* STORAGE - UPPER AND LOWER ZONE ET -     *
* GROUNDWATER FLOW - DEEP PERCOLATION     *
* --------------------------------------- *
* INFILTRATION + INITIAL STORAGE -        *
* FINAL STORAGE                           *
*******************************************

ERROR .............................                          0.039 PERCENT
```

Figure X-13. Continuity check for surface and subsurface for Cypress Creek calibration. The relatively large surface continuity error does not actually exist; it comes from a double accounting of the groundwater flow – a problem that has been fixed.

simple because of the existence of both flow and shallow-well stage data. In addition, only one subcatchment (surface and subsurface) was used, since the purpose of this example was only to illustrate the use of subroutine GROUND, not to provide a thorough simulation.

Figure X-14 shows the predicted groundwater flow hydrograph and the measured total flow hydrograph for the calibration run, and Figure X-15 shows a comparison of the predicted total flow hydrograph to the measured total flow hydrograph for the calibration run. Predicted and measured stages for the calibration can be seen in Figure X-16. The calibration is not especially remarkable in light of the lack of detailed rainfall data for the 47 mi^2 area. The predicted stage hydrograph does not exhibit the short-term variations that are measured, primarily because of the lack of spatial detail in the rain. In addition, the measured stages are at one well near the center of the modeled area and would be expected to show more variation than would the average water table over the 47 mi^2 simulated by SWMM. The existence of more than one gage in the 47 square miles of the catchment and shorter increment rainfall data would have improved the fit seen in Figure X-16. Figures X-17, X-18 and X-19 show similar results for the verification runs. In general, the average recession of the water table is simulated accurately, but not the fluctuations.

Hypothetical Catchment with High Water Table

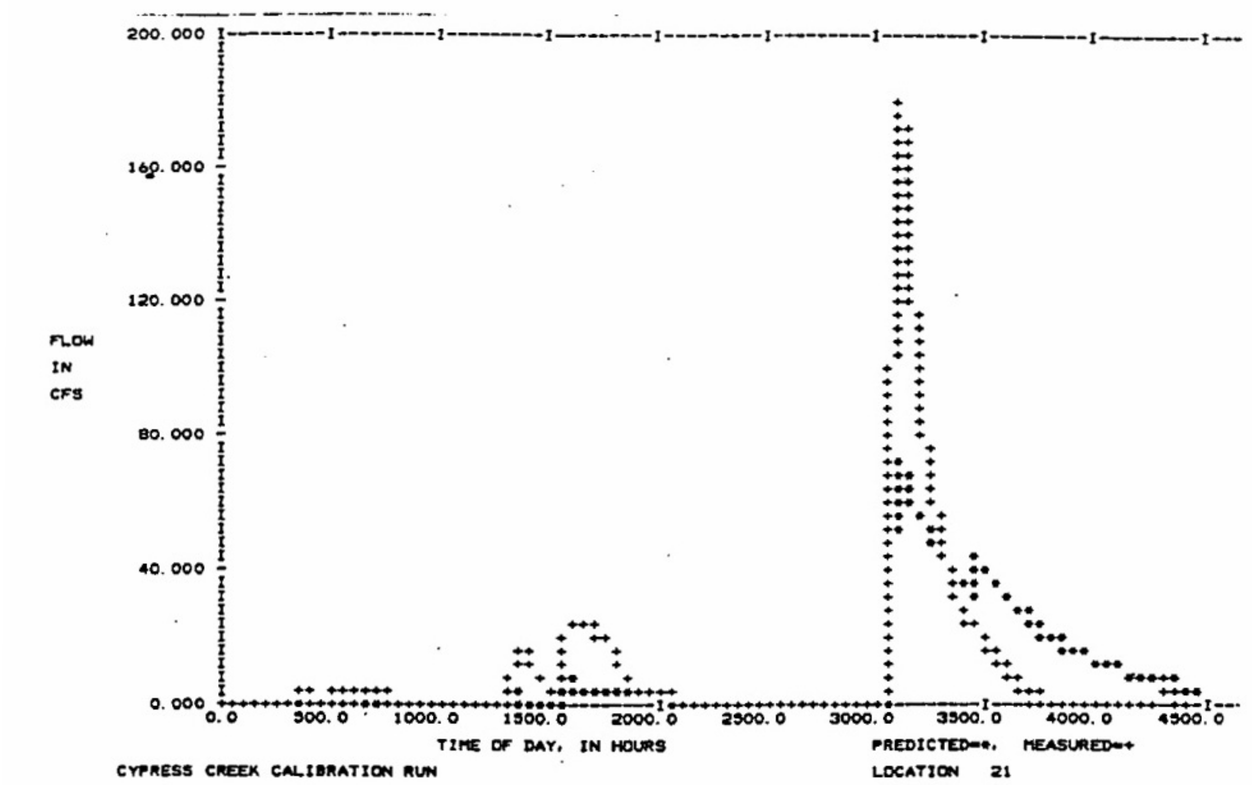
The second example is a 100 ac hypothetical subcatchment with the same soil properties as Cypress Creek and a water table that is initially one foot from the surface. The 10-yr SCS Type II design storm for Tallahassee, Florida, is used for the rainfall input (Figure X-20). This storm is characterized by very high rainfall between hours 11 and 12.

In order to illustrate the influence of a high water table, runs were made with and without the groundwater subroutine. Table X-2 shows the disposition of the rainfall when a high water table is simulated as opposed to when it is ignored. Note that evaporation is about the same, and the difference in the amount of infiltrated water shows up as a direct difference in surface runoff. (The runs were halted before all water had run off.) The two hydrographs and the corresponding water table (for the run in which it is simulated) are shown in Figure X-21. A larger difference in peak flows would have resulted if the flows had not been routed to a very large channel. Also, note that the two hydrographs are identical until about hour eleven into the simulation, when the simulated water table rises to the surface.

Execution time on the IBM 3033 mainframe increased from 0.32 CPU seconds without the groundwater simulation to 0.42 CPU seconds with the groundwater simulation. Thus, some additional computational expense can be expected.

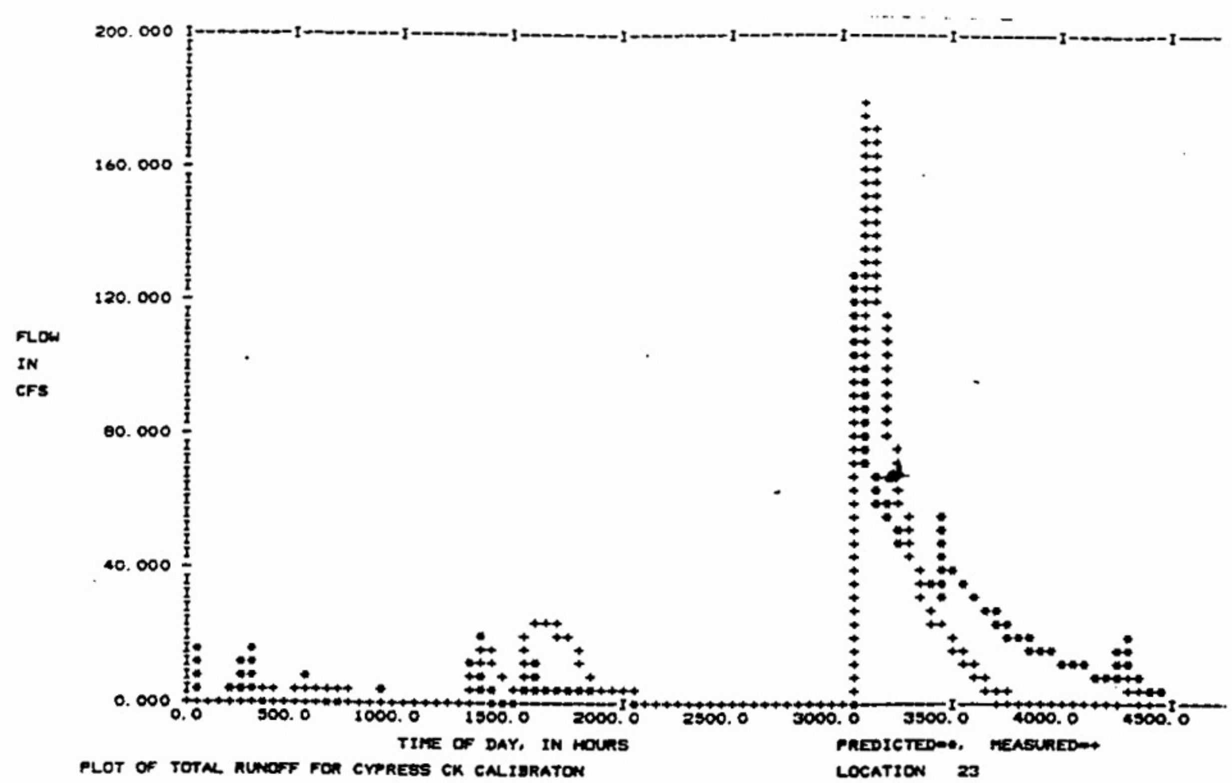
Conclusions

Although the subroutine is fairly simple in design and has several limitations, the new groundwater subroutine should increase the applicability of SWMM. Preliminary test runs have determined it to be accurate in the simulation of water table stage and groundwater flow. Further calibration and verification tests need to be done on other areas to confirm these preliminary results. Also, estimation of parameters, although fairly numerous, appears to be relatively uncomplicated. In addition, parameters are physically based and should be able to be estimated from soils data. The flexible structure of the algorithm should permit a more realistic simulation of catchments in which a major hydrograph component is via subsurface pathways.



```
HYDROGRAPH STATISTICS FOR LOCATION  21

                    VOLUME                  PEAK FLOW                 DURATION                  NO.
              CUBIC FEET    INCHES      TIME,HR   FLOW,CFS     START,HR  END,HR   LENGTH,HR    POINTS
PREDICTED,    0. 14547E+09   1. 293    3105. 000   73. 242      0. 000 4430. 000 4430. 000      194
TOTAL TIME
MEASURED,     0. 16359E+09   1. 454    3120. 000  180. 000      0. 000 4392. 000 4392. 000      184
TOTAL TIME
PREDICTED,    0. 14463E+09   1. 285    3105. 000   73. 242      0. 000 4393. 000 4393. 000      19
OVERLAPPING
TIME
MEASURED,     0. 16359E+09   1. 454    3120. 000  180. 000      0. 000 4392. 000 4392. 000      18
OVERLAPPING
TIME
DIFFERENCES,
  ABSOLUTE    0. 18964E+08   0. 169      15. 000  106. 758
  % OF MEAS                 11. 592                59. 310
```

Figure X-14. Predicted groundwater flow hydrograph and total measured flow hydrograph for Cypress Creek calibration.



```
HYDROGRAPH STATISTICS FOR LOCATION  23
                        VOLUME                       PEAK FLOW                        DURATION                  NO.
              CUBIC FEET     INCHES          TIME,HR   FLOW,CFS         START,HR   END,HR   LENGTH,HR         POINTS
PREDICTED.    0.17127E+09     1.522          3059.000  128.228            0.000 4430.000 4430.000              194
TOTAL TIME
MEASURED.     0.16359E+09     1.454          3120.000  180.000            0.000 4392.000 4392.000              184
TOTAL TIME
PREDICTED.    0.17042E+09     1.514          3059.000  128.228            0.000 4393.000 4393.000              192
OVERLAPPING
TIME
MEASURED.     0.16359E+09     1.454          3120.000  180.000            0.000 4392.000 4392.000              184
OVERLAPPING
TIME
DIFFERENCES.
  ABSOLUTE  -0.68276E+07     -0.061            61.000   51.772
  % OF MEAS                  -4.174                     28.762
```

Figure X-15. Total predicted flow hydrograph and total measured flow for Cypress Creek calibration.

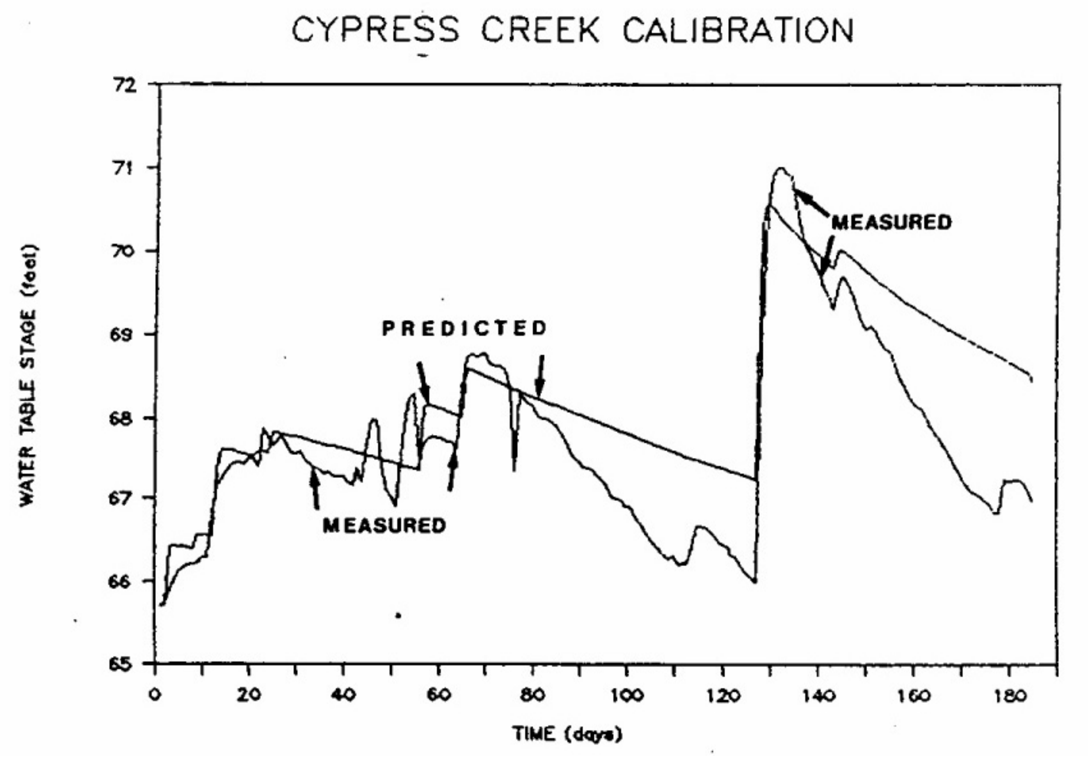


Figure X-16. Predicted and measured stages for Cypress Creek calibration.

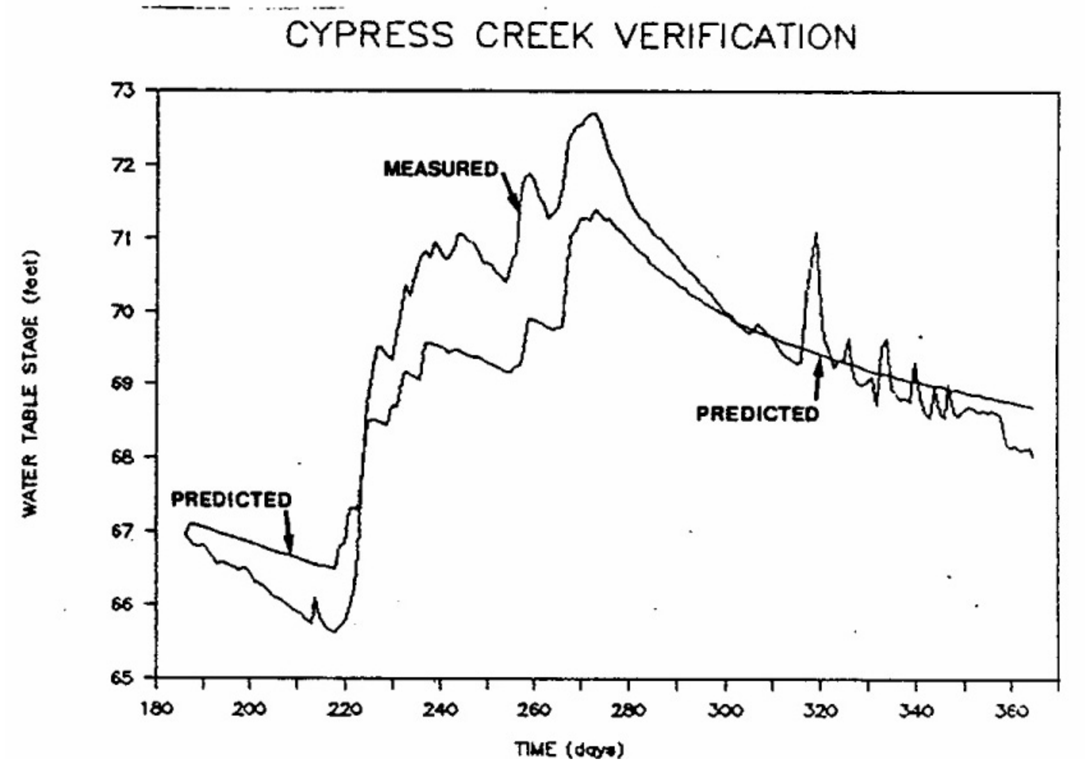
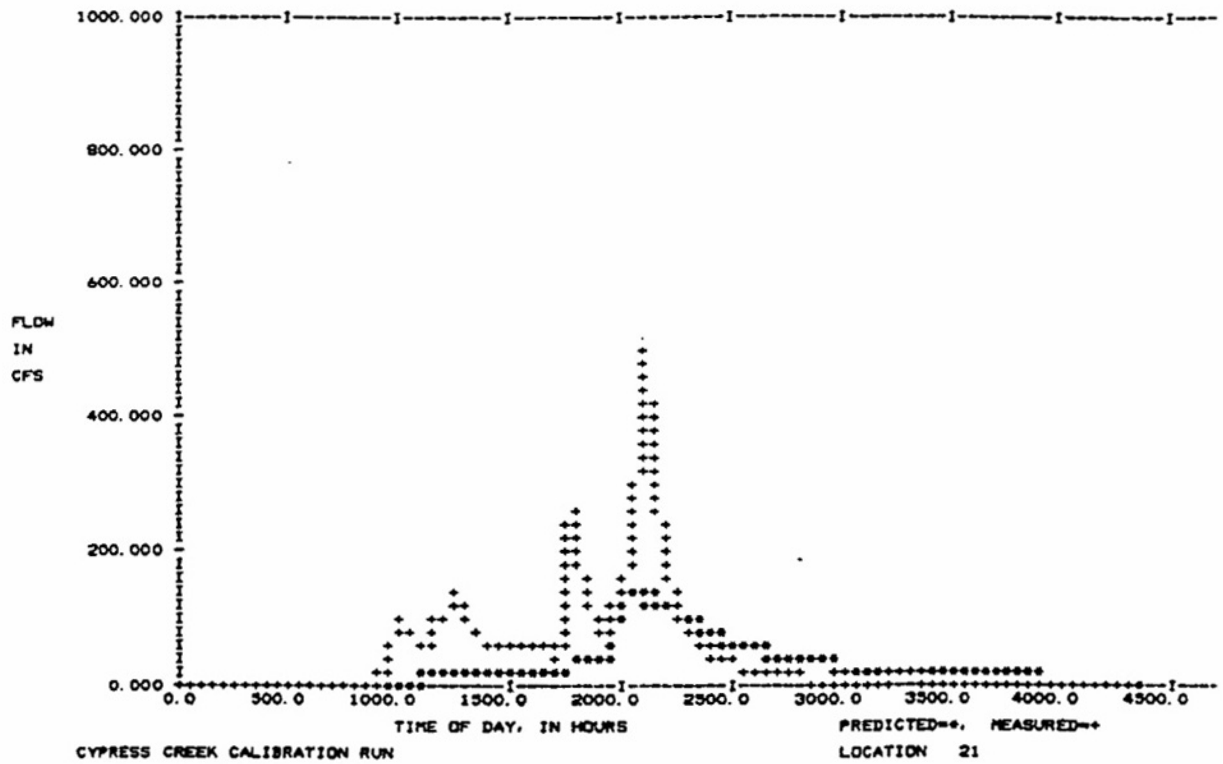
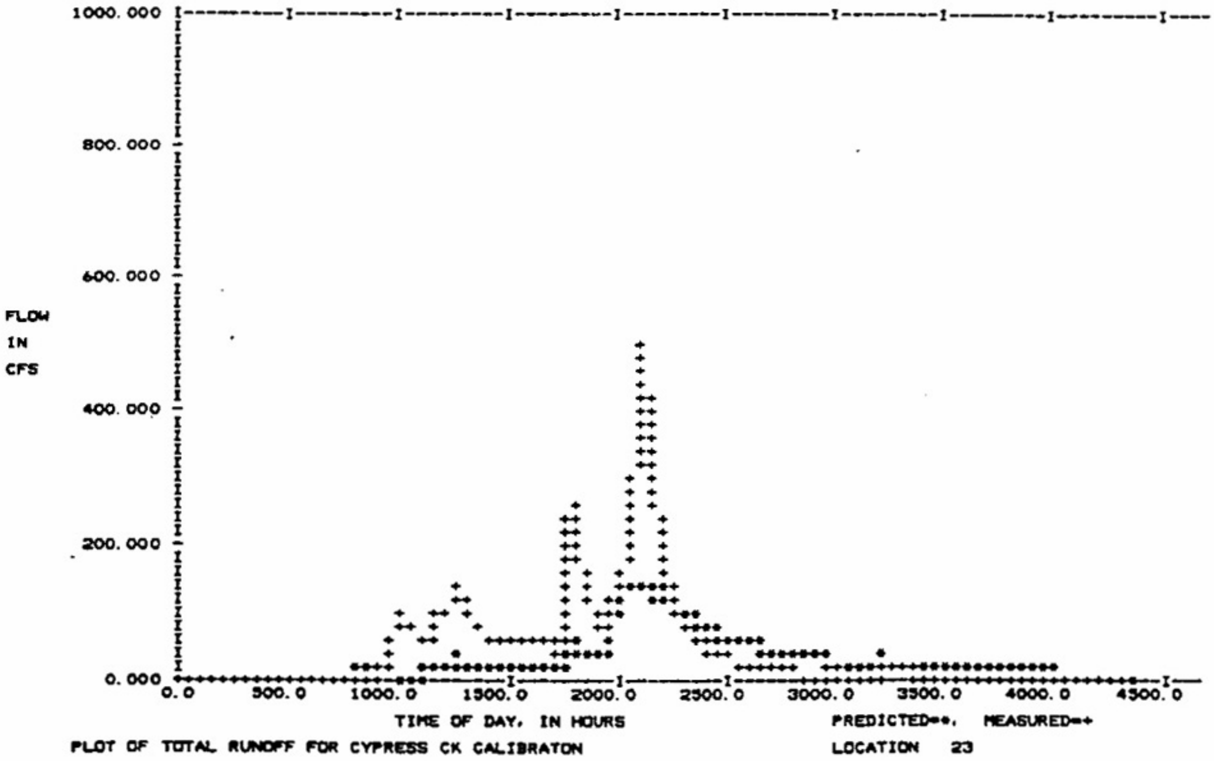


Figure X-17. Predicted and measured stages for Cypress Creek calibration.



```
HYDROGRAPH STATISTICS FOR LOCATION  21
                    VOLUME                 PEAK FLOW                  DURATION                   NO.
               CUBIC FEET   INCHES      TIME,HR  FLOW,CFS      START,HR  END,HR   LENGTH,HR     POINTS
PREDICTED,     0.42533E+09   3.780      2112.000  144.583        0.000 4350.000 4350.000          199
TOTAL TIME
MEASURED,      0.71232E+09   6.330   .  2112.000  500.000        0.000 4320.000 4320.000          181
TOTAL TIME
PREDICTED,     0.42426E+09   3.770      2112.000  144.583        0.000 4312.000 4312.000          197
OVERLAPPING
TIME
MEASURED,      0.71232E+09   6.330      2112.000  500.000        0.000 4320.000 4320.000          181
OVERLAPPING
TIME
DIFFERENCES,
  ABSOLUTE     0.28806E+09   2.560         0.000  355.417
  % OF MEAS                 40.440                 71.083
```

Figure X-18. Predicted groundwater flow hydrograph and total measured flow hydrograph for Cypress Creek verification.



```
HYDROGRAPH STATISTICS FOR LOCATION  23
                       VOLUME                PEAK FLOW                    DURATION                   NO.
             CUBIC FEET      INCHES      TIME,HR   FLOW,CFS       START,HR  END,HR   LENGTH,HR      POINTS
PREDICTED,   0.44397E+09      3.945      2112.000  149.908         0.000 4350.000 4350.000          199
TOTAL TIME
MEASURED,    0.71232E+09      6.330 .    2112.000  500.000         0.000 4320.000 4320.000          181
TOTAL TIME
PREDICTED,   0.44289E+09      3.936      2112.000  149.908         0.000 4312.000 4312.000          197
OVERLAPPING
TIME
MEASURED,    0.71232E+09      6.330      2112.000  500.000         0.000 4320.000 4320.000          181
OVERLAPPING
TIME
DIFFERENCES,
  ABSOLUTE   0.26943E+09      2.394         0.000  350.092
  % OF MEAS                  37.624                 70.018
```

Figure X-19. Total predicted flow hydrograph and total measured flow for Cypress Creek verification.

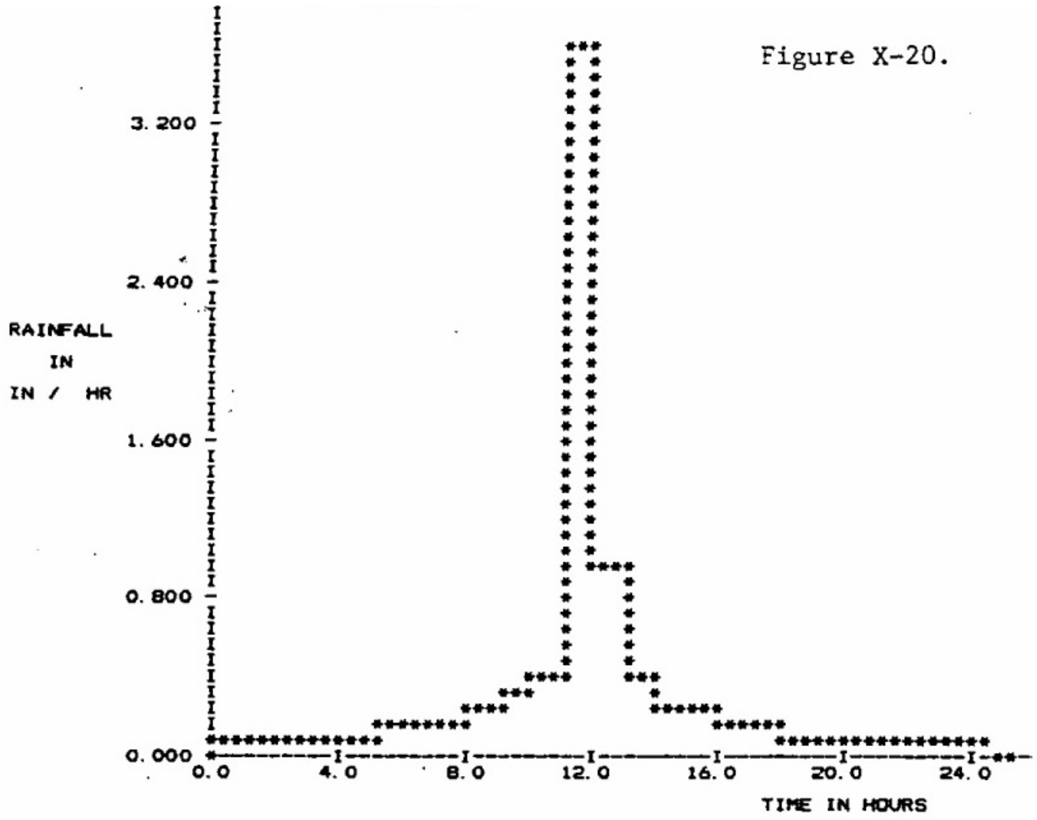


Figure X-20. Hydrograph for hypothetical subcatchment (10-yr SCS Type II design storm for Tallahassee, Florida).

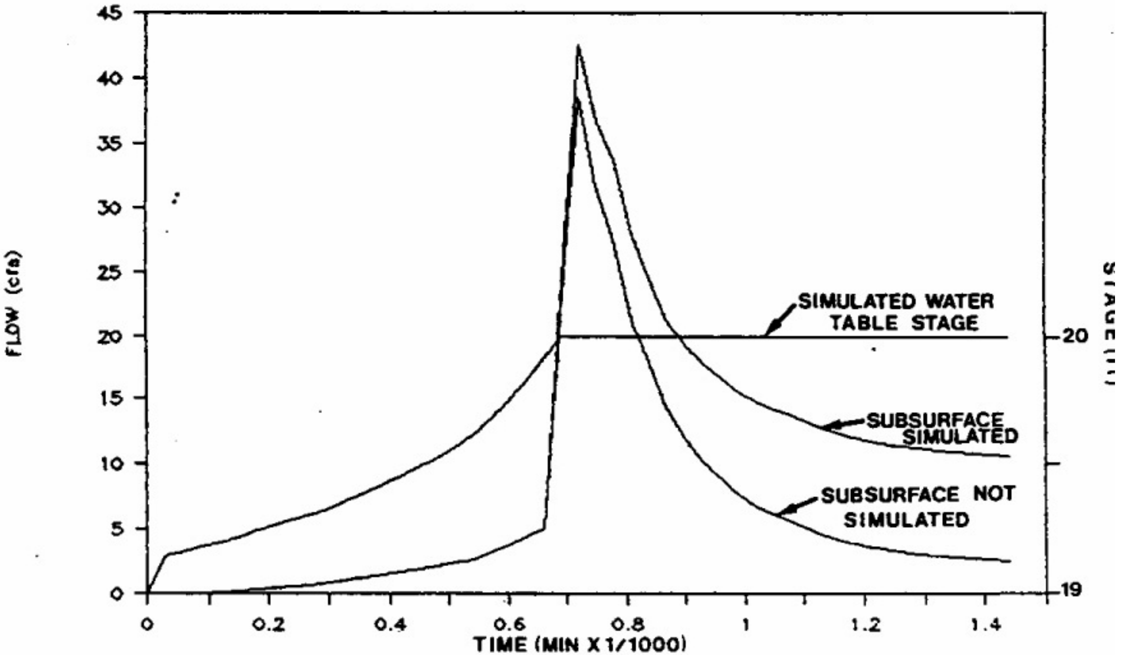


Figure X-21. Hydrographs of surface flow and simulated water table stage from hypothetical subcatchment. The hydrographs are identical until the water table reaches the surface (20 ft).

Table X-2. Fate of Runoff With and Without High Water Table Simulation

	Inches Over Total Basin	
Water Budget Component	With Water Table Simulation	Without Water Table Simulation
Precipitation	8.399	8.399
Infiltration	6.637	1.731
Evaporation	0.103	0.104
Channel flow at inlet	1.495	2.407
Water remaining in channel	0.015	0.038
Water remaining on surface	0.150	4.124
Continuity error	0.001	0.005

Streamflow and Lake Level Changes (RS73B)

Hydrologic/Hydraulic Modeling Results for the PolyMet NorthMet Mine Site

PolyMet Mining, Inc.

September 2008

Streamflow and Lake Level Changes (RS73B)

Hydrologic/Hydraulic Modeling Results for the PolyMet NorthMet Mine Site

PolyMet Mining, Inc.

September 2008

BARR

4700 West 77th Street
Minneapolis, MN 55435-4803
Phone: (952) 832-2600
Fax: (952) 832-2601

RS73B Streamflow and Lake Level Changes: Hydrologic/Hydraulic Modeling Results for the PolyMet NorthMet Mine Site

September 12, 2008

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1.0 Introduction

PolyMet Mining, Inc. (PolyMet) is in the process of environmental review for its NorthMet deposit, near Babbitt in northern Minnesota. As part of this evaluation, Barr Engineering Co. (Barr) has been retained by PolyMet to complete a series of support documents required for the Project Description of the proposed project.

This report is the second part of a series of two reports. The first, entitled *RS73A – Streamflow and Lake Level Changes: Model Calibration Report for the PolyMet NorthMet Mine Site*, was issued as Draft 01 on November 20, 2006. Draft 02 – RS73A was issued on August 31, 2007. Draft 03 – RS73A has been issued in conjunction with this report. The hydrologic/hydraulic model that was developed, calibrated and validated in RS73A was used to evaluate the impacts of the Mine Site on the Partridge River flows and Colby Lake-Whitewater Reservoir water levels (see Figure 1). This document, RS73B, presents the results of those evaluations for the proposed 20-year period of mining operations. The term Mine Site refers primarily to the direct area of influence of open pits, ore stockpiles, mine waste rock and overburden stockpiles, access roads, and other related facilities and civil works (see Figure 2); it does not include the Process Plant and Tailings Basin, which are located in a different watershed (i.e., the Embarrass River watershed).

The hydrologic/hydraulic model was used to estimate relative changes with respect to base conditions (i.e., without mining project) in characteristic flow parameters at several locations along the Partridge River. These changes were evaluated for the proposed project for Mine Years 1, 5, 10, 15 and 20. In addition, one hypothetical high-impact scenario (called “Mine Facilities Off”) reflecting larger than planned impacted areas was evaluated to account for potential uncertainties in the Mine Site development (e.g., conditions that limit the timing of reclaiming of stockpiles, or stockpile footprint change due to unexpected foundation conditions). The analyses of all these scenarios corresponding to conditions during mining operations are presented in this report; two other cases dealing with conditions during and after mine closure are presented in the RS52 report – Closure Plan.

The results of the hydrologic/hydraulic model together with the projected range of make-up water demand for the NorthMet Process Plant were used to conduct a water balance assessment of the Colby Lake-Whitewater Reservoir hydrologic system. A comparison of the water level fluctuations in these two water bodies with and without the NorthMet Project is presented in this report.

1.1 Study Area

Three main features were evaluated in this study: the Mine Site, the Partridge River watershed upstream of Colby Lake, and the Colby Lake-Whitewater Reservoir hydrologic system.

- The Mine Site covers about 4.7 square miles. It is located at the headwaters of the Partridge River watershed. The Mine Site is larger than the actual area to be occupied by the mine facilities (see Figure 2), which ranges from approximately 1.1 square miles at the end of Mine Year 1 to approximately 2.4 square miles by the end of Mine Year 20 when mining operations are expected to cease.

- The Partridge River is a tributary of the St. Louis River, which is part of the Lake Superior southwestern drainage basin. The study area for quantitative hydrologic and hydraulic assessment of the potential impacts of the NorthMet project on the Partridge River flows was defined in the final Scoping Decision Document (SDD) as the catchment area of U.S. Geological Survey (USGS) gaging station #04015475 – Partridge River above Colby Lake at Hoyt Lakes, Minnesota. This catchment area is 103.4 square miles, and it includes the Mine Site described in the previous paragraph. The Partridge River wraps around the Mine Site (see Figure 1). It begins at the One Hundred Mile Swamp on the northwest end of the Mine Site, flows eastward and then southward, to finally turn southwest along the south side of the Mine Site where it joins the south branch of the Partridge River. The catchment areas of the north and south branches of the Partridge River are 26.1 and 28.3 square miles, respectively. Downstream of this confluence, the Partridge River continues flowing southwest for approximately 13 miles to Colby Lake.

- The Colby Lake-Whitewater Reservoir hydrologic system is located southwest of the Mine Site, immediately downstream of the confluence of the Partridge River (whose watershed was described in the previous paragraph) and Wyman Creek. The catchment area of the hydrologic system at the outlet of Colby Lake is 127.8 square miles, and it includes two unnamed tributaries entering Colby Lake on the north as well as the watershed directly draining to Whitewater Reservoir (see Figure 1). The Partridge River flows through Colby Lake. Colby Lake is connected to Whitewater Reservoir on the south through the Diversion Works, which was constructed in 1955 to augment the storage capacity to supply make-up water for taconite mining operations that were active until 2000. Currently, Minnesota Power uses water from Colby Lake for once-through cooling water in its coal powered electric power generating plant at the Laskin Energy Center.

1.2 Preceding Reports

The scenarios evaluated for this report were based on the following preceding reports:

- The Tailings Basin Water Balance Report (RS13) determined the likely range of make-up water demand for the NorthMet Process Plant.

- The Mine Plan Report (RS18) provided the locations of the pits, stockpiles, and other mine-related features at different stages of the Mine Site development.

- The Mine Site Water Balance Report (RS21) summarized the results of the RS22, RS24, and RS25 evaluations.

- The Mine Waste Water Management Systems Report (RS22) defined the areas that will not be contributing runoff to the Partridge River depending of the stage of Mine Site development. The runoff from these areas are treated as process water; that is, precipitation runoff and groundwater that has contacted disturbed surfaces such as open pits and unreclaimed stockpiles and may not meet water discharge limits. Process water will be collected, treated (if required) and diverted to a different watershed for use in the Process Plant.

- The Mine Surface Water Runoff Systems Report (RS24) provided estimates of the runoff contribution from natural undisturbed areas and reclaimed stockpiles depending of the stage of Mine Site development. The runoff from these areas is treated as stormwater; that is, precipitation runoff that has not contacted disturbed surfaces and will be routed to the Partridge River following existing drainage patterns as much as possible.

- The Mine Diking/Ditching Effectiveness Study Report (RS25) defined the perimeter diking system around the exterior of the Mine Site and the diking along the rim of the pits, which changed drainage patterns at different stages of the Mine Site development.

- The Partridge River Level 1 Rosgen Geomorphic Survey (RS26) identified the reaches that were potentially sensitive to changes in stream flows.

- The Streamflow and Lake Level Changes: Model Calibration Report (RS73A) presented the methodology and results of calibrating and validating the hydrologic/hydraulic model of the Partridge River watershed, which was used in RS73B to assess the impacts of the Mine Site development on the water quantity in downstream rivers and water bodies (i.e., the Partridge River, Colby Lake and Whitewater Reservoir).

This report uses data from the RS22, RS24, RS25 and RS73A evaluations. Figure 3 illustrates the interaction between these studies. Readers interested in reviewing all of the Mine Site water management reports may find the following sequence most beneficial for their review: RS73A, RS25, RS22, RS24, RS21, and RS73B. The closure of the Mine Site water management systems is described in RS52.

1.3 Objectives

The objectives of this study were based on the approach to define cumulative effects described in the final Scoping Decision Document and the Work Plan (see Appendix A of the RS73 combined report) that further defined the scope based on discussions with the agencies. The Work Plan has been modified from the January 23, 2006 version that was provided to the agencies to incorporate final comments.

The objectives of the study have changed in some respects from those listed in the Work Plan (these revisions were discussed with John Adams and Mike Liljegren from the Minnesota Department of Natural Resources [MDNR]):

- The Work Plan was developed assuming that the mining operations would increase discharges from the Mine Site to the Partridge River, which would require detailed information on the magnitude and timing of the increased flows to define the potential impacts on the receiving watercourse(s). Detailed hydrologic/hydraulic modeling of the Mine Site was proposed. The NorthMet Project is now proposing a reuse/recycle strategy, with no discharge to surface waters of the State, though stormwater (runoff from undisturbed or reclaimed portions of the Mine Site) will be routed to the Partridge River following existing drainage patterns. Flows in the Partridge River are expected to stay the same or decrease because a portion of the original Mine Site runoff (i.e., the process water) will be reclaimed for use in the Process Plant. Therefore, detailed hydrologic analysis of the Mine Site water management systems with the hydrologic/hydraulic model developed for the Partridge River watershed is no longer warranted. However, the overall effects that diverting Mine Site process water would have on the Partridge River flows and Colby Lake-Whitewater Reservoir water levels were addressed in this report using the referred hydrologic/hydraulic model.

- The Work Plan was developed assuming that cumulative impacts to the physical character of streams and lakes would occur from increases or decreases in flow or changes in the pattern of flow due to various watershed impacts, such as point discharges (e.g., mine dewatering discharges) and changes in watershed runoff caused by alterations in the percentage distribution of land use (mining, timber harvest, residential development, road construction, etc.). In other words, the hydrologic/hydraulic model of the Partridge River watershed was intended to evaluate not only impacts related to the Mine Site development, but also impacts from other activities in the watershed. In addition to PolyMet's proposed project, two main potential changes in the hydrology of the Partridge River watershed were anticipated. The first envisaged change was the possibility of Northshore Mining Company (Northshore) reinitiating full-scale mining of inactive portions of the Peter Mitchell Pit located north of the Mine Site (see Figure 1). Portions of the Peter Mitchell Pit have been allowed to fill with water, but pumping from active mining areas and discharges to the Partridge River currently continue. Further inquiries on this regard indicated that the feasibility of Northshore proceeding with dewatering is small. The second envisaged change was the increase in the rate of timber harvesting in the Partridge River watershed. Information about forest stand information available from both the MDNR and the U.S. Forest Service (USFS) Superior National Forest (SNF) indicate that only 5.6 percent of the watershed has been harvested since 1980, and the corresponding annual rate of timber harvesting is not anticipated to increase in the near future, hence the related impacts on the Partridge River flows are not expected to be significant (Verry, 2000). Therefore, detailed hydrologic analysis of the effects of these potential changes with the hydrologic/hydraulic model developed for the Partridge River watershed is no longer warranted. However, the referred hydrologic/hydraulic model has the capability to evaluate hydrologic impacts of activities that are not related to the Mine Site development.

- The Work Plan indicates that the model may be extended beyond the gaging station record using meteorological data to analyze both wet and dry climatic conditions by creating a synthetic, local streamflow record. However, analysis of the gage data indicate that the 10-year period of flow data at the Partridge River includes a very wet and a very dry year within the period of record (see Section 2.2.1), hence the extended record may not provide data that significantly enhances the overall results of the analysis. Therefore, the model was not extended and the results were based on the existing period of record. Statistical analyses were conducted to provide key parameters that define the flow (Section 3.3) and stream morphology (Section 3.4) impacts.

RS73A presented the development, calibration and validation of a hydrologic/hydraulic model of the existing Partridge River watershed. This RS73B report presents the results of using that hydrologic/hydraulic model to evaluate expected relative changes on the average, minimum and maximum flows along the Partridge River, to define the stream morphology impacts to the Partridge River, and to determine the increase in water level fluctuations at Colby Lake and Whitewater Reservoir as a result of the proposed NorthMet Project. The model is not intended to predict instantaneous flow values, but to provide estimates of overall trends in the flow pattern as the mining project is implemented.

The objectives of this RS73B report are to:

- Present a summary of flow statistics under Existing Conditions (that is, without NorthMet Project) at several locations along the Partridge River.
- Estimate relative changes in flow patterns during various stages of the Mine Site development at the same locations along the Partridge River.
- Define stream morphology impacts to the Partridge River.
- Estimate relative changes in water level fluctuations in Colby Lake and Whitewater Reservoir as a result of the expected reduction in Partridge River flows and the make-up water demand for the Process Plant.

1.4 Report Outline

The remainder of this report is organized as follows:

- Section 2.0 provides the methodology and results from the Partridge River model. This includes a brief summary of the hydrologic/hydraulic model that was developed, calibrated and validated for the Partridge River watershed as part of RS73A. Section 2.0 also presents the existing flow estimates at different locations along the Partridge River and a preliminary floodplain map of the existing Partridge River flood levels in the vicinity of the Mine Site. Furthermore, this section describes the methodology followed to set-up the hydrologic/hydraulic model to analyze various stages of the Mine Site development, including one hypothetical high-impact scenario.

- In Section 3.0, the results of the modeling are reported in terms of the fractional changes in average, minimum and maximum flows at several locations along the Partridge River and conditions with and without NorthMet Project are compared. Section 3.0 also describes the likelihood of major changes in stream morphology as defined by the Rosgen classification method used in RS26.

- Section 4.0 describes the water balance method used to assess the relative impacts to the water level fluctuations in Colby Lake and Whitewater Reservoir when comparing conditions with and without NorthMet Project and presents the conclusions of that assessment.

2.0 Partridge River Model

2.1 Calibrated Model

The hydrologic/hydraulic model developed for the Partridge River watershed was built in XP-SWMM, a physically-based model based on the U.S. Environmental Protection Agency (USEPA)'s Storm Water Management Model (SWMM). This unsteady flow model allows simultaneous hydrologic and hydraulic modeling across a study watershed. Its hydrology module can simulate both single storm events and continuous long-term periods, and it accounts for spatial distribution of rainfall, snowfall and snowmelt, infiltration, groundwater and runoff volumes and flows. Its hydraulic module uses full dynamic flow routing, including the analysis of ditch and natural channel networks as well as the evaluation of water level fluctuations in ponding areas. In addition, XP-SWMM has the capability to define the interaction between surface water and groundwater, which can be particularly important for large wetland areas and other locations where groundwater has a significant impact on the surface water flows. Figure 4 presents a simplified schematic of the connection between the different components included in XP-SWMM.

RS73A shows that the calibrated model for the study area defined in the SDD does a good job in matching recorded baseflows in the Partridge River, which is the period when downstream waters are expected to experience the greatest impacts due to the reduction in the contributing drainage area from the Mine Site, in particular due to dewatering of the mine pits (see Appendix B of RS22). The calibrated model also performs well in capturing the timing and magnitude of peak flows associated with the spring snowmelt event and subsequent summer floods. While some deficiencies do exist in the ability of the model to reproduce instantaneous recorded flows (especially during very large storm events), this hydrologic/hydraulic model was considered adequate to evaluate relative changes on the average, minimum and maximum flows at several locations in the Partridge River during different stages of the proposed mining operation, but not adequate to predict instantaneous flow values. The model is intended to provide estimates of changes in the general trend of the flow pattern in the Partridge River as the mining operation is underway. Details about the model development, calibration and validation were presented in RS73A.

2.2 Partridge River Flows

2.2.1 Existing Conditions

Daily flow data is available at USGS gaging station #04015475 – Partridge River above Colby Lake at Hoyt Lakes for the period September 19, 1978 through November 2, 1988. Additional daily flow data within the study watershed is available at USGS gaging station #04015455 – South Branch Partridge River near Babbitt for the period June 1, 1977 through November 5, 1980. Both flow records are presented in Figure 5.

The correlation coefficient between the two Partridge River flow datasets for the period of coincident record is relatively high ($r^2 = 0.96$), one indication that the study area might be considered hydrologically homogeneous. If the watershed were hydrologically homogeneous, flows at different locations along the Partridge River could be estimated by simply multiplying the ratio of the catchment area at the point of interest and at the USGS gaging station #04015475 (Partridge River above Colby Lake) by the recorded flows at this USGS gaging station. However, the inference of hydrologic homogeneity is difficult to substantiate. First, the period of coincident record is only two years. Second, as discussed in RS73A, the flow record at the USGS gaging station #04015475 may have been impacted by mine discharges on the north branch of the Partridge River, whereas this has not been the case for the USGS gaging station #04015455 that reports flows on the south branch of the Partridge River. Therefore, flows at different locations along the Partridge River need to be predicted with a hydrologic/hydraulic model.

The 1978-1988 period of flow record at the USGS gaging station #04015475 (Partridge River above Colby Lake) is representative of the expected variability in climatic and hydrologic conditions in the study watershed.

- The MDNR follows the definition given by the Climate Prediction Center of the National Weather Service (NWS), which considers a climate normal as that given by 30 years of recent data. The current definition corresponds to the period October 1, 1971 through September 30, 2001. The mean annual precipitation for the Mine Site was 29.1 inches from 1978-1988, whereas the corresponding value was 29.2 inches in 1971-2001. Furthermore, annual precipitation values for the Mine Site during the period 1978-1988 include the second largest (the largest is 41.8 inches; the second largest is 35.9 inches) and the fifth smallest (the smallest is 20.3 inches; the fifth smallest is 25.7 inches) annual precipitation values during the period of climate normal.

- Baker et al. (1979) suggested the ratio of average runoff to precipitation has a mean value of 0.40-0.45 for this region of Minnesota. The average gaged runoff to precipitation ratio in the Partridge River watershed was 0.43 for the period 1978-1988, with a maximum value of 0.60 in water year 1978-1979 and a minimum value of 0.22 in water year 1979-1980. Moreover, the peak flow during water year 1978-1979 was more than twice the annual maxima recorded during the remaining gage period.

A combination of the flow record in USGS gaging station #04015475 (Partridge River above Colby Lake) and the output of the XP-SWMM calibrated model for the study area defined in the SDD was used to estimate flows without the NorthMet Project at seven locations along the Partridge River (see Figure 6):

- Surface water monitoring station SW-001. This location on the north branch of the Partridge River is upstream of all Mine Site facilities (but downstream of the Peter Mitchell Pit discharge), and its catchment area is 6.2 square miles.

- Surface water monitoring station SW-002. This location on the north branch of the Partridge River is northeast of the Mine Site, and its catchment area is 13.3 square miles.

- Surface water monitoring station SW-003. This location on the north branch of the Partridge River is east of the Mine Site, and its catchment area is 15.2 square miles.

- Surface water monitoring station SW-004. This location on the north branch of the Partridge River is immediately upstream of the confluence with the south branch, downstream of 64 percent of the proposed Mine Site facilities by the end of Year 20, and its catchment area is 23.0 square miles.

- Surface water monitoring station SW-004a. This location on the Partridge River is immediately downstream of the confluence of the north and south branches, downstream of 99 percent of the proposed Mine Site facilities by the end of Year 20, and its catchment area is 54.4 square miles.

- Surface water monitoring station SW-005. This location on the Partridge River is at the railway crossing, downstream of 100 percent of the proposed Mine Site facilities by the end of Year 20, and its catchment area is 98.7 square miles. The Mine Site (4.7 square miles) represents less than 5 percent of this watershed.

- USGS gaging station #04015475. This location on the Partridge River is upstream of Colby Lake, and its catchment area is 103.4 square miles.

Locations SW-001, SW-002, SW-003, SW-004 and SW-005 were selected because background water quality data were available (see RS63, RS76 and RS74). Location SW-004a was selected because it is located downstream of 99 percent of the proposed Mine Site facilities. The USGS gage location was selected because flow data were available.

Percent reductions in catchment area at each of these seven locations vary depending on the development stage of the Mine Site (see Section 2.3). Catchment areas and percent reductions (with respect to existing conditions) for each scenario evaluated, including “Mine Facilities Off”, are presented in Table 1.

Using 1978-1988 as the period of analysis, basic statistics (mean, minimum and maximum annual flows) were computed for the flow record at the USGS gaging station #04015475. It is important to note that the average flow for the period of analysis October 1, 1978 to September 30, 1988 (ten complete water years) is 88 cubic feet per second. The average flow for the entire period of record from September 19, 1978 to November 2, 1988 is 87 cubic feet per second.

Ratios of the recorded versus calibrated modeled values (based on the XP-SWMM output at the Partridge River USGS gage #04015475[1]) were then obtained for each statistic and every water year in the period of analysis. These ratios were applied to the appropriate statistics computed from the XP-SWMM output for each of the other six reporting locations in the Partridge River previously listed as surface water monitoring stations, so historic statistics without the NorthMet project could be estimated at these locations. Results of these computations are presented in Table 2.

2.2.2 Flood Levels

The Mine Site lies entirely within the City of Babbitt, for which no Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) has been printed. The 1992 St. Louis County FIRM contains a special note on the City of Babbitt that says, “Panel not printed – Area in Zone X.” Two definitions are given for Zone X on the St. Louis County FIRM: 1) Areas of 500-year flood; areas of 100-year flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from the 100-year flood; and 2) Areas determined to be outside the 500-year flood plain. In either case, the area within and adjacent to the Mine Site area is not FEMA jurisdictional and no Letter of Map Change (LOMC) will be required for any development within the Mine Site.

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[1] Definition of “Existing Conditions” for period used in model calibration (1978-1988) differs from definition of “Existing Conditions” without NorthMet Project. See discussion in Section 2.3

The calibrated XP-SWMM model of the Partridge River watershed was used to estimate flood levels in the vicinity of the Mine Site without NorthMet Project. It is understood that these flood levels are approximate, with an error in the order of 2 feet given the resolution of the available digital elevation model (DEM) developed for the study watershed (see RS73A), the degree of information on the characteristics of culverts, bridges and other hydraulic structures, and the uncertainty in the determination of the input data of the hydrologic/hydraulic model. However, the computed flood levels can be used to define relative elevations and estimate probable impacts based on these relative elevations.

Flood levels were determined by setting the initial depth of the water table at 0.01 inches below the ground surface in the groundwater module of XP-SWMM, hence effectively creating a 100 percent impervious surface. While this assumption may be appropriate for the wetlands-dominated headwaters of the Partridge River, it is a conservative assumption further downstream where the watersheds contain a significant proportion of uplands; therefore the results were restricted to the north branch of the Partridge River. In order to determine the event that produces the highest peak flow, both the 100-year, 24-hr rainfall event and the 100-year, 10-day snowmelt event were simulated. With the groundwater assumption previously described, the watershed responds very quickly to precipitation events, and the 24-hr rainfall event produces higher flows and flood levels than the 10-day snowmelt event. The 100-year, 24-hr rainfall of 5.2 inches and the 500-year, 24-hr rainfall of 6.2 inches were then simulated, in both cases considering the hyetograph proposed by Huff and Angel (1992). Resulting preliminary flood levels for these two events are shown on Figure 7.

Under existing conditions, there is some risk of flooding along the northern end and around the southeast corner of the Mine Site. Moreover, a north-south overflow channel could develop through the Mine Site during large flood events. As indicated in RS25, perimeter dikes will be constructed to minimize this risk. Overall, it is expected the impact of the dikes on flood levels to be negligible because the dikes are being constructed in the fringes of the floodplain. Construction of the dikes will result in a small decrease in wetland storage in the One Hundred Mile Swamp north of the Mine Site. The greatest cumulative impact occurs near the eastern end of the Category 1/2 stockpile, with a decrease of 1 percent in storage area from 2,920 acres to 2,881 acres. The tributary drainage area to this point is 7,070 acres and is 41 percent wetland.

The MDNR oversees and administers the National Flood Insurance Program. For areas within the 100-year floodplain, rules from the Minnesota statutes (Chapter 6120) require that "Increases in upstream flood stages which would result from construction of dikes … shall not increase the stage of the regional flood in excess of 0.5 feet in any one reach or for the cumulative effect of several reaches of a watercourse.” The impact on the 100-year flood level will be much less than this requirement.

2.3 Projected Operations

The calibrated model for the Partridge River watershed was modified to develop a Partridge River model for each of the following development stages of the Mine Site and hypothetical high-impact scenario:

- Current Existing Conditions; that is, without NorthMet but including discharges from Peter Mitchell Pit.
- Year 1; that is, by the end of the first year of mining operations.
- Year 5; that is, by the end of the fifth year of mining operations.
- Year 10; that is, by the end of the tenth year of mining operations.
- Year 15; that is, by the end of the fifteenth year of mining operations.
- Year 20; that is, by the end of the twentieth year of mining operations.
- Mine Facilities Off; that is, a hypothetical high-impact scenario in which all runoff from the footprint of the mine facilities, including reclaimed stockpiles, is collected and diverted to a different watershed.

The model for Current Existing Conditions represents a scenario in which all areas within the Mine Site drain to the Partridge River (see Figure 8), but it differs from the calibrated model with respect to the drainage area. The calibrated model corresponds to a period (1978-1988) when mining operations by Northshore in the Peter Mitchell Pit caused a reduction in the effective drainage area of the Partridge River (see Figure 1); dewatering of the Peter Mitchell Pit was not necessarily discharged to the Partridge River (see RS73A). Under Current Existing Conditions, the Peter Mitchell Pit is inundated and discharges to the Partridge River. The additional drainage area that is included under Current Existing Conditions is 5.2 square miles, for a revised total area of 103.4 square miles to the Partridge River watershed upstream of Colby Lake.

The models for Years 1 to 20 of mining operations represent different development stages of the Mine Site, with footprints of the mine facilities varying between 1.1 square miles in Year 1 and 2.4 square miles in Year 20 (see RS18); the entire Mine Site has a total of approximately 4.7 square miles. During the proposed 20-year period of mining operations, stormwater produced within the Mine Site will be routed to the Partridge River (see RS24). Process water, however, will be collected, treated (if required) and diverted to a different watershed for use in the Process Plant (see RS22). Therefore, the effective drainage area of the Partridge River will decrease as a result of the NorthMet Project development, which has been quantified following the procedures described in Sections 2.4.1 and 2.4.2.

2.4 Predictive Models

2.4.1 Watershed Delineation

The calibrated model for the Partridge River watershed upstream of Colby Lake consists of 75 sub-watersheds, which served as a reference to determine the boundaries of the effective runoff contributing area in each sub-watershed for each stage of Mine Site development (Years 1 to 20) and the hypothetical high-impact scenario (see Section 2.3). More specifically, existing sub-watersheds partially or completely within the Mine Site were modified from the configuration used in the calibrated model to portray the anticipated changes in the drainage patterns resulting from the Mine Site development. No new sub-watersheds were added to the model, although some existing sub-watersheds were completely eliminated (turned off) if their areas are projected to be entirely occupied by mine facilities producing process water rather than stormwater.

The delineation of sub-watersheds partially or completely within the Mine Site took into account the fact that construction and expansion of the mine pits and stockpiles will create watershed divides that change over the lifespan of the mine. Watershed delineations within the Mine Site also reflect the proposed network of interior and perimeter ditches and dikes that alter the existing drainage patterns and that are intended to prevent stormwater from entering the pits or from entering or leaving the Mine Site uncontrolled (see RS24 and RS25). Ditches and dikes, as well as haul roads, will be constructed sequentially and will have an effect on the physical configuration of the sub-watersheds. Watershed divides within the Mine Site were thus established based on the proposed mine facility layouts in Years 1, 5, 10, 15 and 20. Figures 9 through 13 show the sub-watershed delineations for the years analyzed.

For the Mine Facilities Off scenario, all areas within the Year 20 sub-watersheds that will be occupied by mine facilities were eliminated (turned off) from the model (see Figure 14); in other words, all surface runoff and groundwater recharge within the footprint of the Year 20 mine facilities was assumed to be diverted to a location outside the Partridge River watershed. This scenario does not correspond to PolyMet's proposed mine development plans, but it represents a hypothetical high-impact scenario that is included to account for potential uncertainties in the Mine Site development (e.g., conditions that limit the timing of reclaiming of stockpiles, or stockpile footprint change due to unexpected foundation conditions).

2.4.2 Stockpile Runoff and Infiltration

Inactive sections of Category 1/2, Category 3, Category 3 Lean Ore, and Category 4 stockpiles will be reclaimed progressively. Runoff from these reclaimed vegetated surfaces, together with runoff from undisturbed (natural) areas within the Mine Site, will be routed to the Partridge River after removal of suspended sediment. A range of runoff volumes from the reclaimed stockpiles was estimated based on measurements from stockpiles at existing mines in northeastern Minnesota and Saskatchewan (see RS22). The low estimates presented in Table 3 were used to provide a conservative estimate of the impacts to the Partridge River flows. Because any impacts would be expected to be produced from reductions in flow, conservative is defined as the estimate of runoff from the top of the reclaimed stockpiles that causes the largest reduction in runoff produced within the Mine Site.

In general, water that runs off the tops and sides of active stockpiles or water that infiltrates the reclaimed stockpiles and reaches the bottom liner system is considered process water; therefore it will be routed to the Wastewater Treatment Facility (WWTF) and pumped to the tailings basin for reuse/recycle in the Process Plant. Put more simply, process water will not be discharged to the Partridge River at any time during mining operation. The groundwater module of XP-SWMM model, however, considers that part of the water infiltrated in natural conditions may recharge the adjacent stream channel. While stockpiles are active, the watershed is simply removed from the drainage area, because all runoff and infiltration is process water. Because infiltration on reclaimed stockpiles is not going to infiltrate into the natural ground, the volume of runoff from reclaimed vegetated surfaces (i.e., stormwater) required a different treatment in XP-SWMM than volume of runoff from undisturbed (natural) areas so that infiltration did not contribute to groundwater. To accomplish this, the area occupied by the reclaimed stockpile was converted into an equivalent smaller, 100 percent impervious area that produces the same volume of runoff as that estimated for the reclaimed stockpiles according to Table 3. This produces the same surface runoff volume without simulating a contribution to groundwater recharge to the stream channel. Although simulating these areas as 100 percent impervious surfaces resulted in increased peak flows and flashier flood hydrographs, the equivalent 100 percent impervious areas were small compared to the total areas of the affected sub-watersheds, so the error introduced was minimal. The effective watershed area for each model developed is presented in Table 4.

2.4.3 Summary of Modified Model Parameters

Model parameters were estimated for each of the seven scenarios described in Section 2.3 (Existing Conditions, Years 1 to 20, and Mine Facilities Off) using the same methods that were outlined in the RS73A report for the calibrated model. Most parameters used in the calibrated model were maintained for these hydrologic/hydraulic simulations. The parameters that were changed are:

- Catchment areas were calculated using ESRI ArcMap GIS software.

- Sub-watersheds slopes were calculated as area-weighted averages based on the composite DEM developed for the study watershed and ESRI ArcMap GIS software.

- Sub-watersheds widths were calculated based on a digitized average flowpath using an ESRI ArcView script.

- Impervious percentages were calculated as area-weighted averages based on the land cover type distribution within each sub-watershed and imperviousness reference values (see RS73A). The GAP Analysis coverage for the Mine Site was modified to reflect proposed conditions with the NorthMet Project.

- Infiltration parameters were computed for each sub-watershed based on upland and lowland soil types (see RS73A) and the revised sub-watershed areas.

3.0 XP-SWMM Results and Streamflow Impacts

3.1 Simulation Period

The hydrologic/hydraulic simulations for each of the seven scenarios described in Section 2.3 were conducted using XP-SWMM and climate data for the period October 1, 1977 through September 30, 1988. The first water year (October 1, 1977 through September 30, 1978) served to initialize the model, whereas the next ten water years included in the simulation (October 1, 1978 through September 30, 1988) correspond to the period for which flow data is available at the USGS gaging station #04015475 (Partridge River above Colby Lake).

The results obtained from XP-SWMM that are presented in this RS73B report consider that surface overburden would be placed in a separate stockpile at the southeast corner of the Mine Site; the footprint of the separate overburden stockpile was 207 acres by Year 20, which is approximately 11 percent of the footprint of PolyMet's mine facilities by Year 20. With the recently updated plans for the Mine Site (see RS18), surface overburden will be hauled from the mine area and placed in a separate portion of the Category 1/2 stockpile. Depending on the stage of Mine Site development, this change results in approximately 1 percent to 9 percent more stormwater being routed from the Mine Site to the Partridge River (see RS24). Therefore, impacts on Partridge River flows presented in this document are slightly overestimated, as the actual reduction in contributing watershed is less than assumed in this document.

3.2 Statistical Analysis

Summary statistics were calculated from the XP-SWMM output for each of the seven reporting locations listed in Section 2.2.1 and the seven model simulations described in Section 2.3, which accounts to a total of 49 output datasets. It is important to mention that, as explained in more detail in Appendix A, the flow results from the modeling with XP-SWMM were corrected to incorporate the MODFLOW model predictions of the effects of mine pit dewatering on groundwater flows, which have been revised since the publication of Draft 02 – RS73B (see Appendix B of RS22 – Draft 03). This modeling with MODFLOW includes estimation of a cone of depression in the water table that extends beyond the surface watersheds that are affected by mining activity and, in some areas, beyond the Mine Site boundaries; in other words and put more simply, the groundwater catchment area affected by the NorthMet Project is greater than the corresponding surface catchment area affected by the NorthMet Project.

The 49 output datasets (including the correction for the effects of mine pits dewatering on groundwater levels and flows) were processed using a script developed in MS Visual Basic for Applications. This script allows computation of statistics for each year in the model simulation period (1978-1988). The summary statistics presented in this report correspond to the parameters recommended in the "Range of Variability Approach" by Richter et al. (1998), and include:

- Mean flow for each calendar month.
- Annual maximum 1-day, 3-day, 7-day, 30-day and 90-day mean flows.
- Annual minimum 1-day, 3-day, 7-day, 30-day and 90-day mean flows.
- Number of zero-flow days.
- 7-day minimum flow divided by mean flow for year.
- Julian date of each annual 1-day maximum flow.
- Julian date of each annual 1-day minimum flow.
- Number of high pulses each year; that is, the number of times per year the mean daily flow increases above the 75th-percentile of all recorded/simulated mean daily flows.
- Number of low pulses each year; that is, the number of times per year the mean daily flow falls below the 25th-percentile of all recorded/simulated mean daily flows.
- Mean duration of high pulses; that is, the number of days per year with mean flows above the 75th-percentile of all recorded/simulated mean daily flows.
- Mean duration of low pulses; that is, the number of days per year with mean flows below the 25th-percentile of all recorded/simulated mean daily flows.
- Means of all positive differences between consecutive daily values.
- Means of all negative differences between consecutive daily values.
- Number of flow reversals.

Richter et al. (1988) indicate that these statistics are the key parameters used to define the degree of hydrologic alteration due to changes in the watershed. These authors also recommend using these statistics to evaluate flow and stream morphology impacts. The discussion of the XP-SWMM results focuses on mean annual flows, mean annual maximum flows and mean annual minimum flows. The complete set of tabulated results is presented in Appendix A.

3.3 Predicted Impacts to the Partridge River

The results for mean annual flows, daily maximum flows and daily minimum flows are presented in Figures 15 through 20, including the projected five stages of the Mine Site development (Years 1 to 20) and the hypothetical high-impact scenario Mine Facilities Off. Because the hydrologic/hydraulic model of the Partridge River watershed was developed to estimate relative changes in the flow patterns, the results in these figures are expressed in terms of the fractional reduction in the Partridge River flows with respect to the Current Existing Conditions scenario. No graphical results are presented for the surface water monitoring station SW-001, as this is located in the Partridge River upstream of all PolyMet's mine facilities (therefore all results show zero impact).

For the five scenarios representing the projected five stages of the Mine Site development, the predicted reductions in mean and maximum flows are less than 10 percent, with the greatest expected impact around Year 15 to Year 20 of mining operations, when the footprint of the mine facilities is near the maximum area covered by the NorthMet Project, and reclamation of the stockpiles is still underway in the case of Year 15 (see RS22) or vegetation planted in the reclaimed surfaces of the stockpiles has not fully matured yet in the case of Year 20 (see RS52). It is interesting to note in Figures 15 through 20 that the larger impact on the Partridge River mean and maximum flows is at the surface water monitoring station SW-004 (see Figure 17), which is located just upstream of the confluence with the south branch and downstream of 64 percent of the projected Mine Site facilities by the end of Year 20. Surface water monitoring station SW-004a is located immediately downstream of the confluence of the Partridge River north and south branches and it covers 99 percent of the Mine Site. Impacts at this location could be expected to be the greatest, but the unaffected south branch reduced such impacts at SW-004a to less than 7 percent (see Figure 18).

For the five scenarios representing the projected five stages of the Mine Site development, the predicted reductions in minimum flows are more pronounced, with the greatest expected impact in Year 20 of mining operations when, in addition to the factors listed above for mean and maximum flows, the West Pit bottom is reaching its deepest elevation (see RS18). (Appendix B of RS22presents the evaluation of the water table drawdown -cone of depression- effect caused by dewatering of the open pits.) The expected changes in Partridge River minimum flows around the Mine Site vary from a maximum of 22 percent at surface water monitoring station SW-002 (north of the Mine Site) to less than 7 percent at surface water monitoring station SW-004a (south of the Mine Site, downstream of the confluence of the Partridge River north and south branches, and downstream of 99 percent of PolyMet's mine facilities). Changes in minimum flows are predicted to be approximately 3 percent at the outlet of the study area defined in the SDD (see Figure 20).

Reviewing the flow data recorded at the USGS gaging station #04015475 (Partridge River above Colby Lake) during periods of low flow provides an appropriate context for interpreting the predicted reductions in Partridge River minimum flows referred to in the previous paragraph. Seven out of the ten water years of record during 1978-1988 show that for the 30-day period with the lowest flows in a given water year, the daily flow variability during the corresponding 30-day period may range up to 1.6 cubic feet per second; the daily flow variability during the other three water years of record is approximately one order of magnitude greater. The catchment area of the USGS gage is 103.4 square miles, whereas the catchment area of surface water monitoring station SW-002 is 13.3 square miles, hence the ratio of catchment areas would suggest that a flow variability ranging up to 0.2 cubic feet per second at SW-002 could be expected due to climatic variability (based on 1.6 cubic feet per second flow variability range at the USGS gage). This variability of up to 0.2 cubic feet per second is of the same order of magnitude of the greatest predicted flow reduction in Partridge River minimum flows of 0.12 cubic feet per second at SW-002 (22 percent reduction with respect to Current Existing Conditions), indicating that the NorthMet Project impact on Partridge River low flows is not significant.

For the Mine Facilities Off scenario representing the hypothetical high-impact model, the predicted reductions in mean, maximum, or minimum flows are of the same order of magnitude as those predicted for Years 15 and 20.

3.4 Impacts to Partridge River Morphology

RS26 indicated there is some potential for fluvial geomorphic impacts on the Partridge River as a result of the NorthMet project. This expectation was based on the original mining concept that included increasing the discharges from the Mine Site to the Partridge River. As indicated in Section 1.3, however, the proposed project now has a water reuse/recycle strategy, with no discharge of process water to surface waters of the State. Mean annual and average daily maximum flows in the Partridge River are expected to stay the same or decrease by less than 10 percent at any of the seven surface water monitoring stations used here to evaluate the impacts of the Mine Site (see Section 3.3).

The Partridge River watershed is a mix of upland and marshland, with very little development in its watershed. The Partridge River varies from sluggish, marshy reaches to large open ponds to steep boulder rapids.

Based on the proposed decrease in flows and the watershed and river characteristics, it is reasonable to assume the following. First, most of the (natural) sediment delivery from the watershed to the main stem of the Partridge River must be relatively fine sediment (clays, silts and very fine sands) that will be conveyed as wash load, which corresponds to the fraction of fluvially transported sediment that does not affect the channel morphology. A reduction in mean annual and average daily maximum flows by less than 10 percent should not affect the transport capacity of the wash load. Second, the reaches in which the bed is covered with boulders should remain morphodynamically stable unless flows are significantly increased in magnitude and the occurrence of floods becomes more frequent. RS26 lists one potentially sensitive reach located north of the Mine Site along the railroad tracks where the reach was channelized. The anticipated impact of the Mine Site is to slightly reduce maximum flows (less than 10 percent), therefore no impacts are expected on the stream morphology of the Partridge River.

3.5 Conclusions

A hydrologic/hydraulic calibrated model was used to assess the impacts of the proposed NorthMet Project on the Partridge River flows. Basic statistics characterizing flow patterns at different locations along the Partridge River indicate that mean and maximum flows will change by less than 10 percent throughout the stages of Mine Site development. These changes in mean and maximum flows are greatest in the vicinity of the Mine Site but decrease to less than 5 percent at the outlet of the study area defined in the SDD. Changes in minimum flows are more pronounced, in particular along the north branch of the Partridge River, as a result of the water table drawdown effect caused by the mine pits (see Appendix B of RS22). During the year with the greatest expected impact, the expected reduction varies from a maximum of 22 percent north of the Mine Site, to 3 percent at the outlet of the study area defined in the SDD.

An additional modeled scenario demonstrates that a hypothetical complete removal (diversion to a different watershed) of the runoff produced from the mine facilities footprint slightly increases the impacts on the Partridge River flows.

4.0 Colby Lake-Whitewater Reservoir Impacts

4.1 Water Supply for Process Plant

The Process Plant will primarily recycle water for its operation, including water recirculated from the tailings basin and process water (previously treated, if required) reclaimed from the Mine Site; more details are presented in the RS13 and RS22 reports. In addition, the planned water management plan includes providing make-up water to the Process Plant via pumping from Colby Lake.

Cliffs Erie LLC (CE) and Minnesota Power jointly hold a permit to withdraw water from Colby Lake under Water Appropriation Permit 49-135 granted by the MDNR. It is expected that PolyMet will replace CE on the permit as part of the permitting process for the NorthMet Project. PolyMet will only use a portion of the total withdrawal allowed by the joint permit. Water Appropriation Permit 49-135 allows for a withdrawal rate of 12,000 gallons per minute for any continuous 60-day period or a peak withdrawal rate of 15,000 gallons per minute at any time. As indicated in RS13, the average annual withdrawal rate from Colby Lake for the NorthMet Project is expected to vary between 2,300 and 5,000 gallons per minute (associated to a probability of being exceeded of 90 percent and 10 percent, respectively), with a likely average annual withdrawal rate of approximately 3,500 gallons per minute, and monthly pumping rates as high as 8,000 gallons per minute (associated to a probability of being exceeded of less than 1 percent).

4.2 Existing Conditions and Data Available

4.2.1 Hydrologic System

Figure 21 is a schematic diagram of the hydrologic system that was analyzed. Inflows to the Colby Lake-Whitewater Reservoir hydrologic system include the flows from the Partridge River upstream of the confluence with Wyman Creek, the Wyman Creek flows, the flows from two unnamed tributaries at the north end of Colby Lake, municipal discharges from the City of Hoyt Lakes into Whitewater Reservoir, runoff from the watersheds directly tributary to the two water bodies and precipitation directly onto the two water bodies. Outflows and losses include the discharge from Colby Lake into Partridge River, the withdrawal of water from Colby Lake by the City of Hoyt Lakes, seepage losses from Whitewater Reservoir, open water evaporation from the two water bodies, and the projected pumping of make-up water for the NorthMet Process Plant. An additional outflow is given by the withdrawal of water by Minnesota Power for use as cooling water in its power plant at Laskin Energy Center, however this water is returned to Colby Lake, so the net outflow is zero; to be more precise, water losses from cooling at the Laskin Energy Center are negligible and were not included in the water balance calculations.

Within the hydrologic system, water is partitioned between Colby Lake and Whitewater Reservoir depending upon the criteria set in Water Appropriation Permit 49-135, as described in more detail in the Section 4.2.2. The storage capacity of Colby Lake is 5,250 acre-feet when the water level is at 1,439 feet above mean sea level, whereas the storage capacity of Whitewater Reservoir is 16,800 acre-feet when the water level is at 1,437 feet above mean sea level.

4.2.2 Water Appropriation Permit and Diversion Works

Colby Lake has served in the past as a source of processing water under Water Appropriation Permit 49-135, which was originally granted to Erie Mining Company on August 28, 1950. This permit was first transferred to LTV Steel Mining Company (LTVSMC), then to CE and Minnesota Power jointly (October 15, 2002) with the latter presently controlling the Diversion Works between Colby Lake and Whitewater Reservoir. Pumping of water from Colby Lake for supply of make-up water to LTVSMC's mineral process operations stopped in May 2000 (although some minor pumping occurred into early 2001).

Besides the conditions previously listed in Section 4.1, Water Appropriation Permit 49-135 requires that when the water level in Colby Lake falls below 1,439 feet above mean sea level due to low inflows, the withdrawal of water from Colby Lake is authorized up to the rate that can be pumped from Whitewater Reservoir to replace the water withdrawn.

In the past, in order to satisfy Water Appropriation Permit 49-135 and to secure the supply of make-up water for mineral process operations, water was allowed to flow by gravity from Colby Lake into Whitewater Reservoir through an outlet structure that consists of three sluice gates. The stored water was periodically pumped back from Whitewater Reservoir into Colby Lake via three pumps when water levels in Colby Lake fell below desired levels.

4.2.3 Water Level Data

Water level data for Colby Lake are available at the MDNR website (Station ID #69-0249-00) and from Minnesota Power (email communication from Michael Liljegren, MDNR Waters – February 1, 2007), for the periods:

- August 10, 1948 through January 28, 1980 on a bi-weekly to monthly basis;

- January 28, 1980 through August 16, 1992 on a daily basis; and,
- January 1, 1999 through December 31, 2006 on a daily basis.

Water level data for Whitewater Reservoir are available at the MDNR website (Station ID #69-0376-00) and from Minnesota Power (email communication from Michael Liljegren, MDNR Waters – February 1, 2007), for the periods:

- August 10, 1948 through October 14, 1980 on a bi-weekly to monthly basis; and,
- January 14, 2002 through December 31, 2006 on a daily basis.

Additional water level information for both water bodies is available for the period August 17, 1938 through August 10, 1948, with an average of two measurements per year.

Three periods of analysis can be defined:

- "Before mining" – Water level data recorded before 1955, the year in which the Whitewater Reservoir (formerly known as Partridge Lake) was constructed to help secure make-up water for LTVSMC operation. Figure 22 shows that the maximum annual water level fluctuation in Colby Lake was 4.6 feet, and water levels were below elevation 1,439 feet above mean sea level during periods in two out of the 17 years of record. On the other hand, Figure 25 shows that the maximum annual water level fluctuation in Whitewater Reservoir (called the Partridge Lake at this time) was 2.0 feet, and that the mean water level was 33.0 feet below that in Colby Lake.
- "During mining" – Water level data recorded between 1955 and 1992 in Colby Lake, and water level data recorded between 1955 and 1980 in Whitewater Reservoir. LTVSMC stopped withdrawing water from Colby Lake in May 2000; CE has not withdrawn water from Colby Lake after this date. Although pumping continued until 2000, water level data is not available between 1992 and 2000 in Colby Lake nor is it available between 1980 and 2000 in Whitewater Reservoir. Water level data from this period show the effect of operation of Whitewater Reservoir to store water and maintain Colby Lake water levels during mining. Figure 23 shows that the maximum annual water level fluctuation in Colby Lake decreased (with respect to the period "Before mining") to 4.1 feet, and water levels were below elevation 1,439 feet above mean sea level during periods in 36 out of the 37 years of record. On the other hand, Figure 26 shows that the maximum annual water level fluctuation in Whitewater Reservoir

increased (with respect to the period “Before mining”) to as much as 14.3 feet, the mean water level was approximately 30 feet above that during the period “Before mining”, and water levels were above those in Colby Lake during several periods. The water level data show an overall positive trend that raises questions regarding the reliability of Whitewater Reservoir water level data for the period “During mining”.

- “After mining” – Water level data recorded between 2000 and 2006. Figure 24 shows that, similar to the period “During mining”, the maximum annual water level fluctuation in Colby Lake was 3.7 feet and in five out of the six years of record water levels were below elevation 1,439 feet above mean sea level during short periods of time. On the other hand, Figure 27 shows that the maximum annual water level fluctuation in Whitewater Reservoir decreased (with respect to the period “During mining”) to 4.3 feet, and water levels were above those in Colby Lake during short periods in water year 2003-2004.

As explicitly requested by the agencies, the period October 1, 2001 through December 31, 2006 for Colby Lake and the period January 14, 2002 through December 31, 2006 for Whitewater Reservoir are deemed to be “baseline” for purposes of the EIS. These periods reflect the best data available during the most recent operating conditions, as described below.

4.2.4 Flow Data

Existing gage flow data are available on the Partridge River both upstream and downstream of Colby Lake (see Figure 1; more details about gaging stations are provided in RS73A). Daily flow data are available at USGS gaging station #04015475 – Partridge River above Colby Lake at Hoyt Lakes for the period September 19, 1978 through November 2, 1988; at USGS gaging station #04015500 – Partridge River tributary Second Creek near Aurora for the period April 1, 1955 through September 30, 1980; and at USGS gaging station #04016000 – Partridge River near Aurora for the period August 1, 1942 through September 30, 1982.

The coincident period of record for these three stations (henceforth called “historic data period”) extends between September 19, 1978 and September 30, 1980. As indicated in Section 4.0 of RS73A, mine discharges could have had a significant effect on the flows recorded at these three gaging stations, in particular Second Creek. Such an effect cannot be quantified because complete information on mine discharges is unavailable before 1988. Therefore, average flows obtained from different periods of record would have this uncertainty/bias associated with them. Because of the data limitations, it was considered appropriate to use the historic data period for reviewing historical water levels at Colby Lake and Whitewater Reservoir.

4.3 Water Balance for Historic Data Period

4.3.1 Inflows and Outflows

Flow data from the USGS gaging station at Partridge River above Colby Lake were used to estimate the combined inflows to Colby Lake from the Partridge River (gaged at this station), Wyman Creek and two unnamed tributaries at the north end of Colby Lake. The catchment area of the Partridge River gaged at this station is 66,174 acres, which represents 81 percent of the total catchment area of Colby Lake (81,771 acres). The average flow recorded at this gaging station during the historic data period is 77.5 cubic feet per second, which if extrapolated throughout the watershed would be equivalent to a total inflow into Colby Lake of 95.7 cubic feet per second.

The average flows recorded at USGS gaging stations at Partridge River near Aurora and at Second Creek near Aurora during the historic data period are 87.6 and 18.6 cubic feet per second, respectively. Second Creek is the tributary of Partridge River that discharges downstream of Colby Lake but upstream of USGS gaging station #04016000 (Partridge River near Aurora). Therefore, the average flow in the Partridge River that was discharged from Colby Lake during the historic data period would be the difference between these two flow records, or 69 cubic feet per second. An equivalent expression would be that the outflow from Colby Lake into the Partridge River corresponds to 72 percent of the total inflow during the historic data period.

Two other components of the water balance, net precipitation and water withdrawal and discharge by the City of Hoyt Lakes, were considered negligible and were not included in the water balance calculations. Net precipitation (i.e., precipitation minus evaporation) is about 8 inches per year in this area. The combined surface area of Colby Lake and Whitewater Reservoir is 1,710 acres, for which the equivalent average inflow due to net precipitation would be 1.6 cubic feet per second. This value is more than one order of magnitude smaller than the surface inflows and outflows associated with the Partridge River and other tributaries. A similar criterion has been followed to neglect the withdrawal and discharge of water by the City of Hoyt Lakes (approximately 0.5 cubic feet per second) from the water balance calculations.

Seepage losses from Whitewater Reservoir were estimated by Barr (1964) to range between 1 cubic foot per second for a water level at 1,426 feet above mean sea level to 15 cubic feet per second for a water level at 1,440 feet above mean sea level (Figure 28). The validity of these estimates has been confirmed by Adams et al. (2004) after comparison of measured water levels at Whitewater Reservoir during the winters of 2001 and 2002 and corresponding calculated seepage losses. Barr (1964) noted that because the “reservoir is constructed between ridges of glacial drift which contain a number of irregular gravel deposits”, “water stored in the reservoir has been seeping out of the reservoir through these gravel deposits, emerging in potholes and swamps in the surrounding forest and eventually finding its way back to either the St. Louis River or the Partridge River.” It has been assumed that although part of the seepage losses from Whitewater Reservoir may constitute groundwater recharge to the Partridge River, this occurs downstream of USGS gaging station #04016000 (Partridge River near Aurora), so the seepage losses represent net outflows from the Colby Lake-Whitewater Reservoir hydrologic system.

Water appropriations from Colby Lake that are used to provide make-up water for mineral process operations have been reported to the MDNR between 1988 and 2000, the last complete year of operations at LTVSMC. When the mineral processing was operational between 1988 and 1993, the reported pumping rates from Colby Lake varied between 18.7 and 23.9 cubic feet per second, with an average value of 21.1 cubic feet per second (9,470 gallons per minute; that is, approximately three times the expected average annual withdrawal rate by the NorthMet Project – see Section 4.1). This rate was used as an estimate of the withdrawal during the historic data period.

4.3.2 Water Balance Results

A review of the water balance for the historic data period (see results in Table 5) indicates that surface inflows accounted for an average of 95.7 cubic feet per second, while the average discharge from Colby Lake into the downstream Partridge River was 69 cubic feet per second. For the Colby Lake-Whitewater Reservoir hydrologic system to have been in balance over the 2-year historic data period (meaning no positive or negative trend in the water levels), seepage losses from Whitewater Reservoir should have been 5.6 cubic feet per second (Inflows of 95.7 cubic feet per second minus Partridge River flows downstream of Colby Lake of 69 cubic feet per second minus mine make-up water of 21.1 cubic feet per second). These seepage losses are equivalent to a mean water level at Whitewater Reservoir of 1,435.8 feet above mean sea level (see Figure 28). However, water levels recorded at Whitewater Reservoir indicate the average value during the historic data period is 1,440.7 feet above mean sea level, for which seepage losses would have been greater than 15 cubic feet per second. This appears to be a gap between the average water level data and the average flow data.

The water level data presented in Figures 23 and 26 suggest an unusual situation where water levels at Whitewater Reservoir were about 1.5 feet higher than water levels at Colby Lake during the historic data period. The opposite is expected unless surface inflows to Colby Lake were large enough to satisfy the mineral processing demand and yet maintain the water level at Colby Lake above 1,439 feet above mean sea level without need to divert water from Colby Lake to Whitewater Reservoir or to pump water from Whitewater Reservoir to Colby Lake. This would in turn mean that inflows from the direct catchment area of Whitewater Reservoir were sufficient to balance seepage losses and maintain relatively high water levels in the reservoir. Because this would be unusual over a two year period, it appears that the water level data during the historic data period may not be correct. This is one reason why the water balance simulations presented in Section 4.4 do not make use of the historic data period information. This period with the most historic data also includes mine appropriations which are not defined in detail and may skew the results.

4.4 Water Balance for Projected Conditions

Daily water level data exist for Colby Lake and Whitewater Reservoir for the period “After mining” (see Figures 24 and 27). The MDNR requested that this record of water levels (described in Section 4.2.3) be used to analyze the impacts to the Colby Lake-Whitewater Reservoir hydrologic system during NorthMet operations. It is important to recall that water level data for Whitewater Reservoir were not available before October 1, 2001, hence the results presented in this document do not include water year 2000 (October 1, 2000 through September 30, 2001). Furthermore, the results presented in this document do not include water year 2005 (October 1, 2005 through September 30, 2006) because at the time the analysis was completed, spatially distributed precipitation for the study area had not been generated for this water year to use as input in the XP-SWMM model of the Partridge River watershed. However, Figure 24 (for Colby Lake) and Figure 27 (for Whitewater Reservoir) show that the water level fluctuations during water year 2005 are bracketed by the water level data between October 1, 2001 and September 30, 2005.

Water balance calculations were conducted for the period October 1, 2001 through September 30, 2005 using:

- Recorded water level data for the period “After mining” (see Section 4.2.3),
- Inflows predicted using the hydrologic/hydraulic model developed for the Partridge River watershed (see Section 2.4),

- Outflows from Colby Lake to the Partridge River downstream obtained from a rating curve revised by Adams et al. (2004),

- Estimates of seepage losses from Whitewater Reservoir (see Section 4.3.1), and

- Information on gate and pump operation provided by the MDNR (see Section 4.2.2).

The water balance for this period was then updated to evaluate the potential impacts of the proposed NorthMet project on the water levels of the two water bodies, using revised inflows during mining and withdrawals for make-up water. These water balance calculations served to evaluate the potential impacts of the proposed NorthMet project on the water levels of Colby Lake and Whitewater Reservoir.

Comparisons are made between modeled water level fluctuations for Current Existing Conditions driven by climate normal precipitation (i.e., the base case) and modeled water level fluctuations under conditions reflecting the expected impact of PolyMet operations. The model results cannot be compared to recorded fluctuations because of lack of concurrent data: flow data in the Partridge River are not available for the period when water level data in Colby Lake and Whitewater Reservoir are available and mining activities were not taking place (that is, the period when LTVSMC stopped withdrawing water from Colby Lake).

4.4.1 Estimating Inflows to Colby Lake

No streamflow data is available for the period “After mining”. For Current Existing Conditions in which there is no NorthMet mining activity, the hydrologic/hydraulic model was used to predict flows in the Partridge River watershed (accounting for 81 percent of the tributary area of Colby Lake-Whitewater Reservoir) during this period, based on recorded precipitation data. This information combined with the water levels recorded in Colby Lake and Whitewater Reservoir allowed a cumulative mass balance approach to estimating flows in the remaining catchment area (accounting for the remaining 19 percent of the tributary area of Colby Lake-Whitewater Reservoir) that is not included in the hydrologic/hydraulic model referred to previously.

These calculations estimated that the average flow from the Partridge River watershed was 71.0 cubic feet per second, and the average flow from the entire tributary area of Colby Lake-Whitewater Reservoir was 91.9 cubic feet per second. Both of these flow estimates are similar to the ones presented in Section 4.3.1, which were obtained from gaged data. The difference is in the outflow from Colby Lake to the Partridge River, which for the period “After mining” is 90 percent of the total inflows as opposed to the estimate of 72 percent for the period “During mining” (more specifically, the historic data period) presented in Section 4.3.1. This difference is explained by the withdrawal of water by LTVSMC for its mineral processing operations. The maximum and minimum daily inflows simulated with the hydrologic/hydraulic model were 1,807 and 3.2 cubic feet per second, respectively. These flow values are similar to the variability recorded during the 1978-1988 period, in which maximum and minimum daily inflows were 1,960 and 0.54 cubic feet per second, respectively. On the other hand, it is important to mention that the mean annual precipitation during the period October 1, 2001 through September 30, 2005 corresponded to 87 percent of the mean annual precipitation during the period of climate normal (see Section 2.2.1).

As indicated in Section 3.3, the greatest expected impact (reduction) on the Partridge River mean annual flows will occur around Year 15 to Year 20 of mining operations. Therefore, these scenarios represent the greatest reduction in Partridge River flows due to the NorthMet Project, and so depict expected lowest inflows to the Colby Lake-Whitewater Reservoir hydrologic system and greatest impact on water level fluctuations in the two water bodies due to the Mine Site development (without considering climatic variability).

Inflows from the tributary area were estimated based on the results of the hydrologic/hydraulic model for the Partridge River watershed under Year 15 conditions (for both modeled climate normal precipitation and modeled 2001-2005 precipitation) and the flows estimated with the water balance calculation described in the previous paragraph for the remaining 19 percent of the tributary area. These inflows were used for the water balance simulations presented in Section 4.4.3.

Comparisons against long periods of drought in the Partridge River are also included in this analysis to account for the potential effect of climatic variability in addition to that due to the NorthMet Project. The modeled inflows for the period October 1, 2001 through September 30, 2005 that would correspond to the mean annual precipitation associated with the climate normal (see Section 2.2.1) were reduced by 21 percent, 27 percent and 31 percent to represent low flows with recurrence of 10, 25 and 50 years, respectively. These percent reductions are based on a frequency analysis of precipitation data for the period defining the climate normal (see Appendix B of RS74); the analysis assumes that the low flows are extended for the entire four-year period October 1, 2001 through September 30, 2005. Such an assumption is equivalent to probabilities of occurrence (over the 4-year period of analysis) of 7.48 percent for the 2001-2005 flow conditions, of 1.26 percent for the 10-year low-flow conditions, of 0.12 percent for the 25-year low-flow conditions, and of 0.02 percent for the 50-year low-flow conditions.

The results of three flow conditions evaluated for the period 2001-2005 are presented in the main text of this RS73B report:

1. Average flow conditions, based on climate normal precipitation.
2. 2001-2005 flow conditions, based on 2001-2005 precipitation.
3. 50-year low-flow conditions. (The results for 10-year and 25-year low-flow conditions are presented in Appendix B.)

4.4.2 Make-up Water Demand for NorthMet

Two constant make-up water demand rates were evaluated: the expected average annual withdrawal rate of approximately 3,500 gallons per minute (7.8 cubic feet per second), and an annual withdrawal rate of 5,000 gallons per minute (11.1 cubic feet per second), which has a probability of being exceeded once every 10 years (see RS13). A third case was evaluated to determine the effect of the monthly make-up water demand rate of 8,000 gallons per minute (17.8 cubic feet per second). More specifically, the third case analyzed consisted of combining a demand of 8,000 gallons per minute during three months of the year and 4,400 gallons per minute during the other nine months of year (henceforth called the "combined high demand"); the equivalent annual demand has a probability of being exceeded once every 100 years (see RS13).

The water balance calculations presented in Section 4.4.3 consider that the make-up water is constantly pumped from Colby Lake. When inflows to Colby Lake from its tributary area are insufficient to simultaneously satisfy the make-up water demand and the conditions of Water Appropriation Permit 49-135, water is pumped from Whitewater Reservoir to Colby Lake.

4.4.3 Operation of Diversion Works

The main objective of the water balance calculations is to determine if the make-up water demand for the NorthMet Project can be satisfied simultaneously with the reductions in Partridge River flows during the proposed 20-year period of mining operations while still complying with the requirements established in Water Appropriation Permit 49-135.

The MDNR has expressed a desire to minimize the water level fluctuations in Colby Lake and Whitewater Reservoir during mine operation. For modeling purposes, four operational criteria based on water surface elevations in Colby Lake and Whitewater Reservoir and inflows to these two water bodies were selected. In the water balance calculations, the sluice gates allowing water to flow from

Colby Lake to Whitewater Reservoir are opened for a period of 24 hours whenever all of the following conditions are met on any given day:

1. The water level in Colby Lake is above a threshold elevation. Three threshold elevations have been evaluated: 1,439.25 feet above mean sea level, 1,439.50 feet above mean sea level, and 1,439.75 feet above mean sea level. This criterion seeks to reduce the potential for water levels in Colby Lake to fall below 1,439 feet above mean sea level during periods of low flow. This is a conservative assumption because water levels in Colby Lake fell below 1,439 feet above mean sea level during the “Before mining” and “After mining” periods (see Section 4.2.3).

2. The water level in Whitewater Reservoir is below an elevation of 1,440 feet above mean sea level. This criterion reduces water level fluctuations in Whitewater Reservoir.

3. The water level in Colby Lake is greater than the water level in Whitewater Reservoir. This allows water to flow in one direction only, from Colby Lake to Whitewater Reservoir.

4. The inflows from the Partridge River and other tributaries to Colby Lake and Whitewater Reservoir is greater than a given flow value. This flow value was found by iteration, such that water level fluctuations in Colby Lake are minimized when the water surface of Colby Lake is near 1,439 feet above mean sea level.

The Diversion Works operational criteria listed above for diverting water from Colby Lake to Whitewater Reservoir allow any of the three sluice gates to be opened on any given day. Section 4.4.4 presents a comparison of the impacts on minimum water levels in Colby Lake and Whitewater Reservoir as a result of the number of sluice gates to open: one, two or three.

As indicated in Section 4.2.2, pumping from Whitewater Reservoir to Colby Lake is required to avoid water levels in Colby Lake to fall below desired levels. Pumping may occur at rates of 4,000 gallons per minute, 8,000 gallons per minute, or 12,000 gallons per minute depending on how many of the three existing pumps are operated to provide a flow equal to or greater than the make-up water demand from NorthMet. The installed capacity of each of the pumps is 4,000 gallons per minute.

4.4.4 Water Balance Results

In general and as agreed with the agencies, the discussion of the water balance results presented below refers to annual values in the case of Colby Lake and to the period April-October in the case of Whitewater Reservoir. The complete set of tabulated results is presented in Appendix B.

The “base case” referred to below corresponds to average flow conditions driven by climate normal precipitation, and zero-demand.

Selected Water Management Strategy for Operation of Diversion Works

The invert (runout elevation) of the outlet structure in Colby Lake allowing water to flow downstream to the Partridge River is at 1,438.50 feet above mean sea level. Put more simply, when water levels in Colby Lake fall below 1,438.50 feet above mean sea level, there is no outflow from Colby Lake to the Partridge River. The MDNR has indicated this is not an acceptable situation. Therefore, the water balance calculations presented in this RS73B report have been based on criteria for operation of the Diversion Works (see Section 4.4.3) that make sure this situation will not occur, while securing supply of make-up water for the NorthMet Project and minimizing water level fluctuations in both Colby Lake and Whitewater Reservoir.

The water balance results presented in this section allow selection of a water management strategy for operation of the Diversion Works that is in compliance with Water Appropriation Permit 49-135 and meets the abovementioned objectives from the MDNR. All the water balance simulations were based on: a) average flow conditions (i.e., based on climate normal precipitation) for the period 2001-2005; and b) a make-up water demand of 3,500 gallons per minute (i.e., the expected average annual make-up water demand). The results allow comparison of the combined effect of different threshold elevations (i.e., the first criterion for operation of the Diversion Works) and the number of sluice gates allowed to be opened for diversion of water from Colby Lake to Whitewater Reservoir. The numbering of the different combinations of threshold elevations and number of sluice gates to open is presented in Table 6. Furthermore, the water balance results presented in this section include the determination of the inflow criterion (see Table 7) for diversion of water from Colby Lake to Whitewater Reservoir (i.e., the fourth criterion for operation of the Diversion Works).

A summary of the water balance results for all scenarios listed in Table 6 is presented in Figures 29 and 30. These figures show that Scenario 2b provides one suitable water management strategy for operation of the Diversion Works because:

1. The minimum water level in Colby Lake is above the runout elevation of 1,438.50 feet above mean sea level at all times.
2. The maximum water level fluctuation in Colby Lake is just 0.1 feet greater than the smallest value of the nine scenarios evaluated.
3. The maximum water level fluctuation in Whitewater Reservoir is the smallest value of the nine scenarios evaluated.
4. There is no need for the fourth criterion considered for operation of Diversion Works (see Table 7).

Therefore, Scenario 2b is selected as the water management strategy to use for further evaluation of the NorthMet Project impacts on water levels in Colby Lake and Whitewater Reservoir under other flow conditions and make-up water demands.

<u>**NorthMet Project Effects Only (Scenario 2b and Average Flow Conditions)**</u>

The water balance results presented in Table 8 indicate that a NorthMet Project make-up water demand of 3,500 or 5,000 gallons per minute combined with the greatest reduction in Partridge River flows due to the Mine Site development (i.e., Year 15) have a very small impact on the average water surface elevation in Colby Lake (0.1 feet). The water level fluctuation in Colby Lake decreases from 3.9 feet with the base case to 3.6 feet with a make-up water demand of 3,500 or 5,000 gallons per minute. The main difference between the modeled base case and the modeled conditions with the NorthMet Project is given by the number of days per year that pumping from Whitewater Reservoir to Colby Lake must take place in order to satisfy the make-up water demand: 197 days per year of pumping for the 3,500 gallons per minute-demand, and 204 days per year of pumping for the 5,000 gallons per minute-demand. Overall, changes in Colby Lake water levels are minimal with the NorthMet Project (see Figures 31 and 32).

The water balance results presented in Table 8 indicate that for the period April-October, the average water surface elevation in Whitewater Reservoir decreases with respect to the base case by 0.4 feet with the 3,500 gallons per minute-demand, and by 1 foot with the 5,000 gallons per minute-demand; the decrease is a result of the NorthMet make-up water demand. The main impact of this NorthMet make-up water demand is on the maximum water level fluctuations in Whitewater Reservoir, in particular due to changes in the predicted minimum water elevations (see Figures 31 and 33). Water level fluctuations during the period April-October increase from 2.9 feet in the base case to 4.2 feet with a make-up water demand of 3,500 gallons per minute, and to 6.8 feet with a make-up water demand of 5,000 gallons per minute.

Climatic Variability Effects (2001-2005 Flow Conditions, and 50-Year Low-Flow Conditions) in Addition to NorthMet Project Effects (Scenario 2b)

Table 9 compares the model results for zero-demand to a model selected water management strategy (Scenario 2b) with NorthMet make-up water demands of 3,500 and 5,000 gallons per minute, under conditions of reduced inflows to the Colby Lake-Whitewater Reservoir hydrologic system that are representative of 2001-2005 precipitation conditions (approximately 87 percent of climate normal precipitation). Similarly, Table 10 compares the model results for zero-demand to a model selected water management strategy (Scenario 2b) with NorthMet make-up water demands of 3,500 and 5,000 gallons per minute as well as the combined high demand, under conditions of reduced inflows to the Colby Lake-Whitewater Reservoir hydrologic system that are representative of a long period of drought lasting four years with recurrence of 50 years (approximately 69 percent of climate normal precipitation). Results for low flows with recurrence of 10 and 25 years are included in Appendix B.

The water balance results presented in Table 9 indicate that, for 2001-2005 flow conditions, NorthMet make-up water demands of 3,500 and 5,000 gallons per minute not only can be fully satisfied following the criteria established under Water Appropriation Permit 49-135 (see Section 4.1), but also have a very small impact on both the average water surface elevation and the maximum water level fluctuation in Colby Lake (variations are less than 0.1 feet when compared to the corresponding zero-demand case; see Figures 34 and 35). Reduced inflows to the Colby Lake-Whitewater hydrologic system have a larger impact on water surface elevations in Whitewater Reservoir (see Figures 34 and 36). Table 9 shows that for the period April-October, the average water surface elevation in Whitewater Reservoir decreases with respect to the zero-demand case by 0.5 feet with the 3,500 gallons per minute-demand, and by 1.2 feet with the 5,000 gallons per minute-demand; the decrease is a result of the NorthMet make-up water demand. Water level fluctuations in Whitewater Reservoir during the period April-October increase from 2.8 feet in the zero-demand case to 4.6 feet with a make-up water demand of 3,500 gallons per minute, and to 7.8 feet with a make-up water demand of 5,000 gallons per minute.

The water balance results presented in Table 10 indicate that, for a hypothetical 4-year period of drought with recurrence of 50 years, NorthMet make-up water demands of 3,500 and 5,000 gallons per minute as well as the combined high demand not only can be fully satisfied following the criteria established under Water Appropriation Permit 49-135 (see Section 4.1), but also have a very small impact on both the average water surface elevation and the maximum water level fluctuation in Colby Lake (variations are less than 0.1 feet when compared to the corresponding zero-demand case; see Figures 37 and 38). Reduced inflows to the Colby Lake-Whitewater hydrologic system have a larger impact on water surface elevations in Whitewater Reservoir (see Figures 37 and 39). Table 10 shows that for the period April-October, the average water surface elevation in Whitewater Reservoir decreases with respect to the zero-demand case by 0.7 feet with the 3,500 gallons per minute-demand, by 1.7 feet with the 5,000 gallons per minute-demand and with the combined high demand; the decrease is a result of the NorthMet make-up water demand. Water level fluctuations in Whitewater Reservoir during the period April-October increase from 2.8 feet in the zero-demand case to 5.9 feet with a make-up water demand of 3,500 gallons per minute, to 9.9 feet with a make-up water demand of 5,000 gallons per minute, and to 9.7 feet with the combined high demand.

4.5 Conclusions

The main conclusion from the predictive water balance calculations is that the make-up water demand for the NorthMet Project can be satisfied by the Colby Lake-Whitewater Reservoir hydrologic system during mining operations while staying in compliance with the Water Appropriation Permit 49-135. Considering average inflow conditions and the likeliest make-up water demand of 3,500 gallons per minute, maximum water level fluctuations in Colby Lake and Whitewater Reservoir (for the period April-October) would vary by no more than 0.3 and 1.4 feet with respect to the base case, respectively (see Table 8). Although the model selected water management strategy (that is, Scenario 2b with a threshold elevation for diverting water from Colby Lake to Whitewater Reservoir set at 1,439.50 feet above mean sea level, and only two gates allowed to be opened for this diversion operation) seeks to minimize water level fluctuations in both Colby Lake and Whitewater Reservoir, it is inevitable that the main impact of the NorthMet Project be an increase in water level fluctuations in Whitewater Reservoir.

It is important to recall that the measured water level fluctuation in Colby Lake during the period of LTVSMC operation was as high as 4.1 feet (see Figure 23), which is greater than the water level fluctuations modeled after including the impacts of the NorthMet Project greatest reduction in Partridge River flows (around Mine Year 15) and any of the flow conditions and projected water withdrawal rates from Colby Lake that have been evaluated in this RS73B report (see Tables 8, 9 and 10). Furthermore, the fraction of time that water levels in Colby Lake fall below 1,439 feet above mean sea level is 9.0 percent for model average flow conditions and the likeliest make-up water demand of 3,500 gallons per minute, which is less than the corresponding 11.2 percent recorded during the period of LTVSMC operation.

It is also worth recalling that the measured water level fluctuation in Whitewater Reservoir during the period of LTVSMC operation was as high as 14 feet in one year (see Figure 26), whereas modeled water level fluctuations (April-October) considering the impacts of the NorthMet Project for the very conservative combination given by the combined high demand (with a probability of being exceeded once every 100 years) and 50-year low-flow conditions (with a probability of occurrence over the 4-year period of analysis of 0.02 percent) are no more than 9.7 feet (see Table 10). Impacts on minimum water levels in Whitewater Reservoir are better depicted in Figures 40 through 42, which are based on the water balance results presented in Tables 8 through 10 and the recent bathymetric surveys conducted by the MDNR on the east part of Whitewater Reservoir (email communication from Mike Liljegren on September 27, 2007). These figures show the shoreline corresponding to estimated average water levels for zero-demand and minimum water levels for zero-demand and 3,500 and 5,000 gallons per minute-demand under average flow conditions, 2001-2005 flow conditions, and 50-year low-flow conditions, respectively. When comparing minimum water levels for the zero-demand and the annual average 3,500 gallons per minute-demand under average flow conditions (see Figure 40), it can be seen that the shoreline retreat is less than 10 feet except in two short, localized reaches where the shoreline retreat can be as much as approximately 75 feet; this shoreline retreat will be evident during less than 10 percent of the period April-October (see Figure 33).

Alternative sources of make-up water during extreme drought conditions are discussed in RS13.

5.0 References

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Tables

Table 1: Tributary areas and percent reductions (with respect to existing conditions) at flow reporting locations in the Partridge River for different stages of Mine Site development

Location	**Existing Conditions**		**Year 1**		**Year 5**		**Year 10**		**Year 15**		**Year 20**		**Mine Facilities Off**	
	Area (sq mi)	**%Red**	**Area (sq mi)**	**%Red**	**Area (sq mi)**	**%Red**	**Area (sq mi)**	**%Red**	**Area (sq mi)**	**%Red**	**Area (sq mi)**	**%Red**	**Area (sq mi)**	**%Red**
SW-001	6.22	0.0	6.22	0.0	6.22	0.0	6.22	0.0	6.22	0.0	6.22	0.0	6.22	0.0
SW-002	13.30	0.0	12.93	2.8	12.89	3.1	12.85	3.4	12.85	3.4	12.85	3.4	12.85	3.4
SW-003	15.16	0.0	14.81	2.3	14.74	2.8	14.64	3.4	14.65	3.4	14.65	3.3	14.64	3.4
SW-004	23.01	0.0	21.98	4.5	21.78	5.4	21.61	6.1	21.51	6.5	21.52	6.5	21.50	6.6
SW-004a	54.14	0.0	52.70	2.7	52.08	3.8	51.63	4.6	51.44	5.0	51.40	5.1	51.42	5.0
SW-005	98.72	0.0	97.28	1.5	96.67	2.1	96.20	2.6	96.01	2.7	96.02	2.7	95.99	2.8
USGS Gage	103.40	0.0	101.95	1.4	101.34	2.0	100.87	2.4	100.69	2.6	100.70	2.6	100.67	2.6

Table 2: Flow statistics at USGS gaging station #04015475 and six surface water monitoring stations (including correction accounting for ratios of the recorded versus calibrated modeled values) for the 10-year period from 1978-1988

Statistic	**Units**	**Location**						
		USGS Gage	**SW-005**	**SW-004a**	**SW-004**	**SW-003**	**SW-002**	**SW-001**
Mean Annual Flow	cfs	88	83	45	19	12	11	4.7
Max 1-Day Flow	cfs	1,960	1,859	1,163	385	246	193	68
Avg. Max 1-Day Flow	cfs	748	722	474	166	107	90	32
Max 3-Day Flow	cfs	1,840	1,753	1,002	365	214	173	57
Max 7-Day Flow	cfs	1,446	1,380	759	291	171	140	42
Max 30-Day Flow	cfs	710	676	356	148	91	77	30
Max 90-Day Flow	cfs	362	344	180	75	46	39	15
Min 1-Day Flow	cfs	0.54	0.49	0.22	0.09	0.07	0.05	0.01
Avg. Min 1-Day Flow	cfs	3.6	3.3	1.6	0.62	0.42	0.32	0.06
Min 3-Day Flow	cfs	0.65	0.59	0.28	0.11	0.08	0.06	0.01
Min 7-Day Flow	cfs	0.79	0.68	0.32	0.13	0.09	0.07	0.01
Min 30-Day Flow	cfs	1.2	1.1	0.55	0.21	0.15	0.12	0.03
Min 90-Day Flow	cfs	2.2	2.1	1.15	0.52	0.34	0.29	0.11

Table 3: Estimated runoff from reclaimed stockpiles, reported as a percentage of precipitation

Stockpile Type	**Runoff (Percent of Precipitation)**
Category 1/2	5%
Category 3	8%
Category 4	10%

Table 4: Effective catchment areas for seven scenarios modeled with XP-SWMM calibrated model for the Partridge River watershed

Scenario	**Catchment area (square miles)**
Existing Conditions	103.4
Year 1	102.0
Year 5	101.3
Year 10	100.9
Year 15	100.7
Year 20	100.7
Mine Facilities Off	100.7

Table 5: Colby Lake historic data period water balance results demonstrating inconsistency between water levels and inflows and outflows

Month-Year	Total Flow to Colby Lake (cfs)	Total Flow from Colby Lake (cfs)	Water Level in Colby Lake (ft)	Water Level in Whitewater Reservoir (ft)	Seepage Loss from Whitewater Reservoir (cfs)	Estimated Mine Demand (cfs)	Colby Lake Storage (acre-ft)	Whitewater Storage (acre-ft)	Change in Storage (cfs)	Inflow minus Outflow (cfs)
October-78	45.6	22.7	1,439.06	1,440.39	15.8	21.3	6,108	20,937		-14.2
November-78	29.0	14.9	1,438.91	1,440.51	15.8	21.3	6,046	21,108	1.82	-24.7
December-78	21.1	15.7	1,438.80	1,440.20	15.8	21.3	6,000	20,670	-8.13	-23.5
January-79	2.6	8.2	1,438.85	1,440.01	15.8	21.3	6,021	20,406	-4.09	-38.6
February-79	1.6	8.9	1,438.85	1,439.99	15.1	21.3	6,021	20,378	-0.46	-43.2
March-79	10.8	12.3	1,439.15	1,439.50	13.7	21.3	6,146	19,714	-9.07	-27.4
April-79	561.0	408.1	1,439.45	1,440.00	15.8	21.3	6,271	20,392	13.50	102.3
May-79	491.1	425.5	1,439.75	1,440.50	15.8	21.3	6,396	21,093	13.89	14.6
June-79	245.4	205.6	1,440.05	1,441.34	15.8	21.3	6,521	22,326	22.83	-20.1
July-79	51.9	42.5	1,439.55	1,441.09	15.8	21.3	6,312	21,952	-9.80	-17.9
August-79	49.9	34.2	1,438.91	1,440.99	15.8	21.3	6,046	21,804	-6.97	-14.4
September-79	79.3	37.4	1,439.39	1,441.09	15.8	21.3	6,246	21,952	5.85	-1.0
October-79	42.1	18.8	1,438.89	1,440.96	15.8	21.3	6,038	21,760	-6.73	-7.1
November-79	92.9	47.2	1,439.61	1,442.01	15.8	21.3	6,337	23,362	31.97	-23.4
December-79	18.7	21.7	1,439.09	1,441.15	15.8	21.3	6,121	22,041	-25.83	-14.3
January-80	6.5	16.1	1,439.00	1,440.97	15.8	21.3	6,083	21,775	-5.11	-41.6
February-80	2.4	10.0	1,438.90	1,440.00	15.8	21.3	6,042	20,392	-23.95	-20.7
March-80	2.2	5.8	1,438.80	1,438.97	11.8	21.3	6,000	19,019	-23.77	-12.9
April-80	180.9	73.6	1,439.10	1,438.89	11.8	21.3	6,125	18,917	0.38	73.9
May-80	57.2	55.7	1,439.50	1,441.29	15.8	21.3	6,292	22,251	58.85	-94.4
June-80	47.5	35.7	1,439.60	1,441.91	15.8	21.3	6,333	23,204	16.72	-42.0
July-80	12.1	13.5	1,439.01	1,441.04	15.8	21.3	6,087	21,878	-26.42	-12.0
August-80	18.2	9.1	1,438.71	1,439.69	14.4	21.3	5,962	19,969	-34.20	7.7
Average	90.0	67.1	1,439.17	1,440.54	15.2	21.3	6,155	21,187	-0.9	-12.8

Table 6: Numbering of different combinations of threshold elevations and number of sluice gates to open for diversion of water from Colby Lake to Whitewater Reservoir

Scenario	Threshold elevation (feet-MSL)	Number of gates to open
1a	1,439.25	3
1b	1,439.25	2
1c	1,439.25	1
2a	1,439.50	3
2b	1,439.50	2
2c	1,439.50	1
3a	1,439.75	3
3b	1,439.75	2
3c	1,439.75	1

Table 7: Inflow criterion (cfs) for diversion of water from Colby Lake to Whitewater Reservoir, under zero-demand and 3,500 gallons per minute-demand

Scenario	0 gpm	3,500 gpm
1a	140	180
1b	50	140
1c	0	0
2a	0	180
2b	0	0
2c	0	0
3a	0	180
3b	0	0
3c	0	0

Table 8: 4-year model results comparing water level impacts for various make-up water demands assuming average flow conditions in the Partridge River, with Colby Lake water level above 1,439.50 feet above mean sea level for water to be diverted to Whitewater Reservoir via 2 sluice gates (Scenario 2b)

	Colby Lake			**Whitewater Reservoir**		
	Base Case	**Projected Future Conditions**		**Base Case**	**Projected Future Conditions**	
Make-up water demand (gpm)	0	3,500	5,000	0	3,500	5,000
Average Elevation[1] (feet)	1,439.45	1,439.42	1,439.44	1,439.33	1,438.94	1,438.33
Maximum Elevation[1] (feet)	1,442.75	1,442.51	1,442.45	1,440.26	1,440.25	1,440.23
Minimum Elevation[1] (feet)	1,438.85	1,438.88	1,438.84	1,437.41	1,435.98	1,433.34
Maximum Fluctuation[1] (feet)	3.90	3.63	3.61	2.85	4.22	6.84
Days Pumping Into Colby Lake[2]	NA	NA	NA	0	787	815
Days Flowing Into Whitewater Reservoir[2]	156	161	174	NA	NA	NA
Percent Time[2] Below 1,439 feet	10.5	9.0	0.5	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

Table 9: 4-year model results comparing water level impacts for various make-up water demands assuming 2001-2005 flow conditions (approximately 87 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.50 feet above mean sea level for water to be diverted to Whitewater Reservoir via 2 sluice gates (Scenario 2b)

	Colby Lake			**Whitewater Reservoir**		
	Zero-demand	**Projected Future Conditions**		**Zero-demand**	**Projected Future Conditions**	
Make-up water demand (gpm)	0	3,500	5,000	0	3,500	5,000
Average Elevation[1] (feet)	1,439.39	1,439.36	1,439.32	1,439.29	1,438.78	1,438.05
Maximum Elevation[1] (feet)	1,442.48	1,442.21	1,442.13	1,440.23	1,440.24	1,440.21
Minimum Elevation[1] (feet)	1,438.83	1,438.85	1,438.78	1,437.41	1,435.63	1,432.64
Maximum Fluctuation[1] (feet)	3.65	3.36	3.35	2.82	4.52	7.84
Days Pumping Into Colby Lake[2]	NA	NA	NA	0	854	883
Days Flowing Into Whitewater Reservoir[2]	158	168	179	NA	NA	NA
Percent Time[2] Below 1,439 feet	13.5	11.0	0.5	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

Table 10: 4-year model results comparing water level impacts for various make-up water demands assuming 50-year low flow conditions (approximately 69 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.50 feet above mean sea level for water to be diverted to Whitewater Reservoir via 2 sluice gates (Scenario 2b)

	Colby Lake				**Whitewater Reservoir**			
	Zero-demand	**Projected Future Conditions**			**Zero-demand**	**Projected Future Conditions**		
Make-up water demand (gpm)	0	3,500	5,000	CHD[3]	0	3,500	5,000	CHD[3]
Average Elevation[1] (feet)	1,439.30	1,439.27	1,439.31	1,439.29	1,439.18	1,438.46	1,437.50	1,437.49
Maximum Elevation[1] (feet)	1,441.91	1,441.75	1,441.65	1,441.65	1,440.23	1,440.21	1,440.18	1,440.17
Minimum Elevation[1] (feet)	1,438.79	1,438.82	1,438.65	1,438.67	1,437.36	1,434.31	1,430.29	1,430.41
Maximum Fluctuation[1] (feet)	3.12	2.93	3.00	2.98	2.83	5.86	9.87	9.74
Days Pumping Into Colby Lake[2]	NA	NA	NA	NA	0	943	964	962
Days Flowing Into Whitewater Reservoir[2]	154	161	179	181	NA	NA	NA	NA
Percent Time[2] Below 1,439 feet	38.5	31.0	3.5	12.5	NA	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

[3] CHD = Combined high demand (8,000 gallons per minute during three months of the year and 4,400 gallons per minute during the other nine months of year).

Figures

Figure 1: Location of Study Area

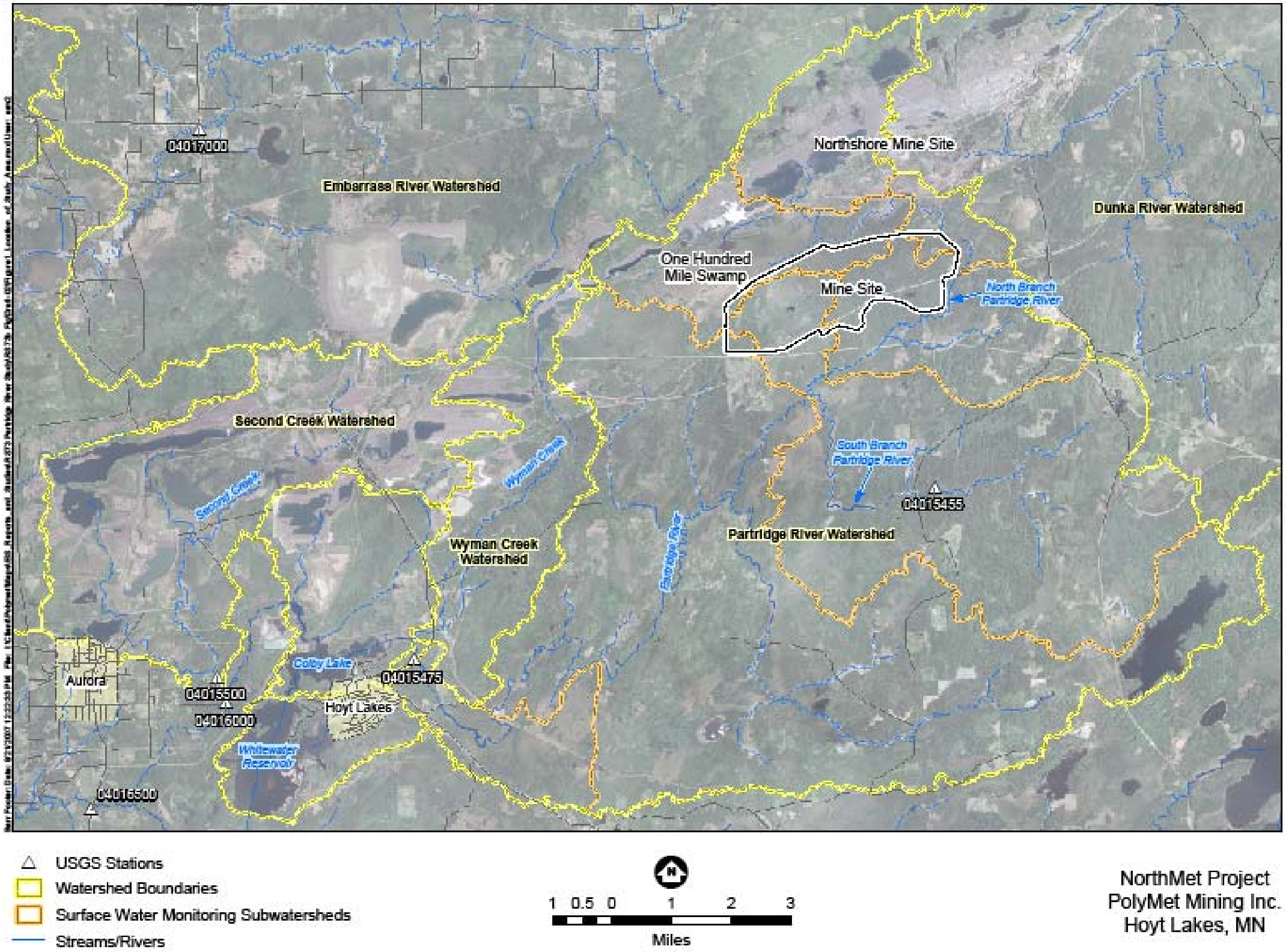


Figure 2: Layout of Mine Facilities in Year 1 and Year 20

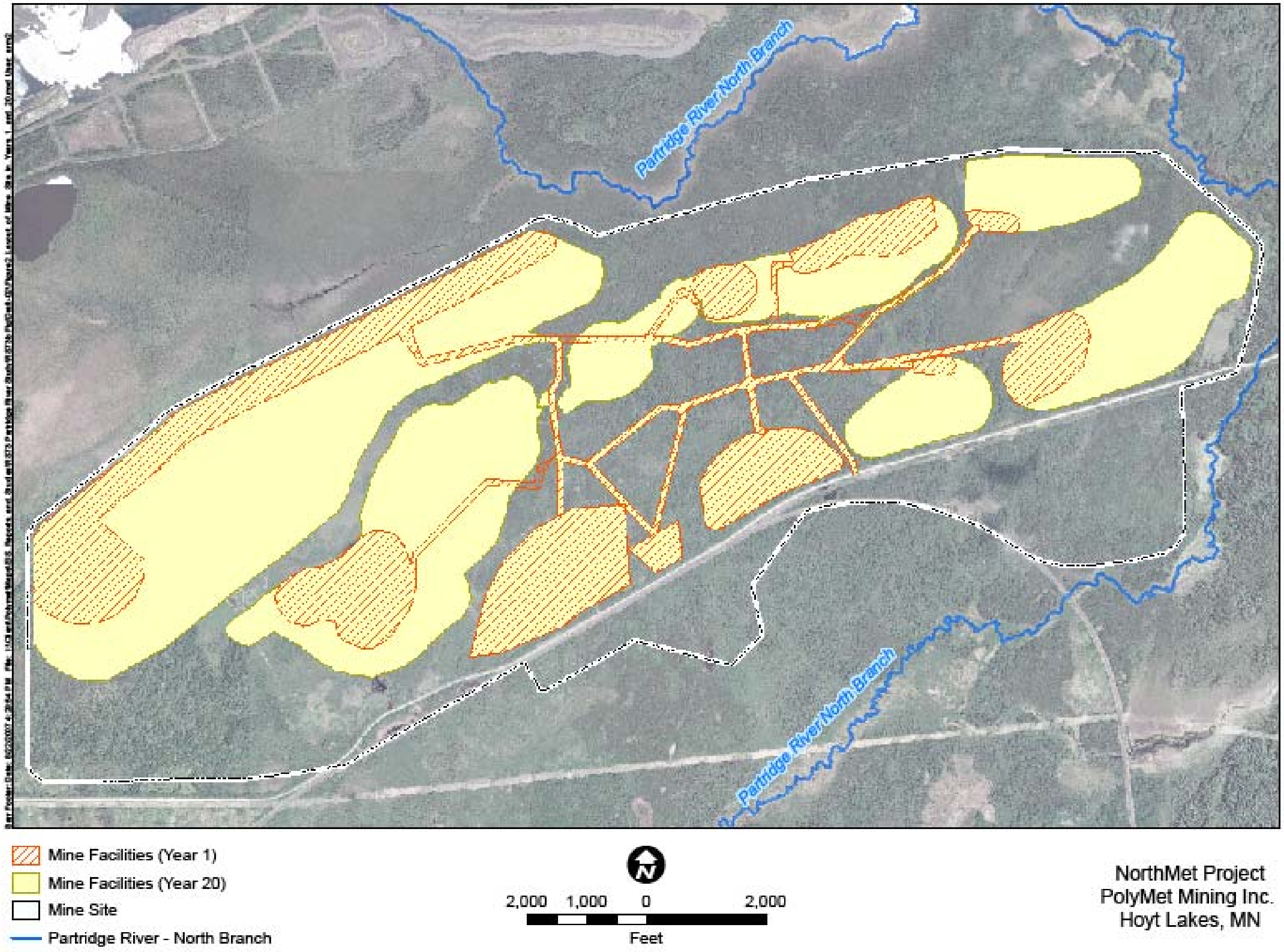
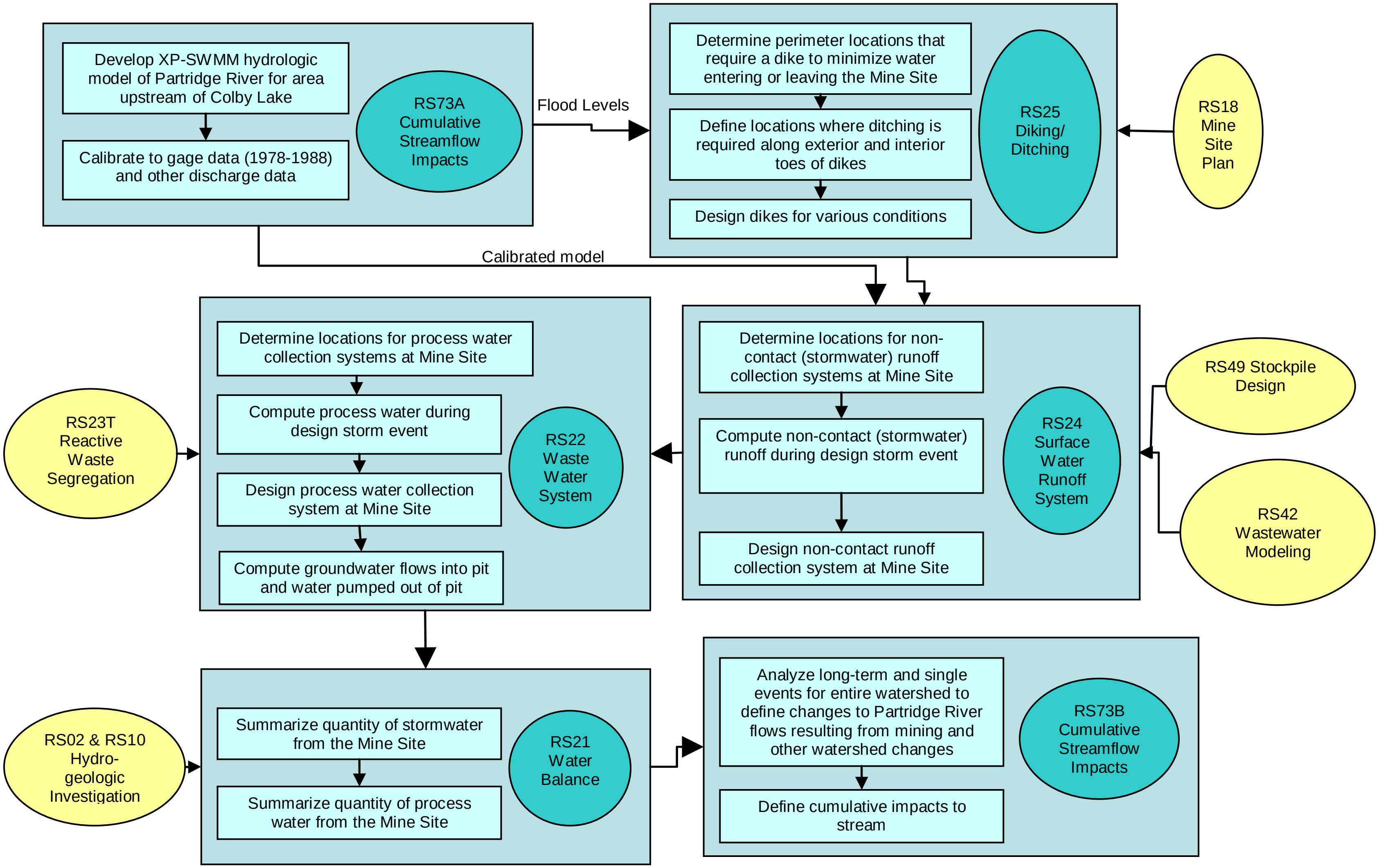


Figure 3: Mine Site Water Management



NOTES: This flow chart provides a general idea of the various tasks. Predecessor tasks ○ are only listed at the first occurrence. Closure and reclamation will be evaluated in RS52.

Figure 4: Schematic of water balance components in XP-SWMM

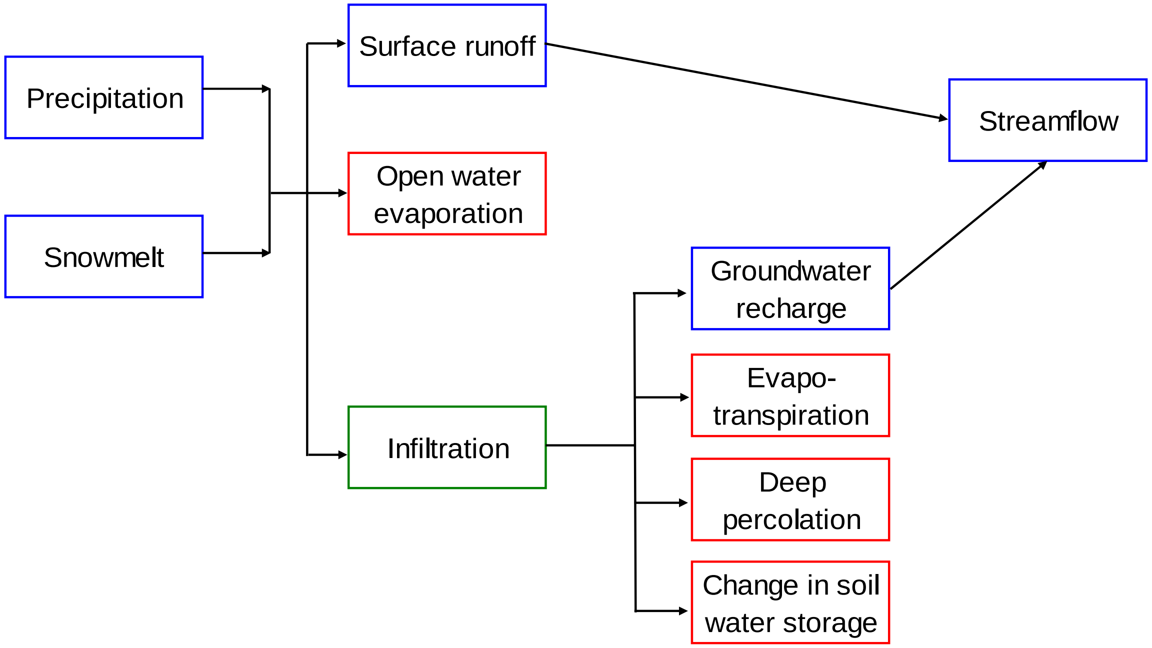


Figure 5: Flow records at USGS gaging stations in Partridge River watershed

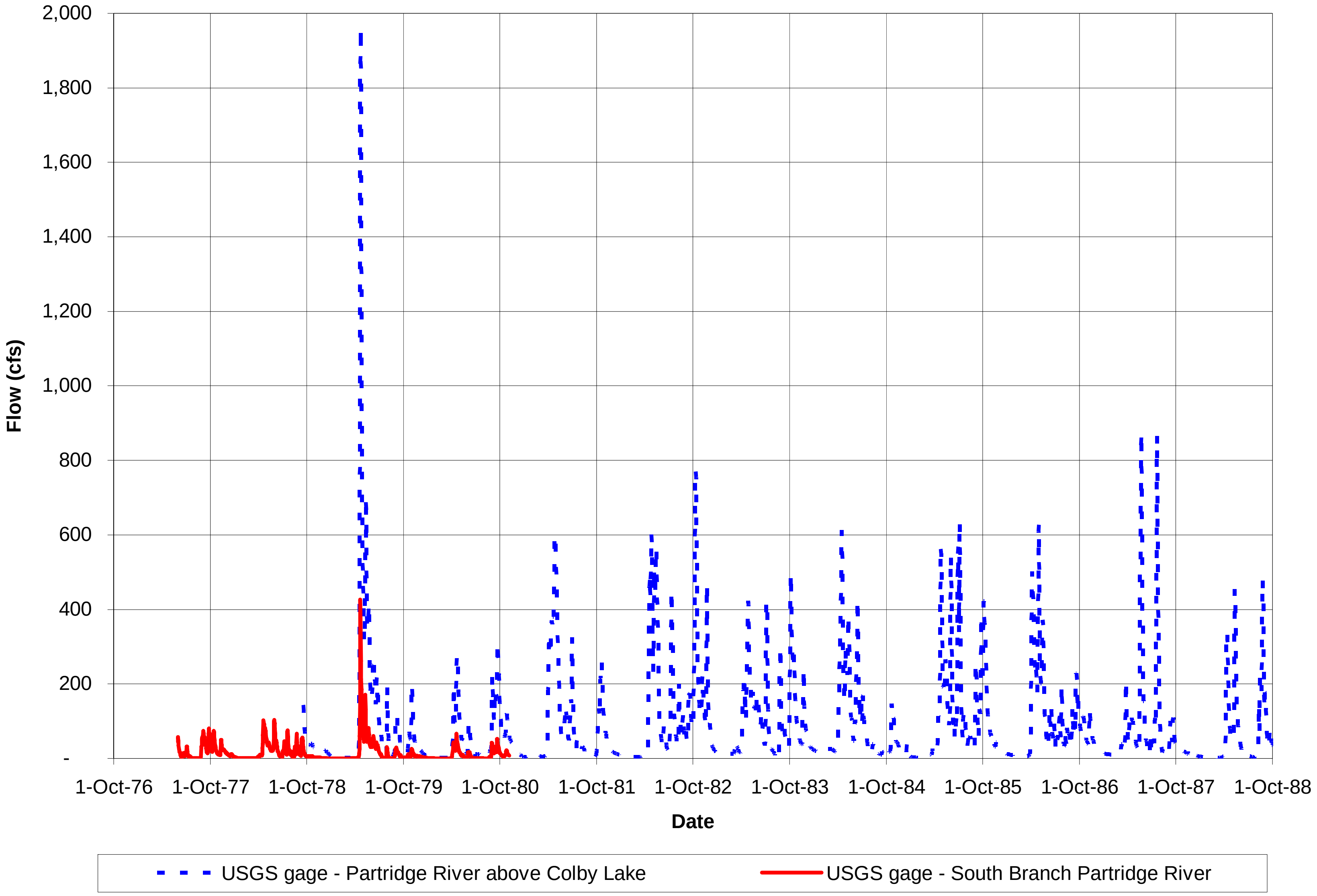


Figure 6: Location of surface water monitoring stations

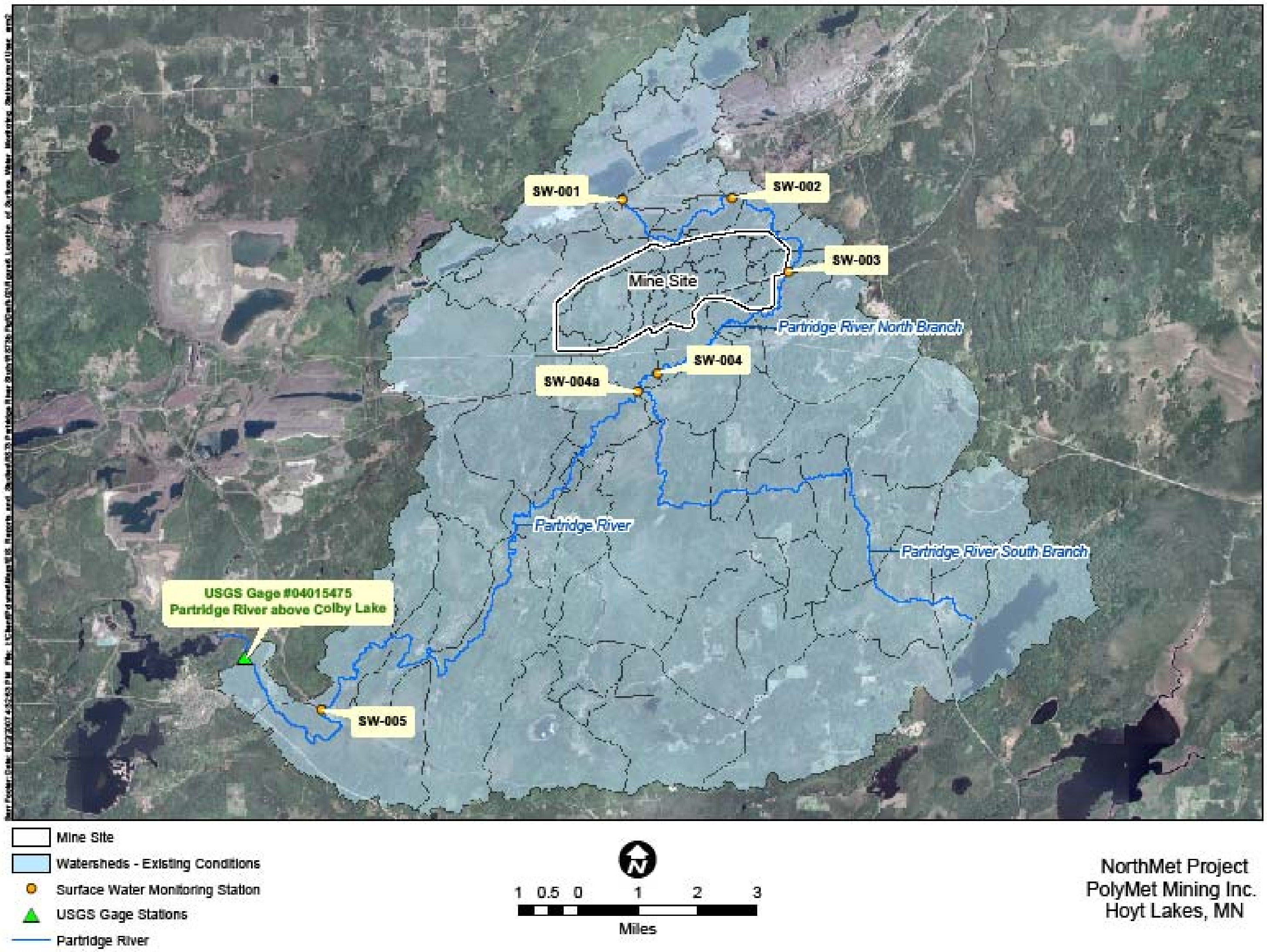


Figure 7: Approximate floodplain of the Partridge River in the vicinity of the Mine Site

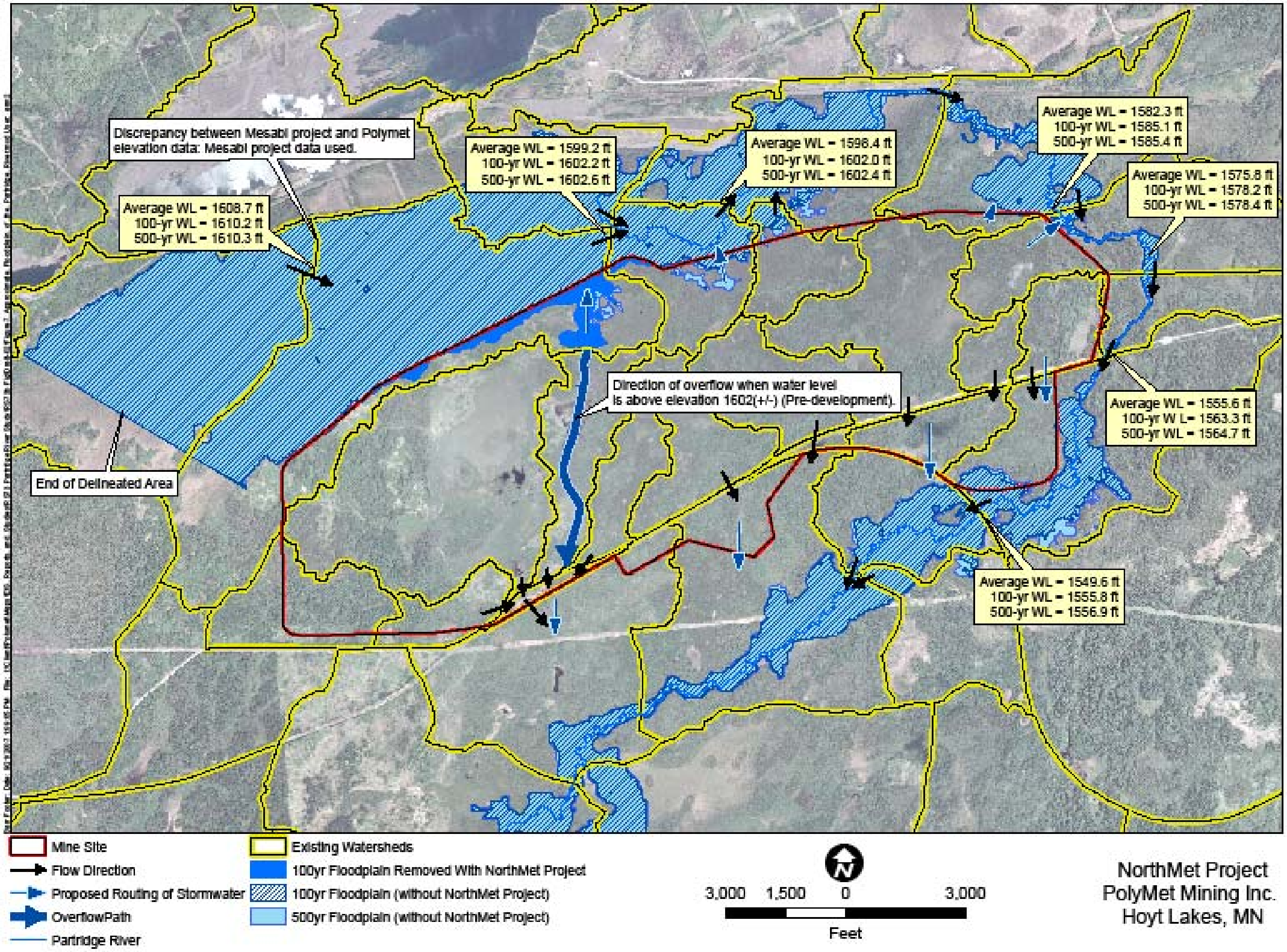


Figure 8: Sub-watershed boundaries near Mine Site defined in XP-SWMM under Existing Conditions scenario

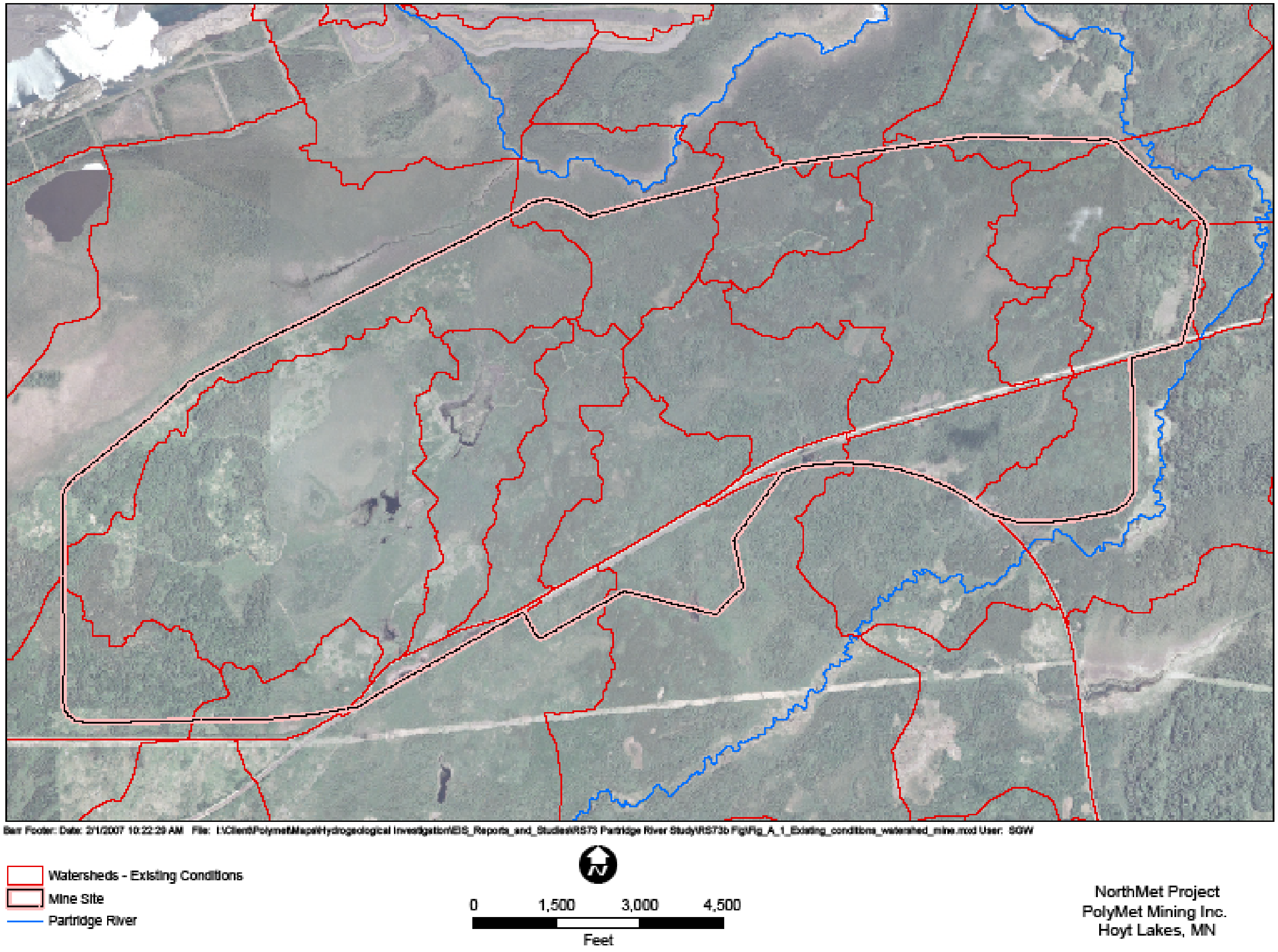


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Figure 9: Sub-watershed boundaries near Mine Site defined in XP-SWMM for Year 1 scenario

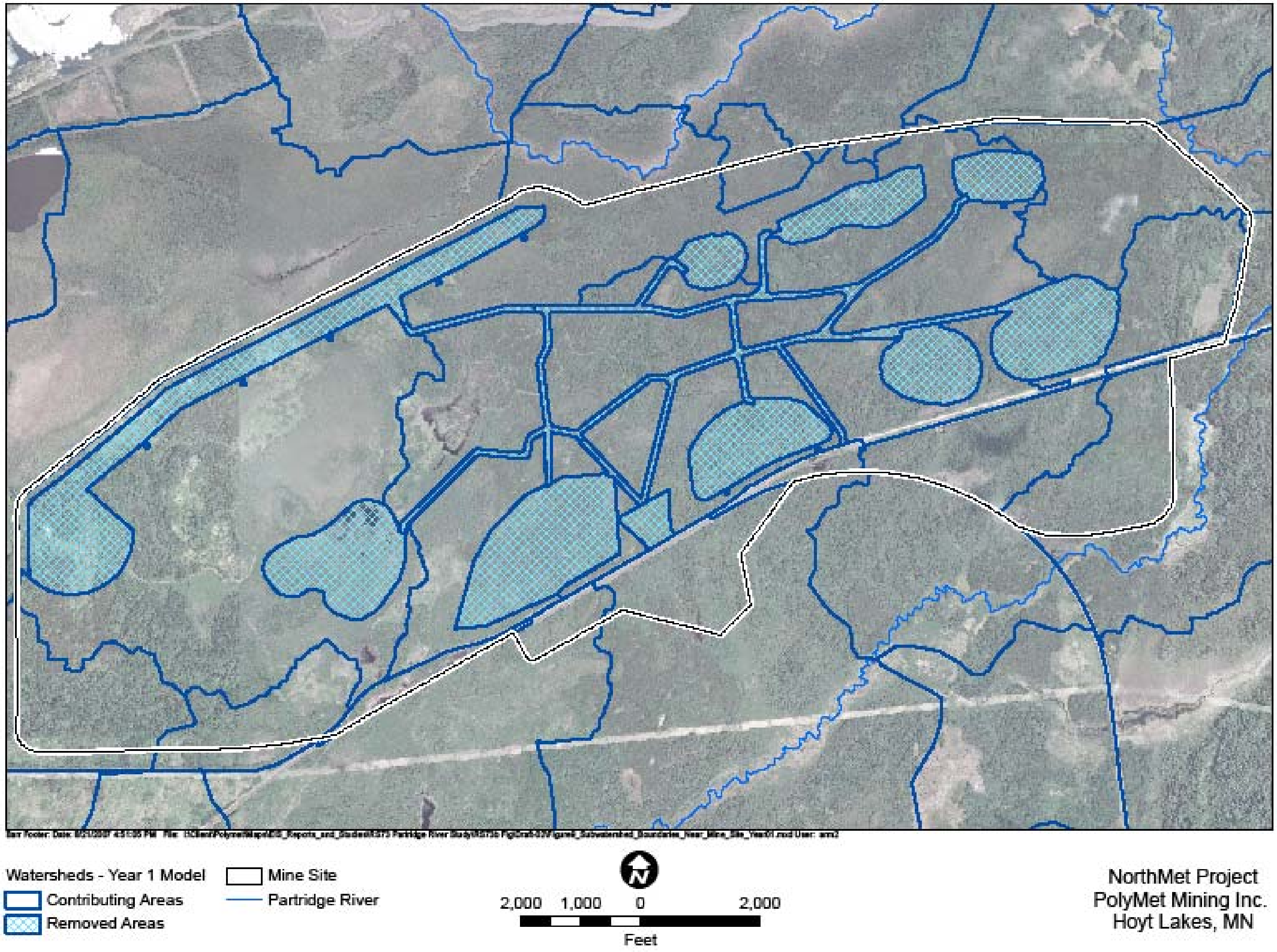


Figure 10: Sub-watershed boundaries near Mine Site defined in XP-SWMM for Year 5 scenario

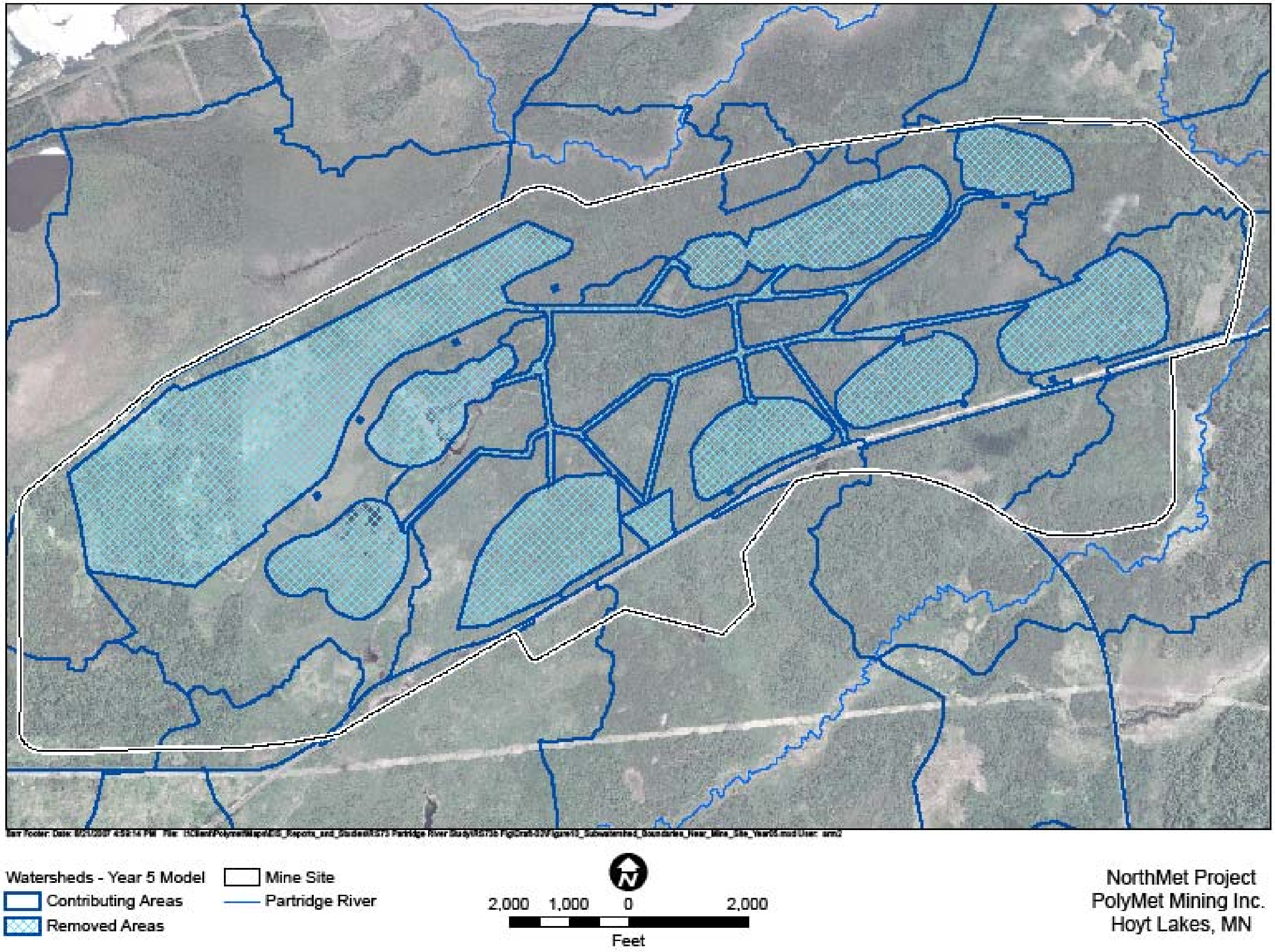


Figure 11: Sub-watershed boundaries near Mine Site defined in XP-SWMM for Year 10 scenario

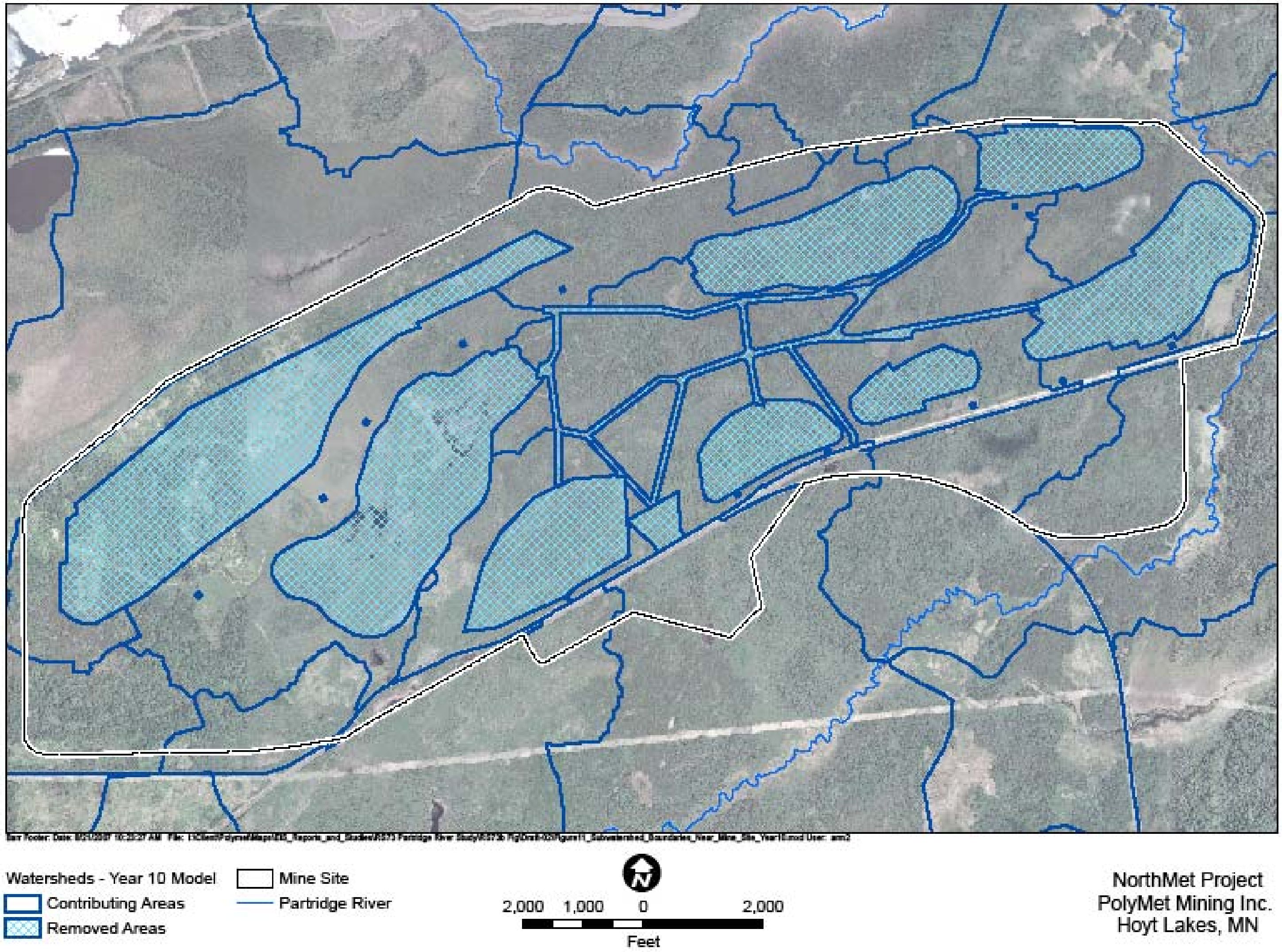


Figure 12: Sub-watershed boundaries near Mine Site defined in XP-SWMM for Year 15 scenario

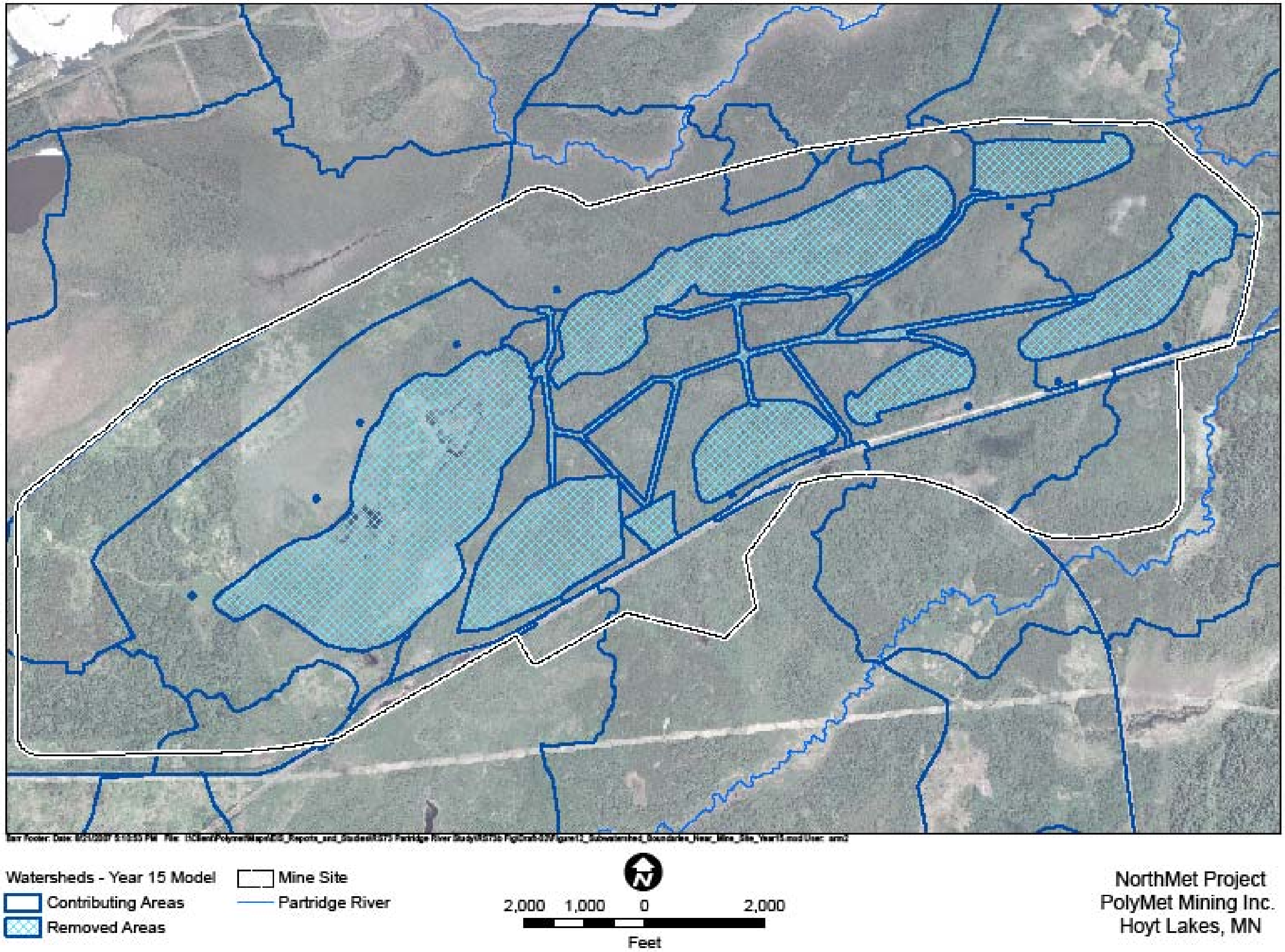


Figure 13: Sub-watershed boundaries near Mine Site defined in XP-SWMM for Year 20 scenario

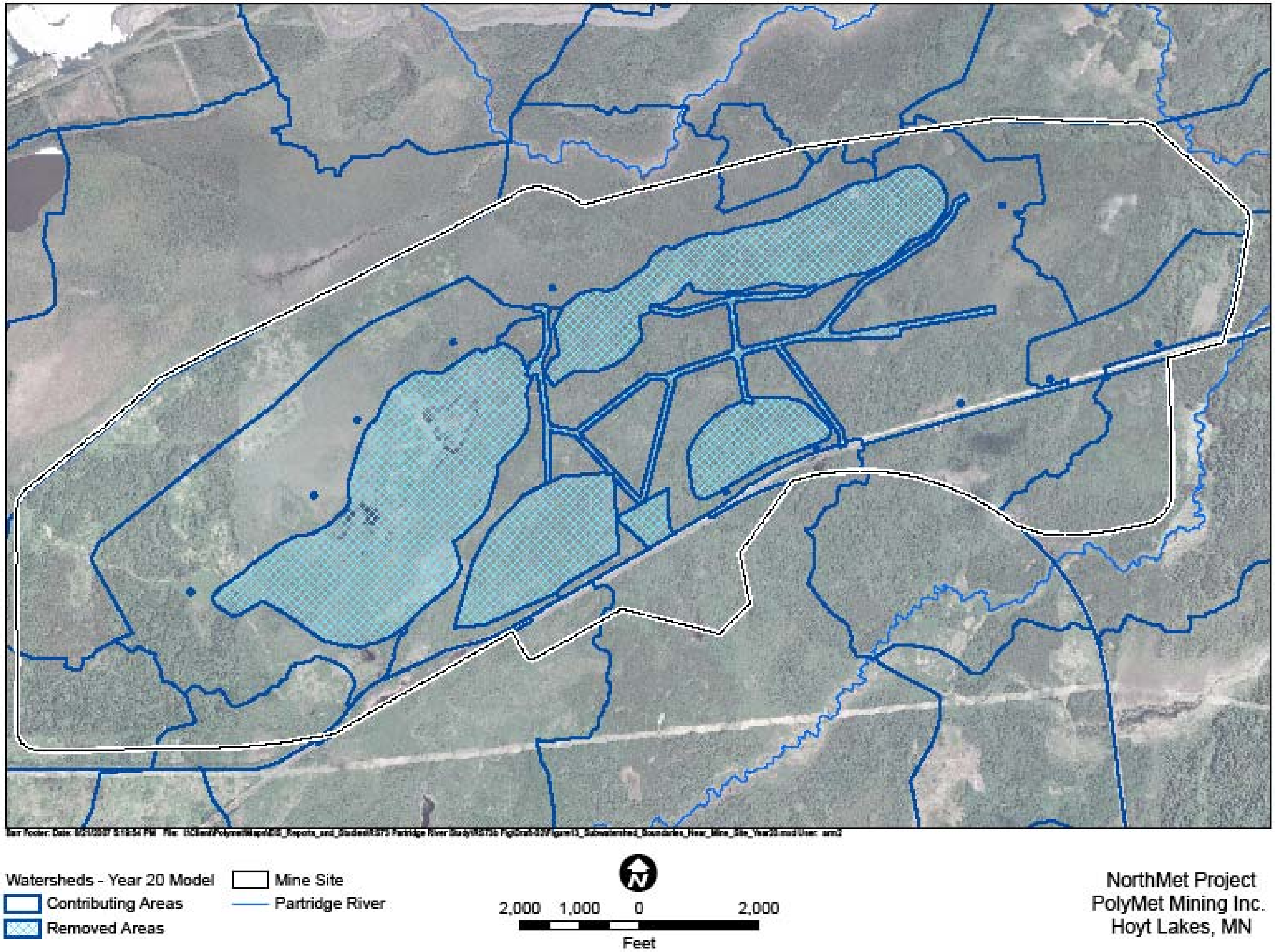


Figure 14: Sub-watershed boundaries near Mine Site defined in XP-SWMM for Mine Facilities Off scenario

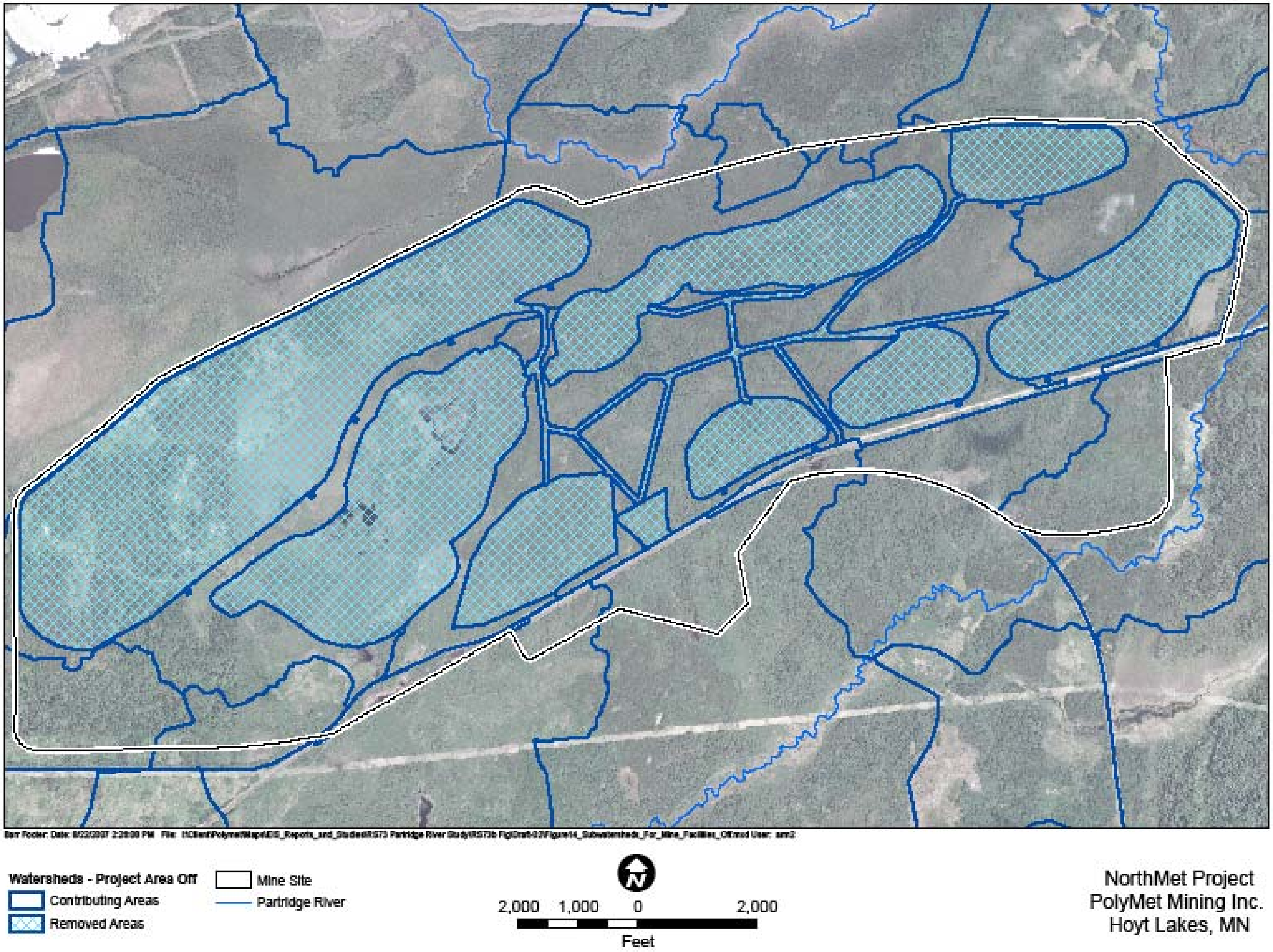


Figure 15: Flow statistics at surface water monitoring station SW-002 for projected development stages of Mine Site

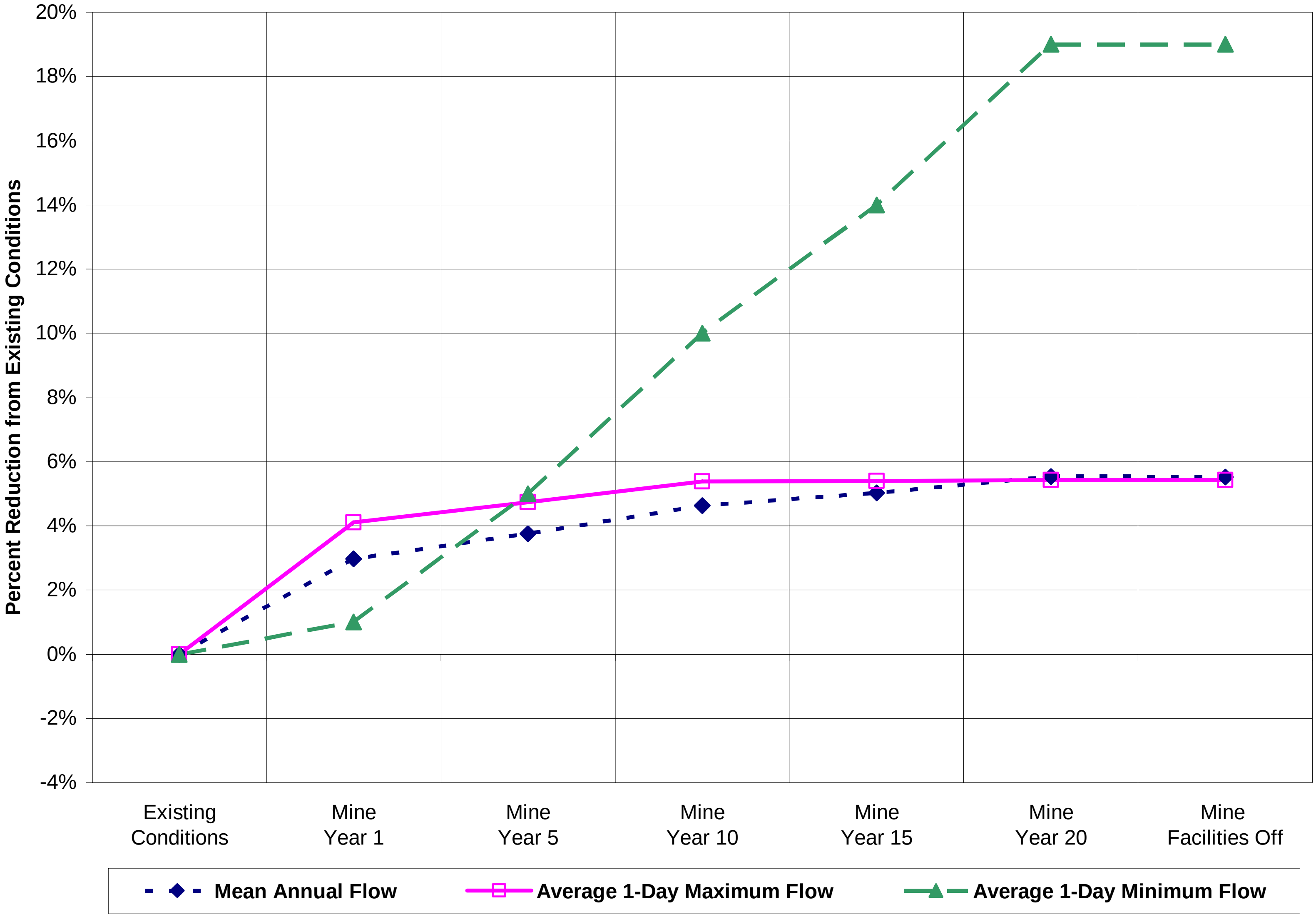


Figure 16: Flow statistics at surface water monitoring station SW-003 for projected development stages of Mine Site

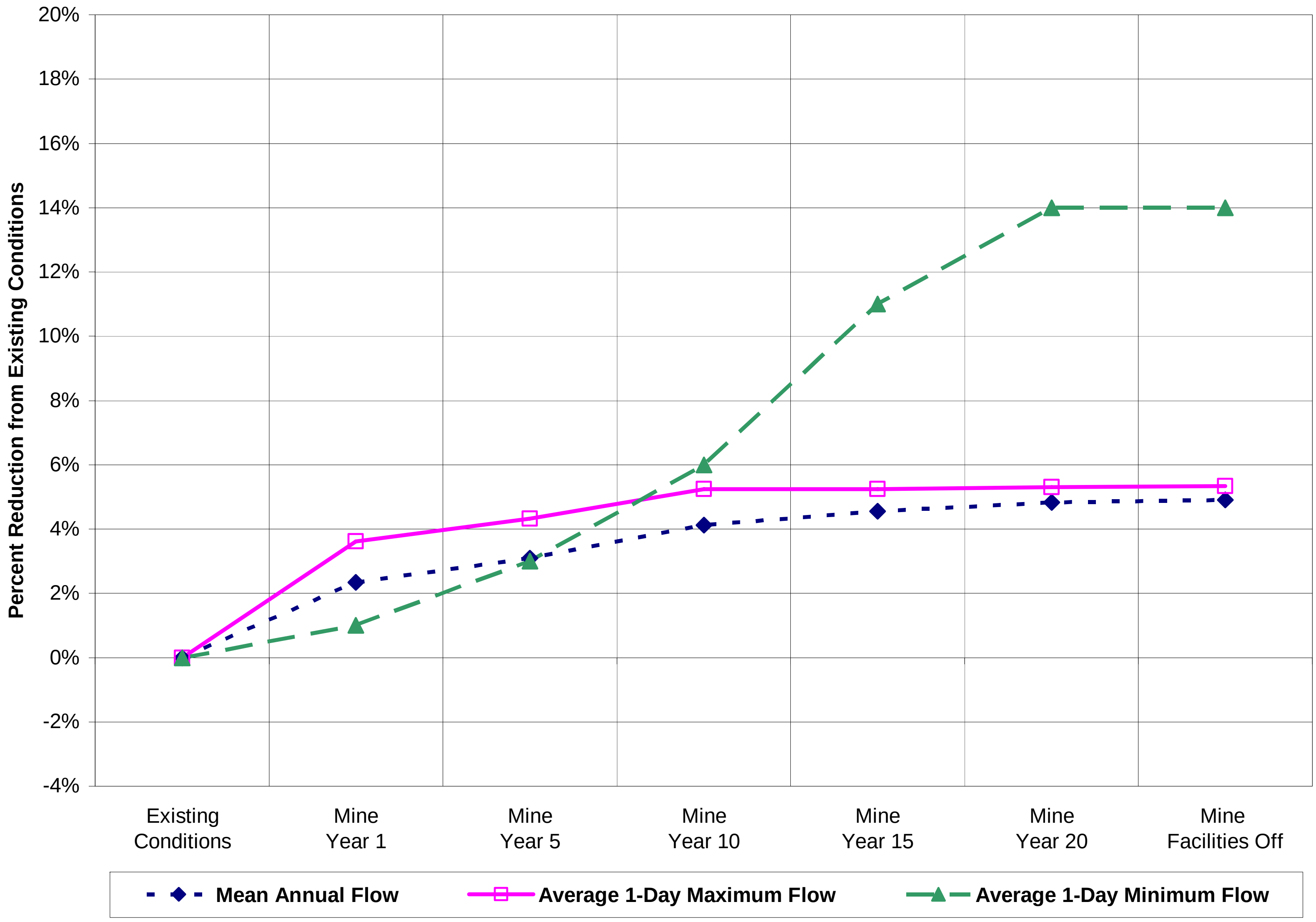


Figure 17: Flow statistics at surface water monitoring station SW-004 for projected development stages of Mine Site

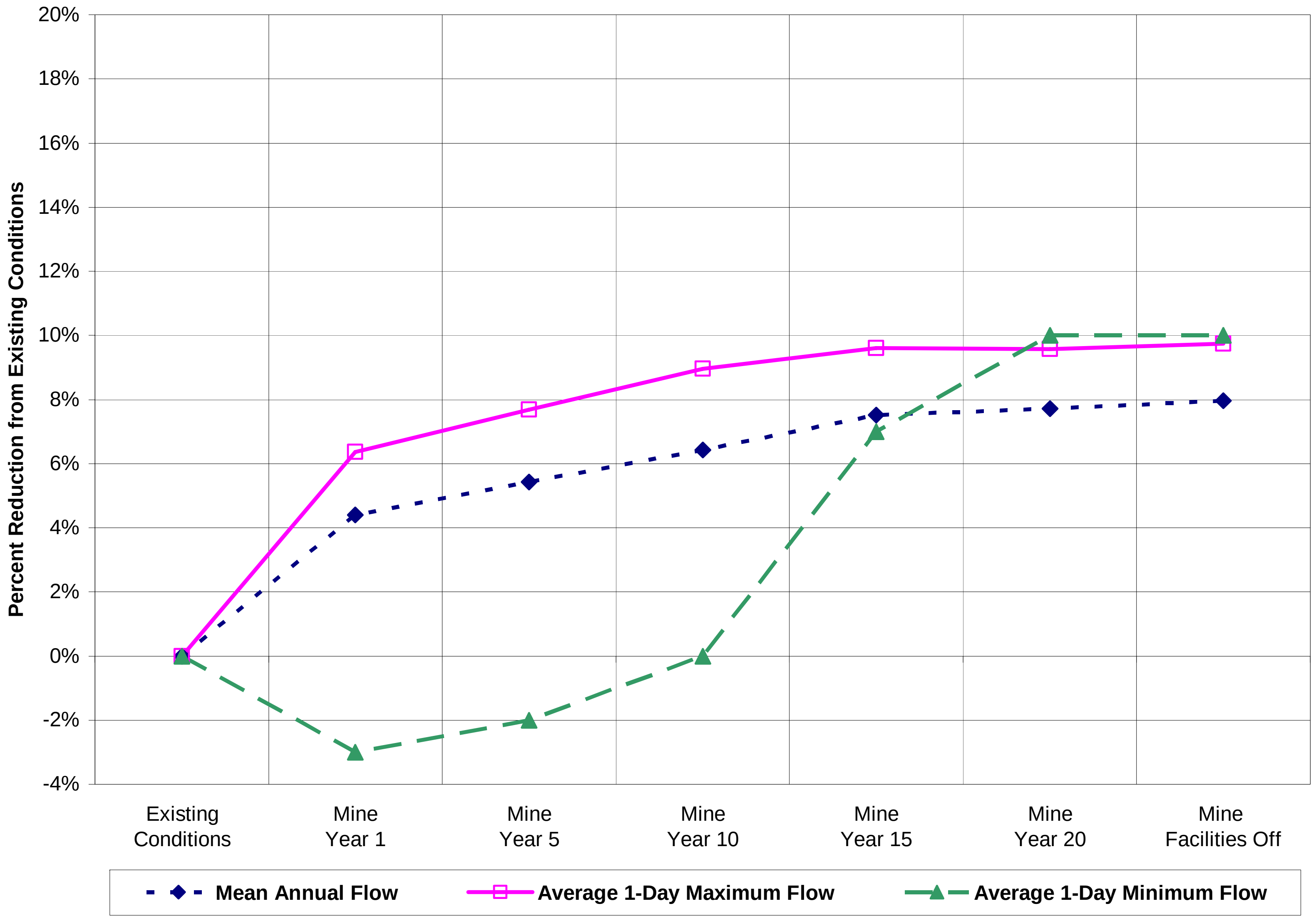


Figure 18: Flow statistics at surface water monitoring station SW-004a for projected development stages of Mine Site

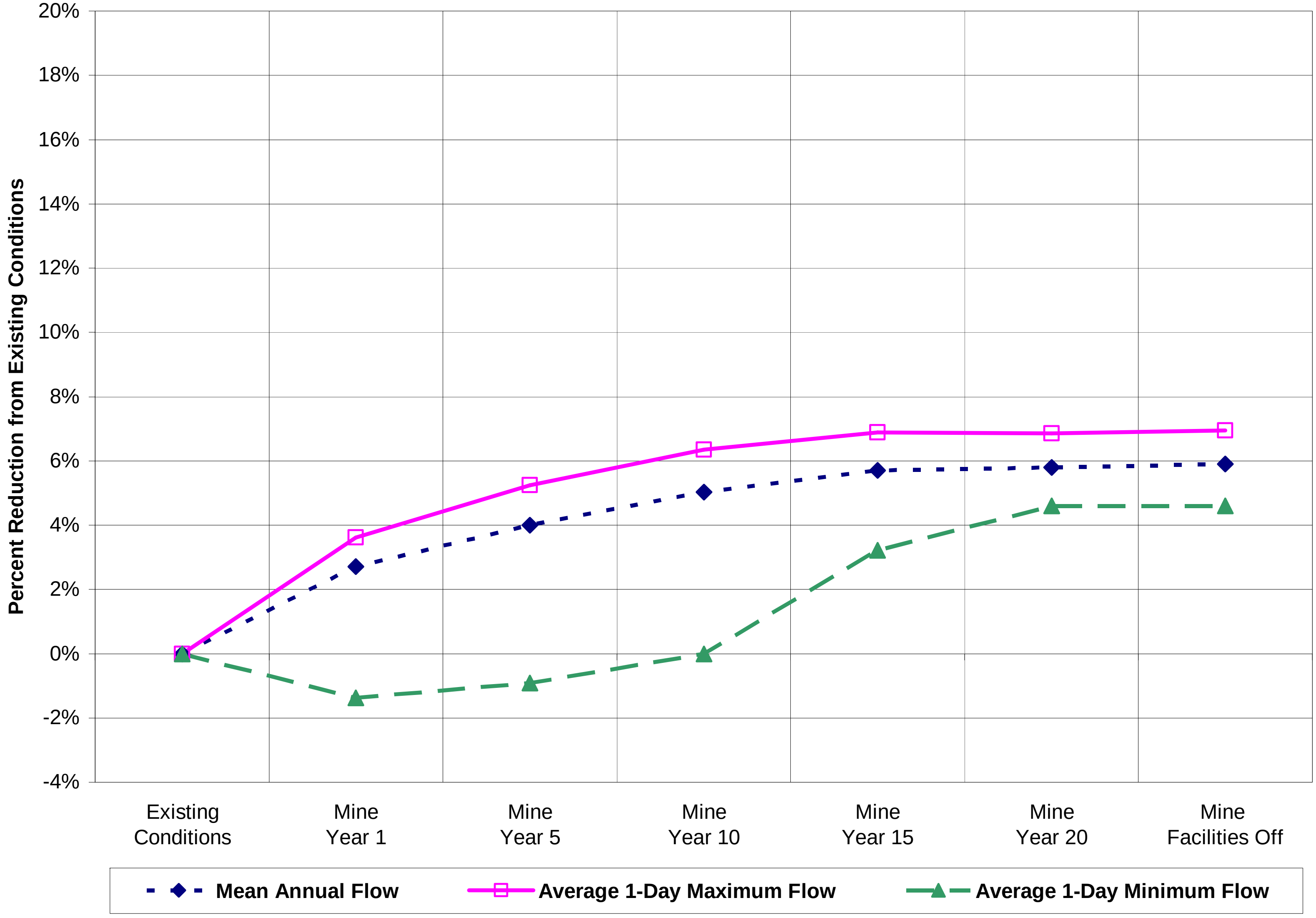


Figure 19: Flow statistics at surface water monitoring station SW-005 for projected development stages of Mine Site

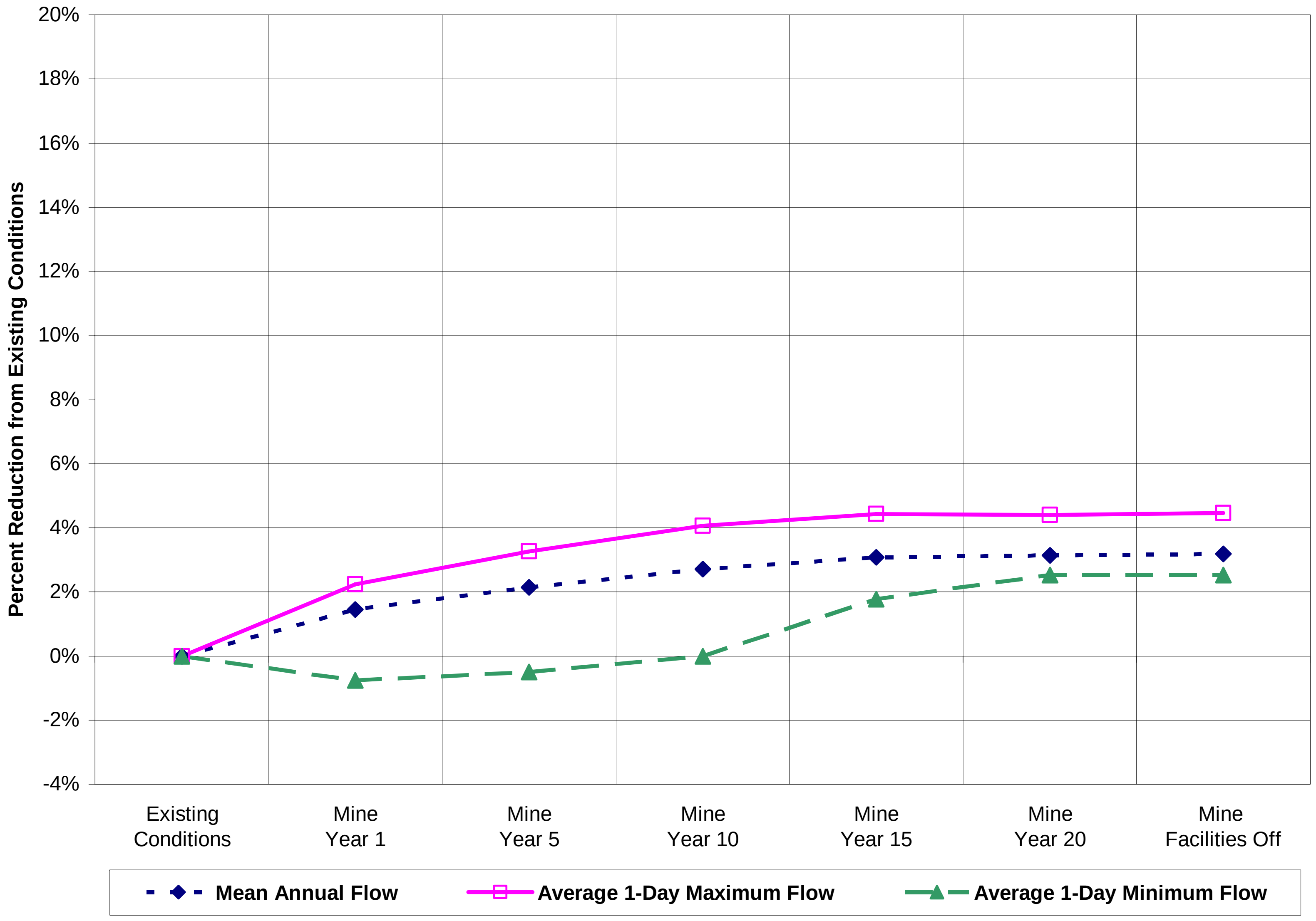


Figure 20: Flow statistics at USGS gage (Partridge River above Colby Lake) for projected development stages of Mine Site

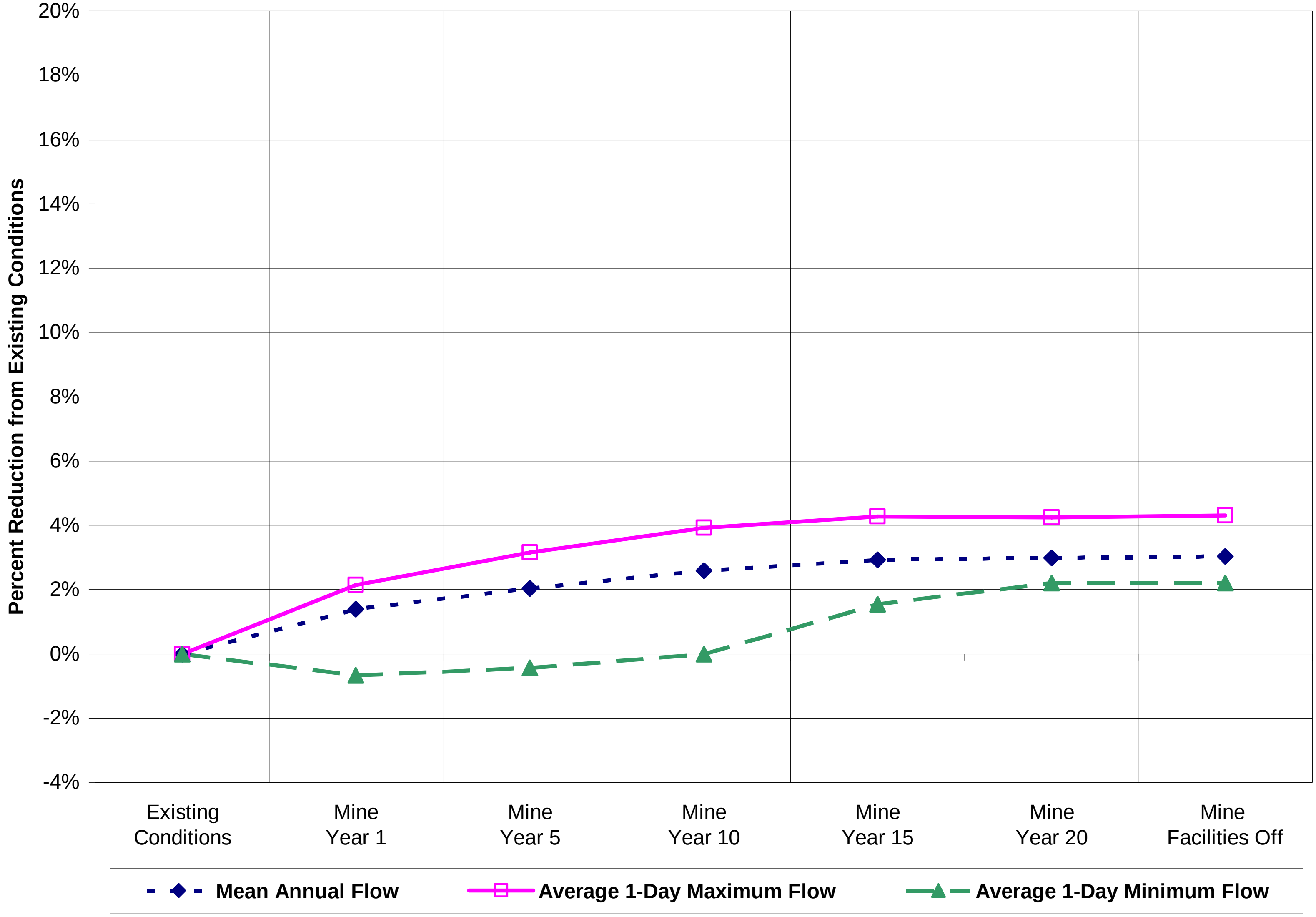


Figure 21: Schematic of Colby Lake-Whitewater Reservoir hydrologic system

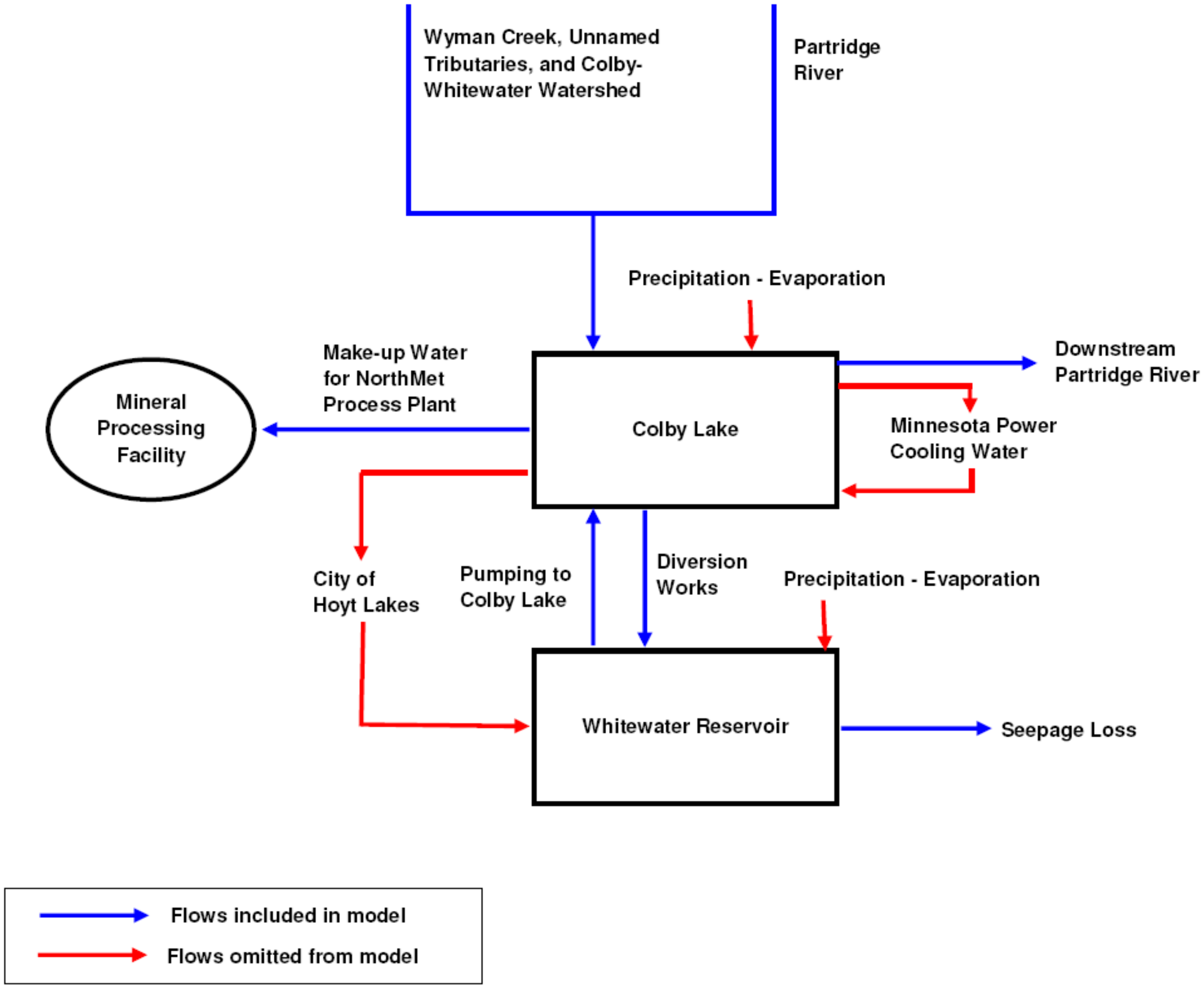


Figure 22: Water levels in Colby Lake for period “Before mining”

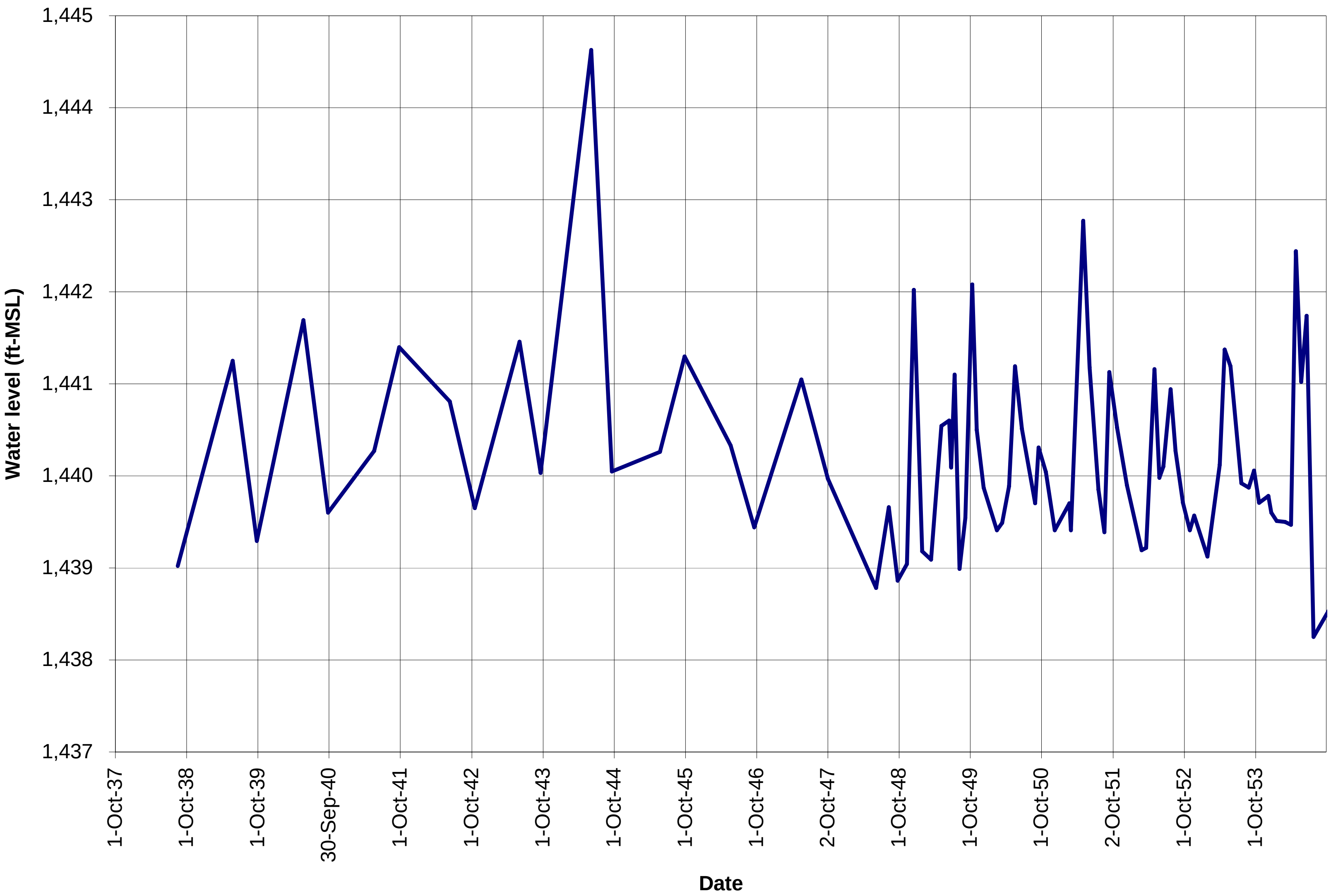


Figure 23: Water levels in Colby Lake for period “During mining”

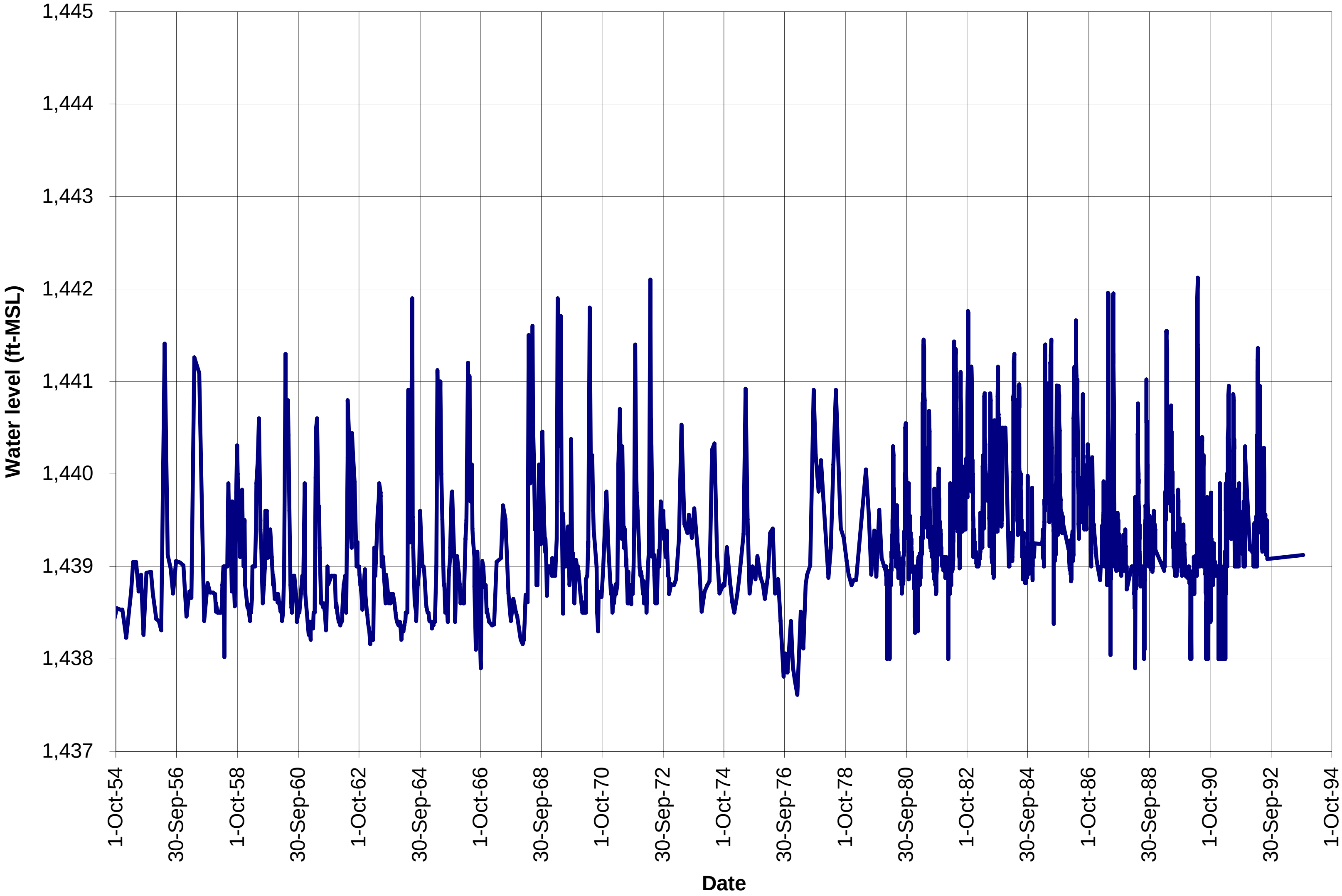


Figure 24: Water levels in Colby Lake for period “After mining”

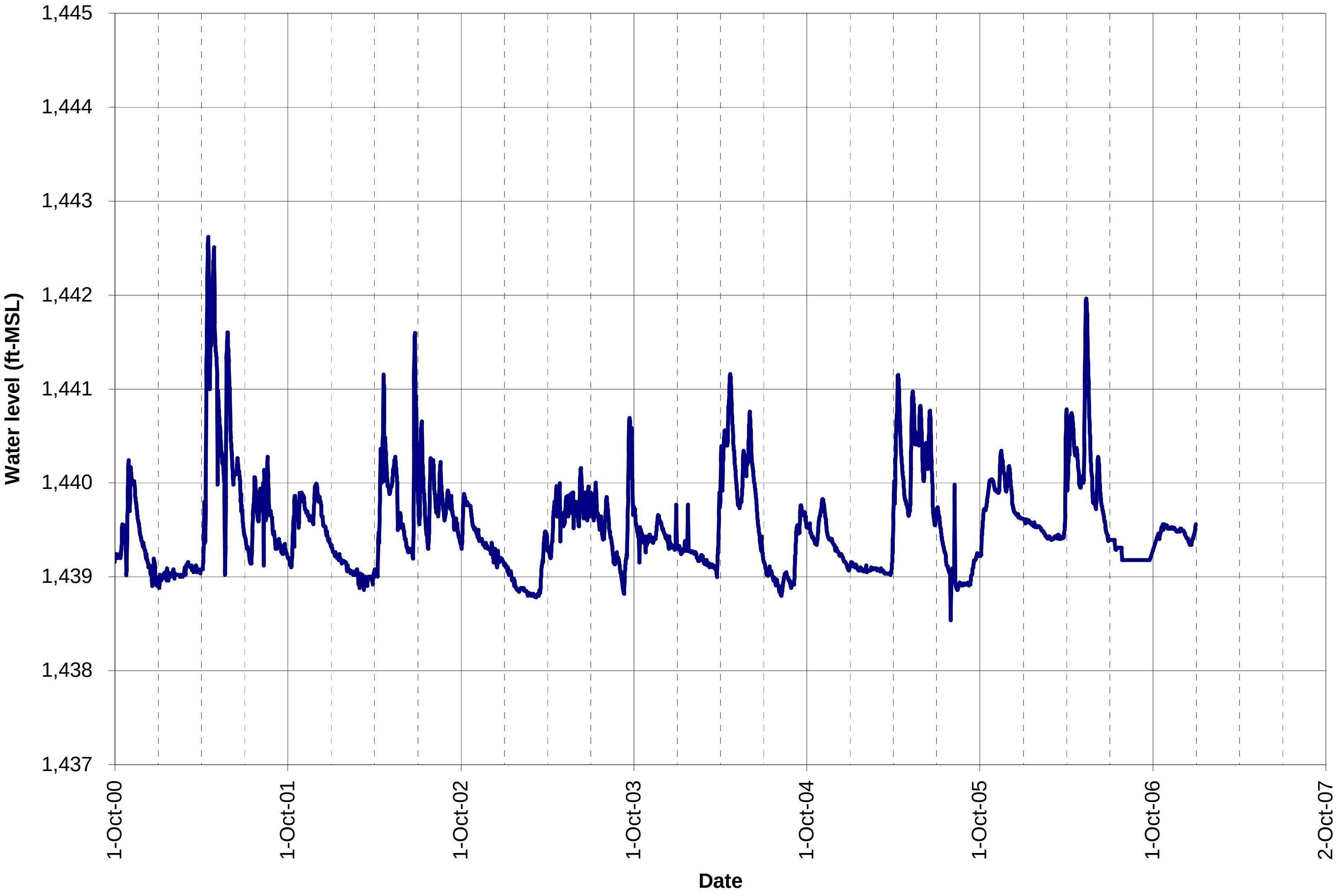


Figure 25: Water levels in Whitewater Reservoir for period “Before mining”

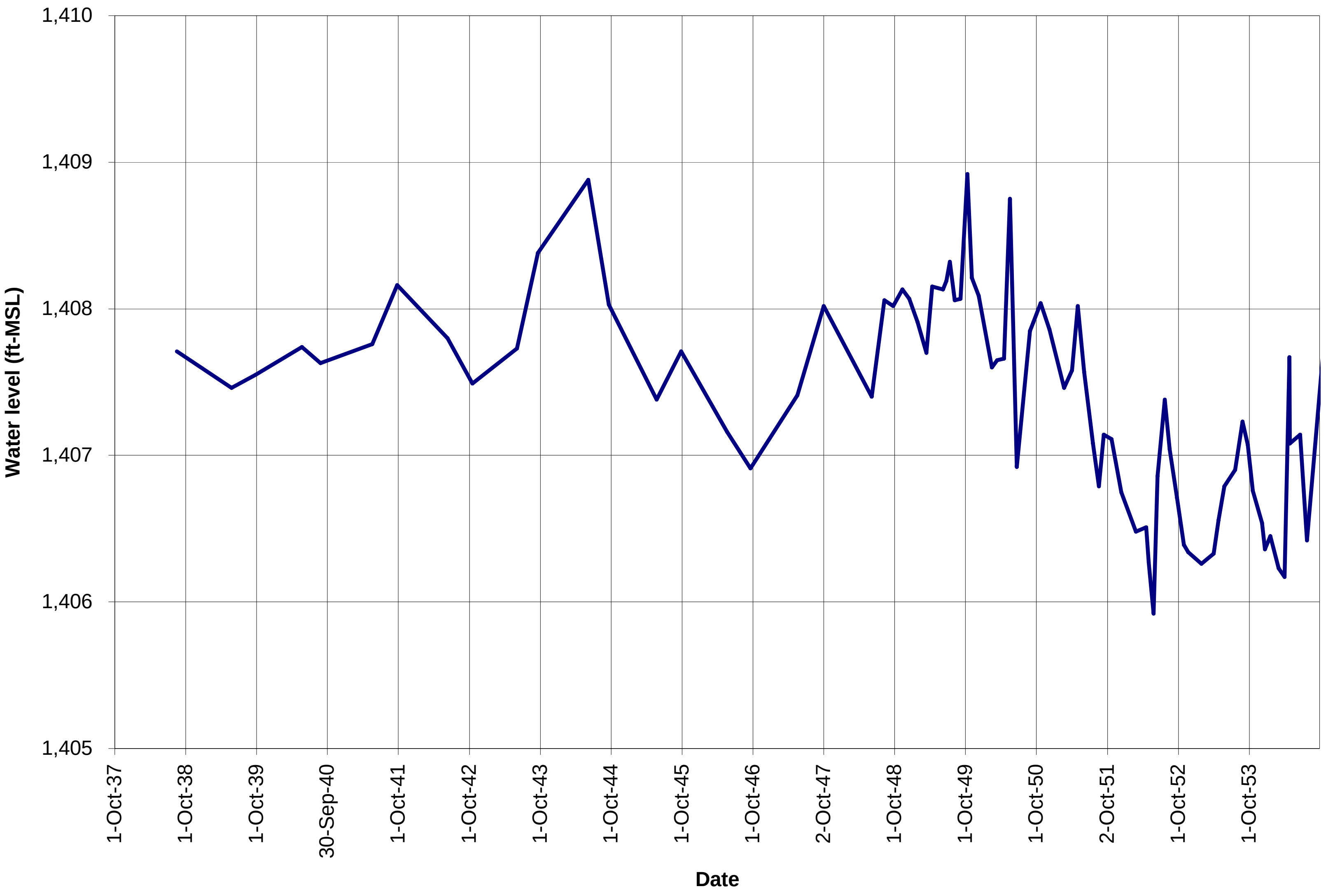


Figure 26: Water levels in Whitewater Reservoir for period “During mining”

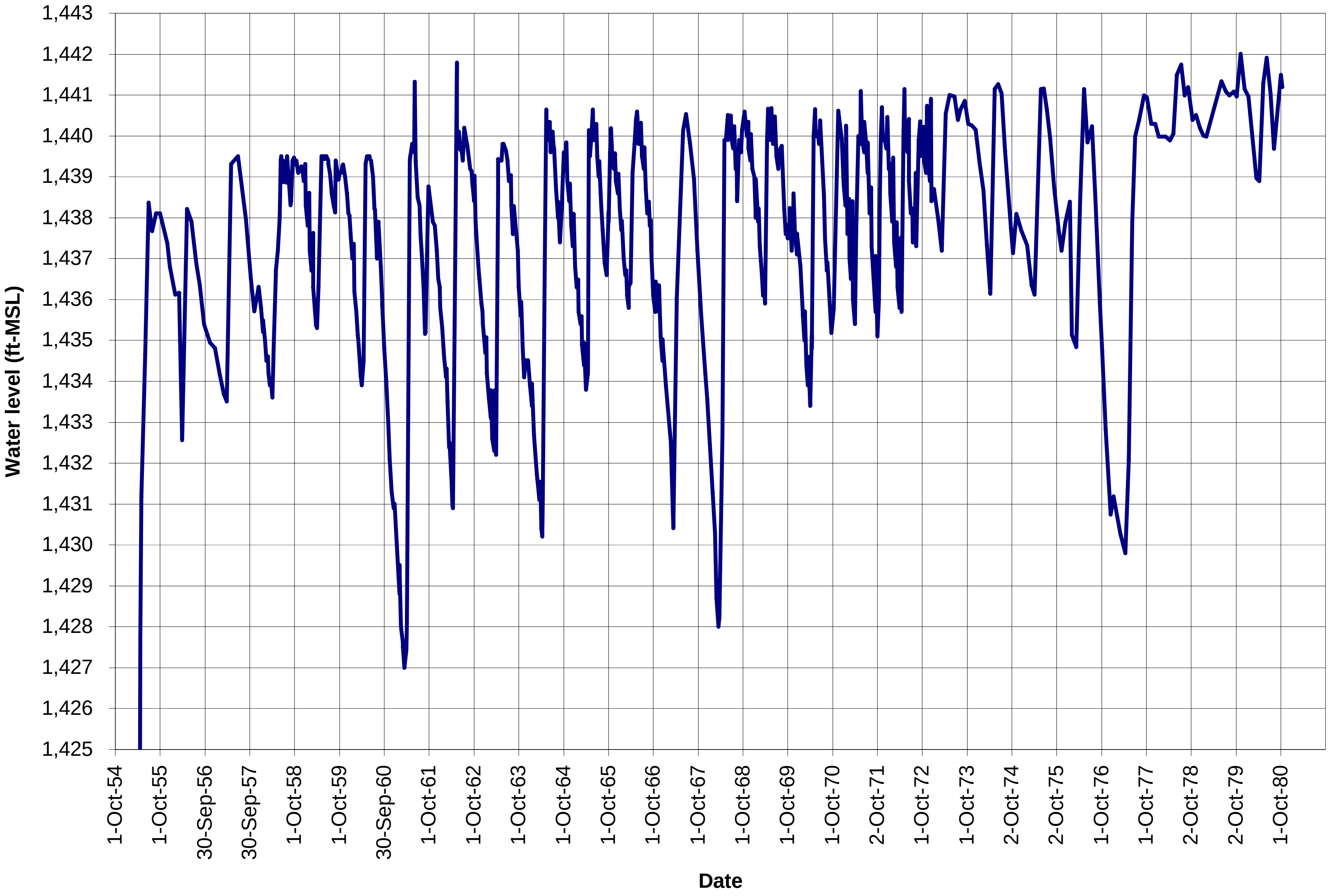


Figure 27: Water levels in Whitewater Reservoir for period “After mining”

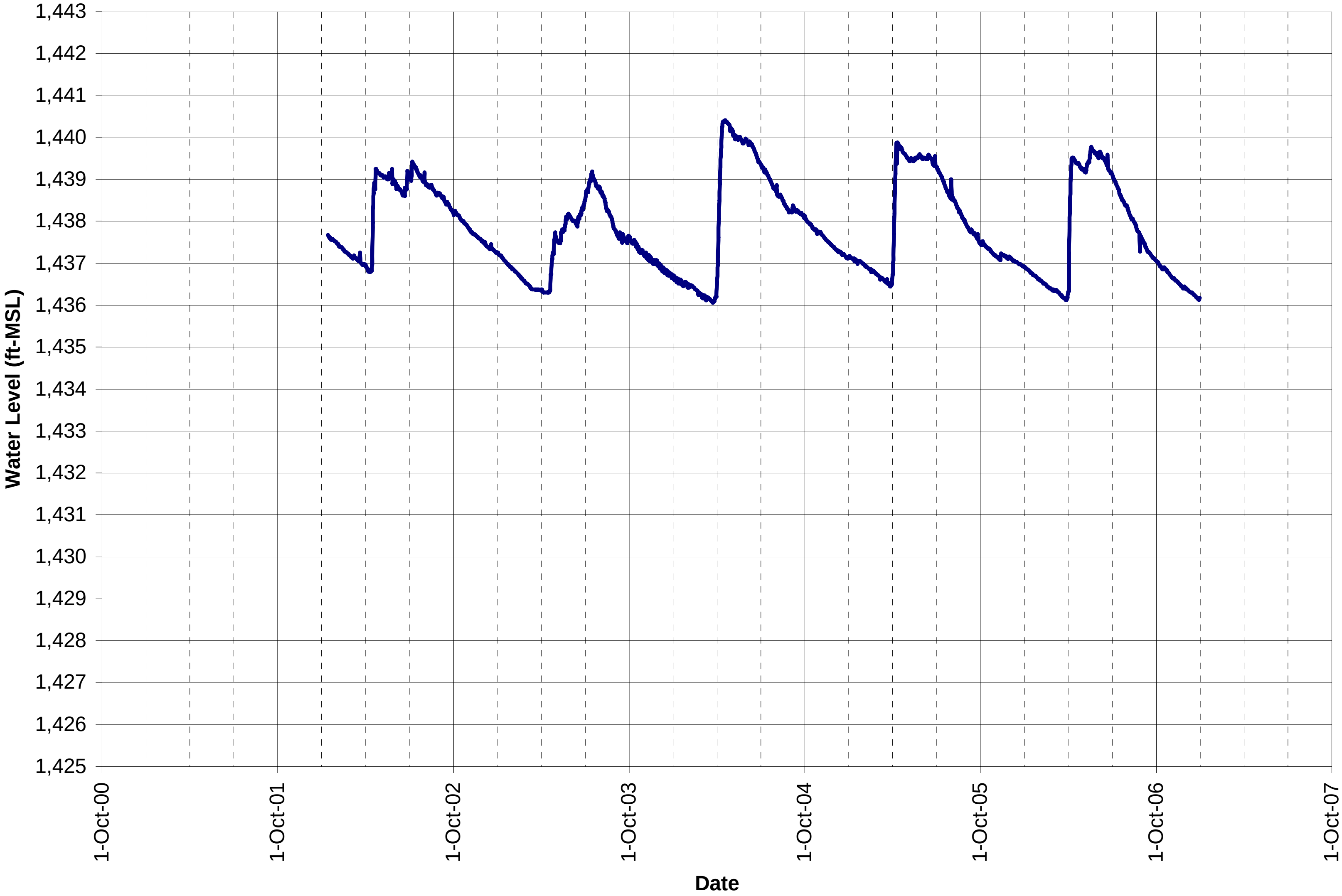


Figure 28: Estimated seepage loss from Whitewater Reservoir as a function of water elevation (Barr, 1964)

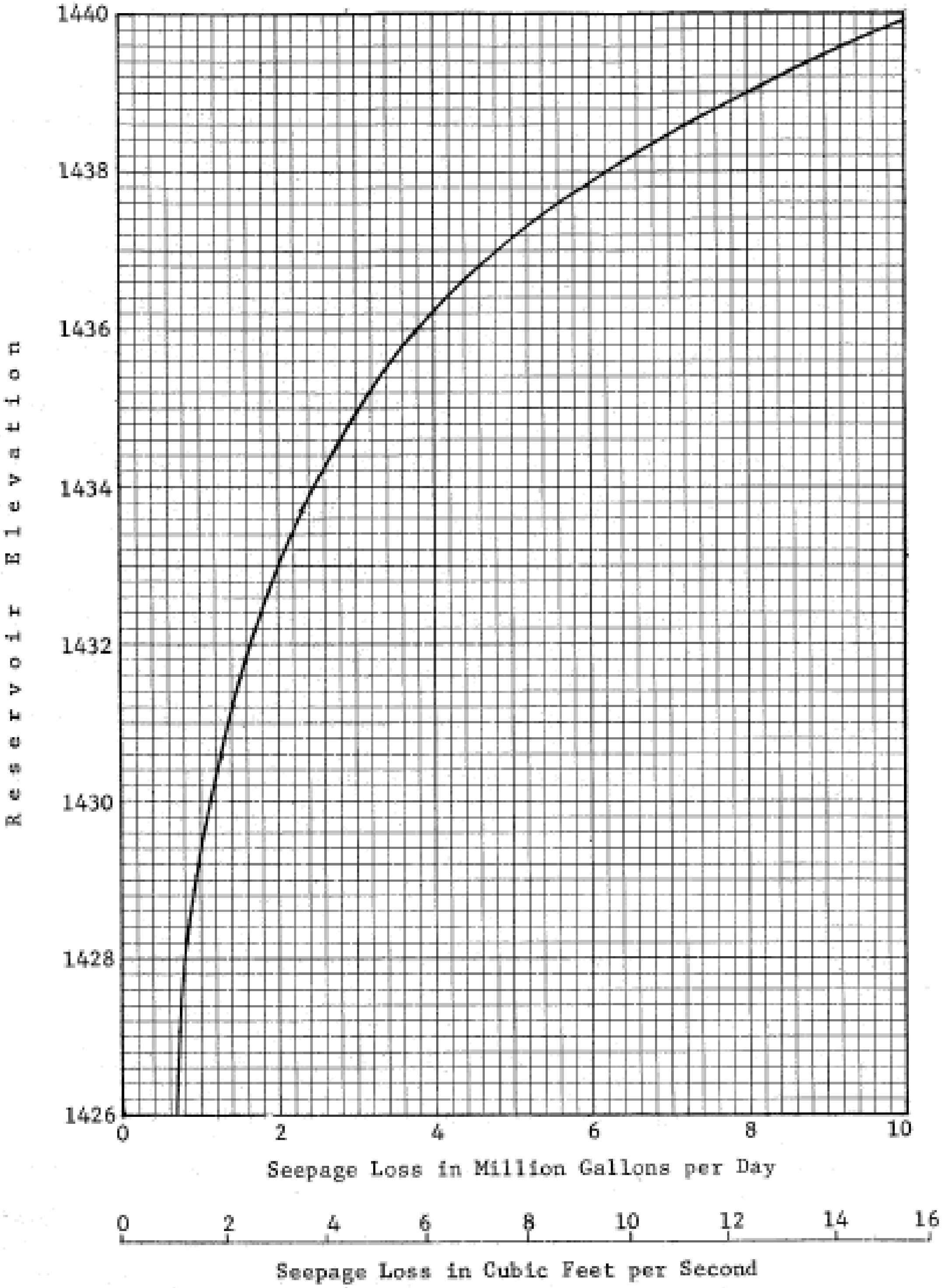


Figure 29: Modeled minimum water levels in Colby Lake and Whitewater Reservoir, based on 4-year water balance calculations under average flow conditions and nine different operational scenarios of Diversion Works

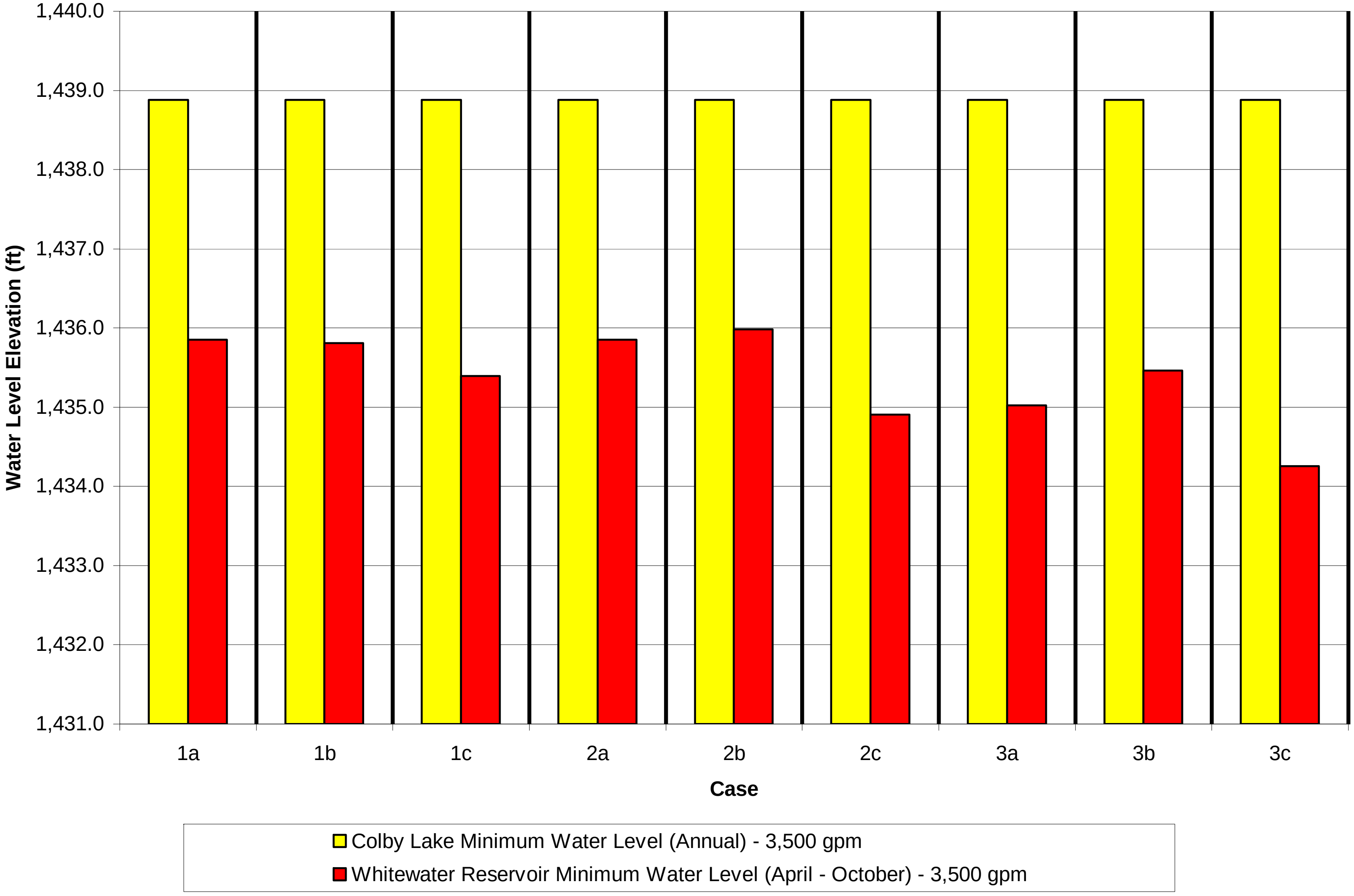


Figure 30: Modeled maximum water level fluctuations in Colby Lake and Whitewater Reservoir, based on 4-year water balance calculations under average flow conditions and nine different operational scenarios of Diversion Works

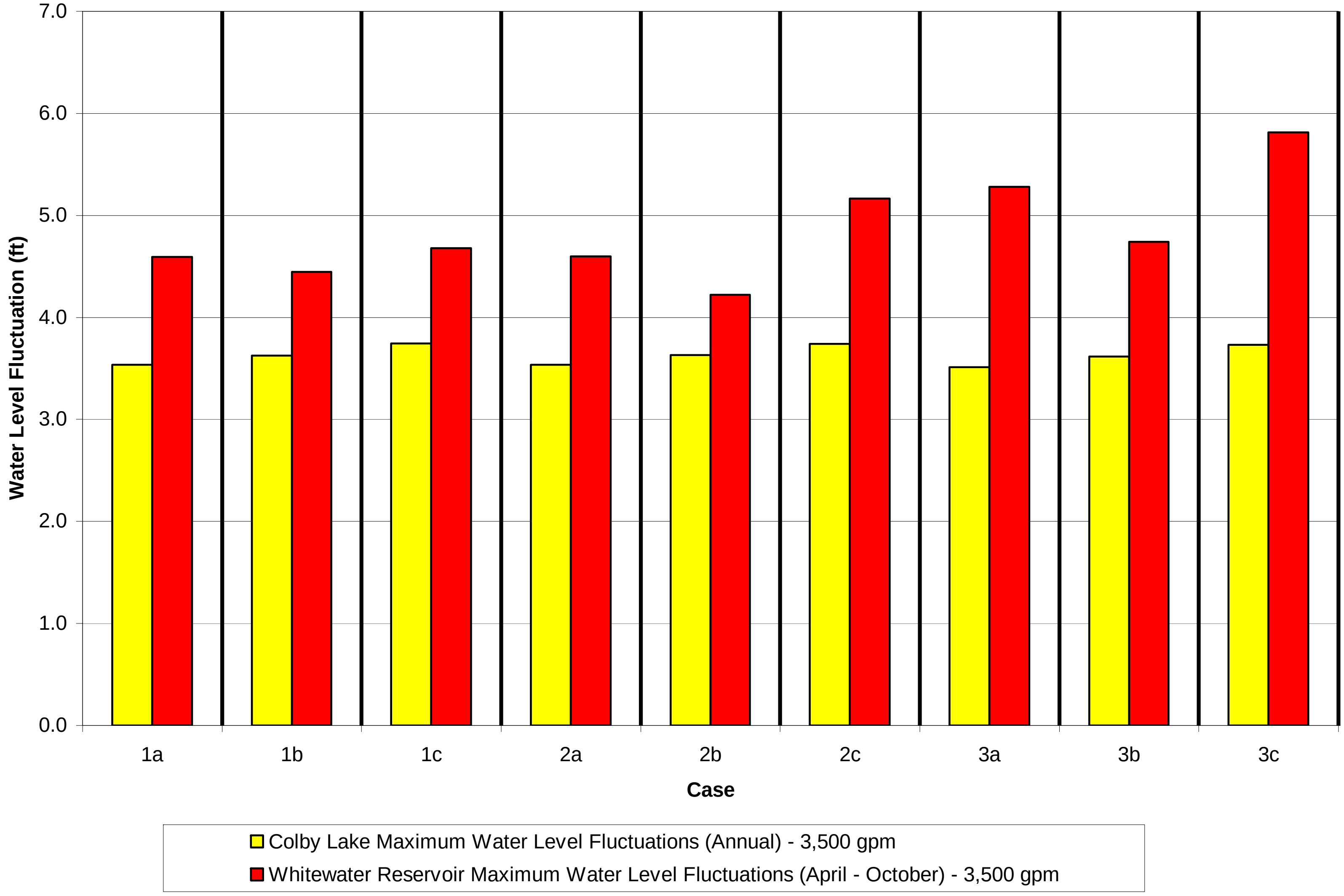


Figure 31: Modeled water levels in Colby Lake and Whitewater Reservoir for average flow conditions, Scenario 2b, including base case and various make-up water demands

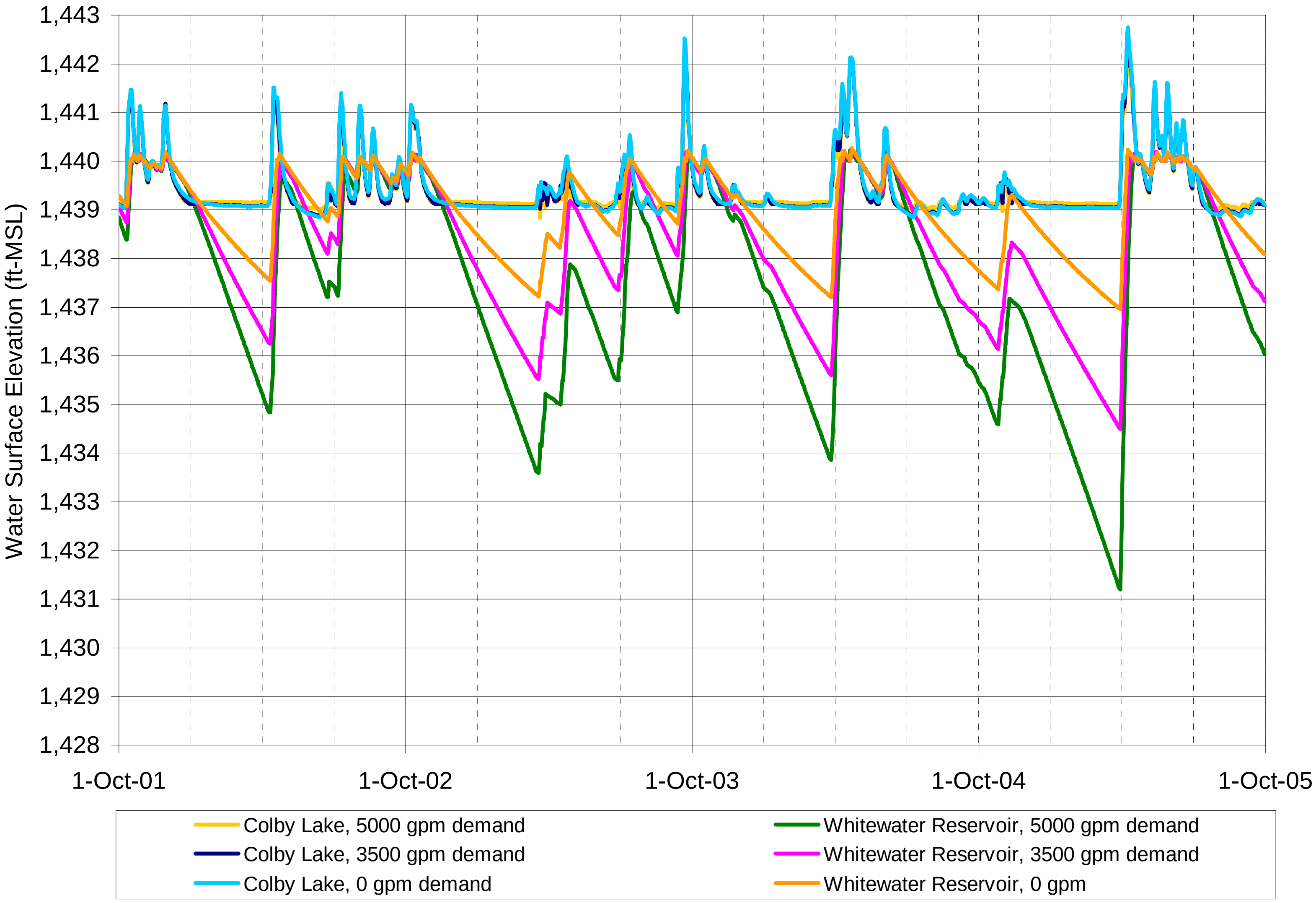


Figure 32: Elevation-duration curves for Colby Lake under average flow conditions, Scenario 2b, including base case and various make-up water demands

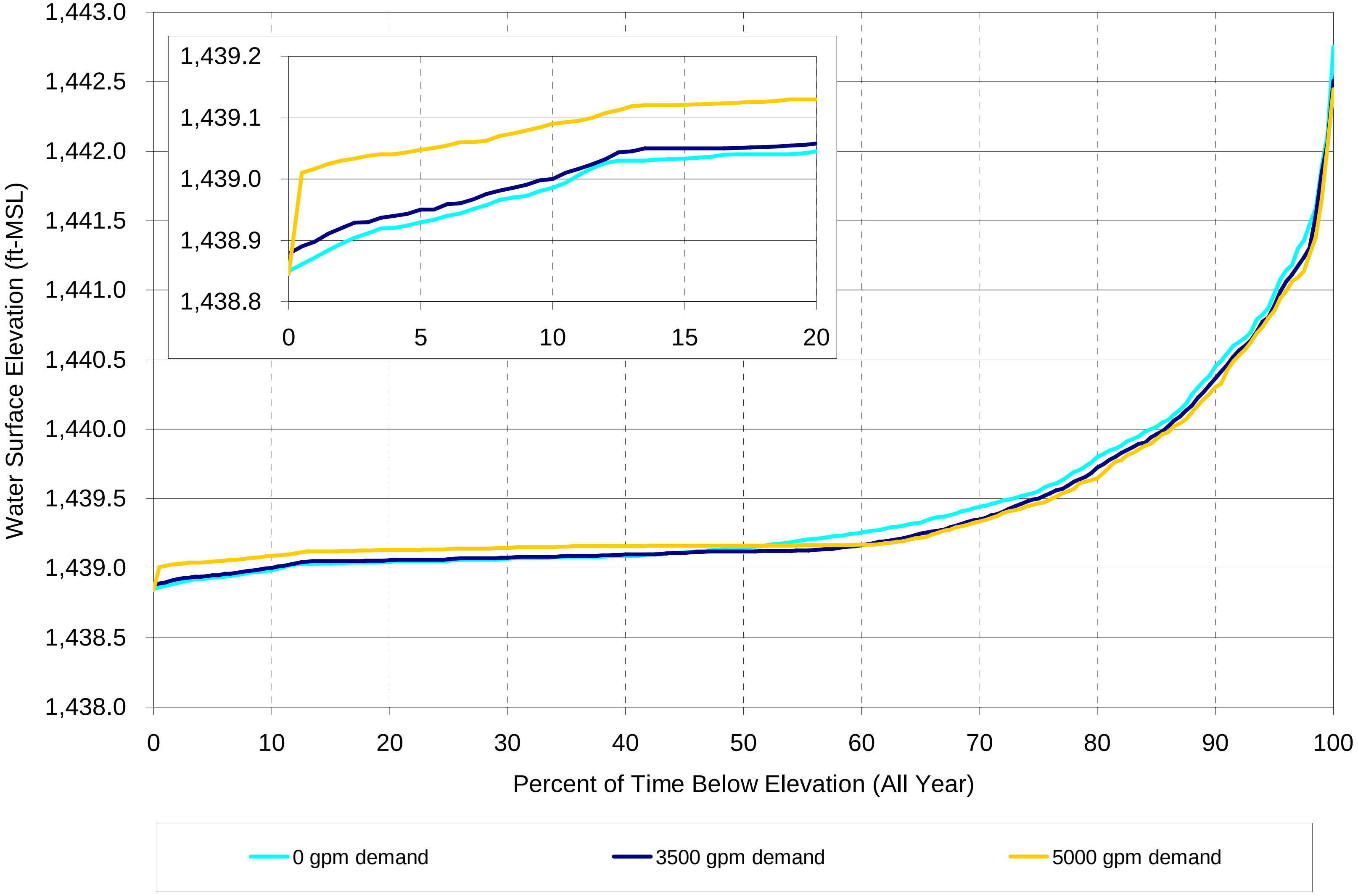


Figure 33: Elevation-duration curves for Whitewater Reservoir under average flow conditions, Scenario 2b, including base case and various make-up water demands

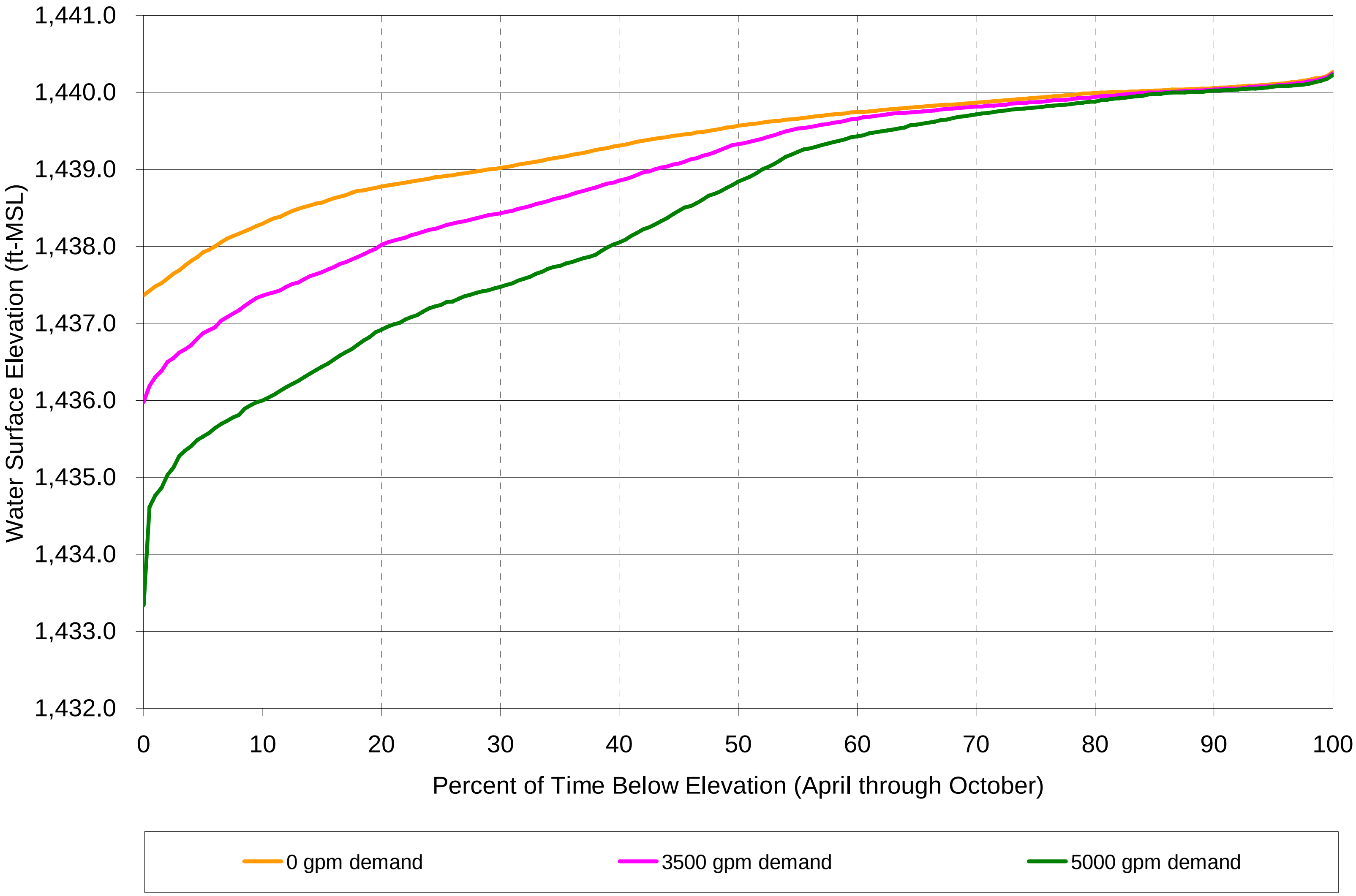


Figure 34: Modeled water levels in Colby Lake and Whitewater Reservoir for 2001-2005 flow conditions, Scenario 2b, including zero-demand case and various make-up water demands

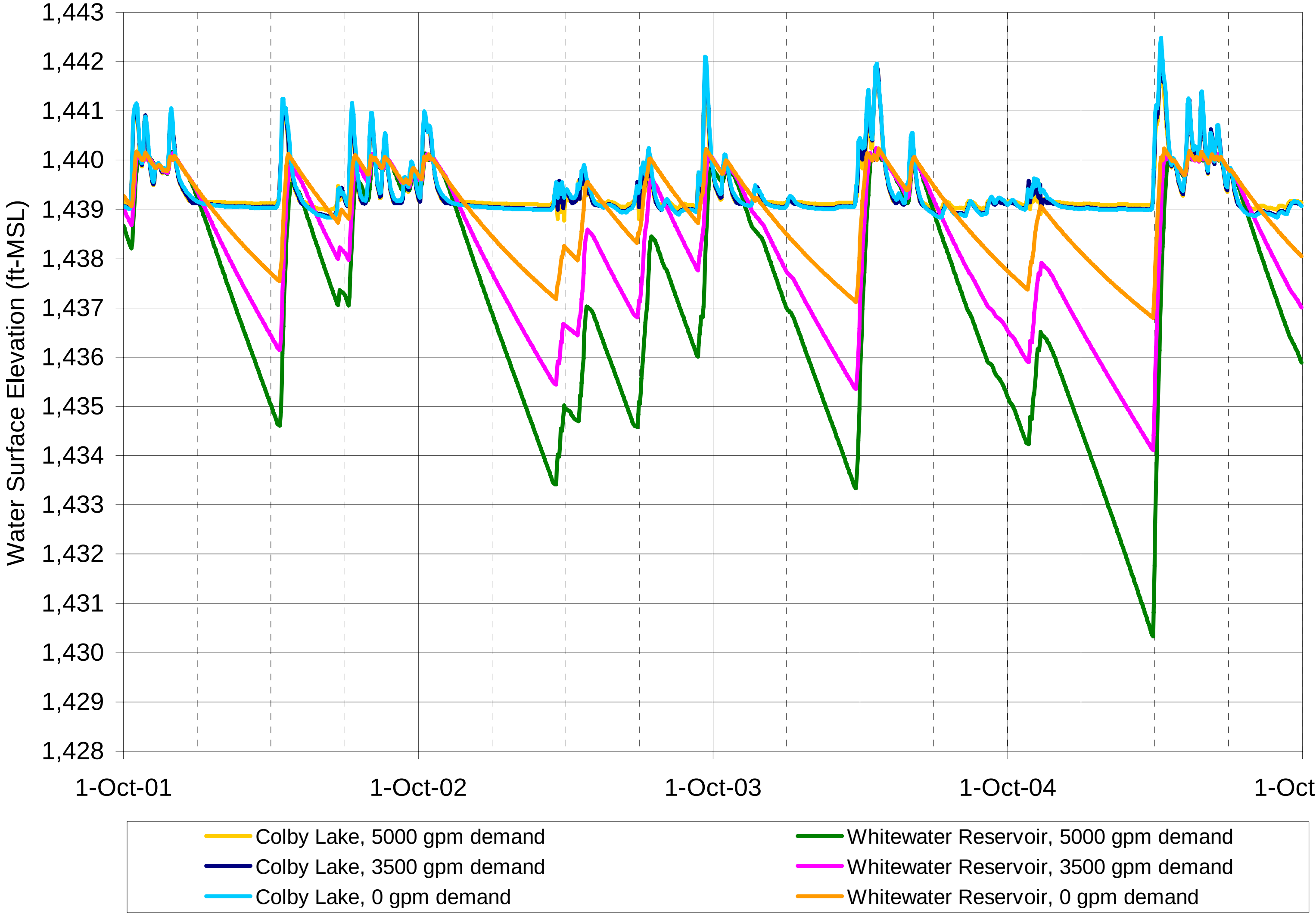


Figure 35: Elevation-duration curves for Colby Lake under 2001-2005 flow conditions, Scenario 2b, including zero-demand case and various make-up water demands

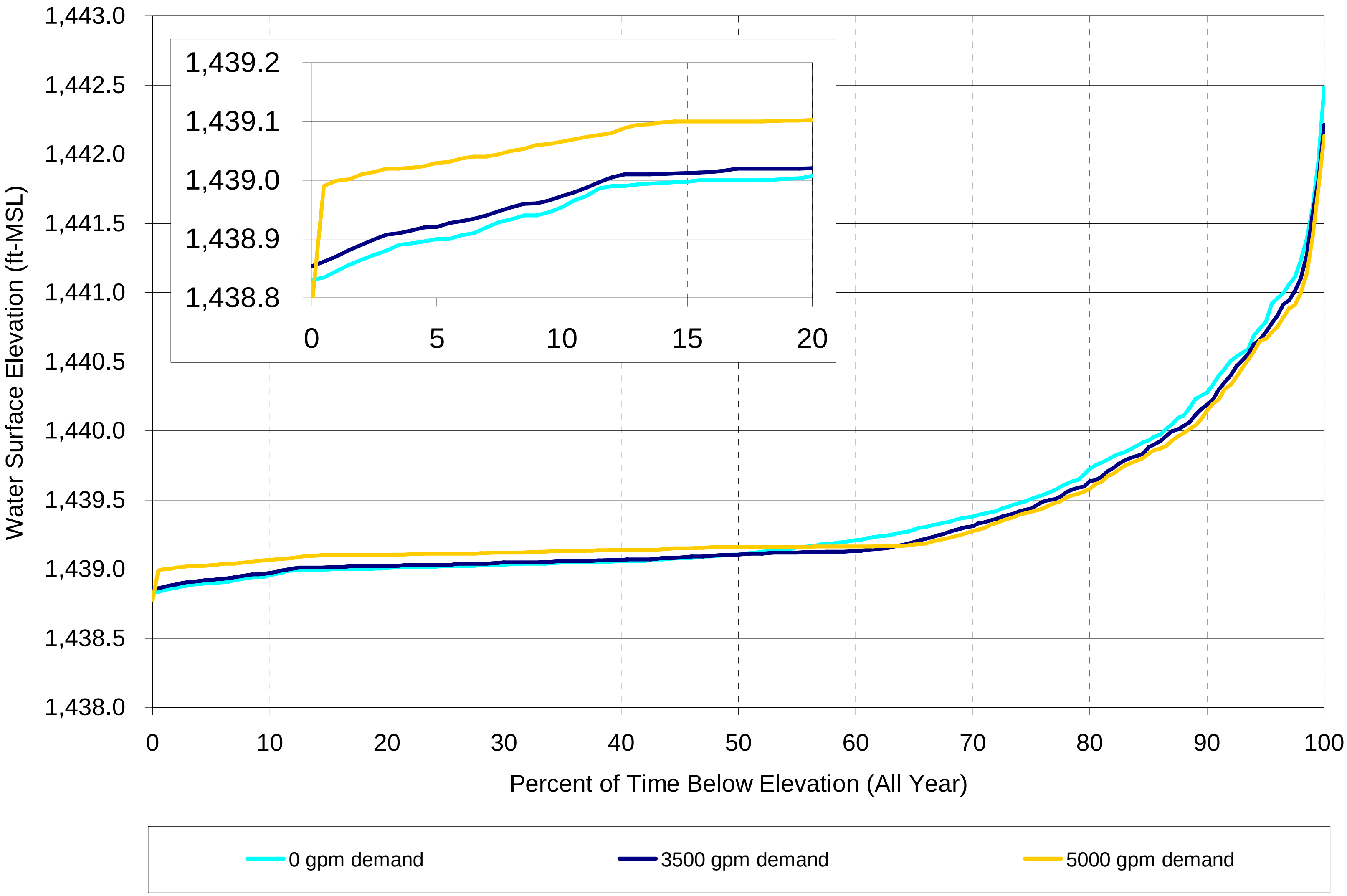


Figure 36: Elevation-duration curves for Whitewater Reservoir under 2001-2005 flow conditions, Scenario 2b, including zero-demand case and various make-up water demands

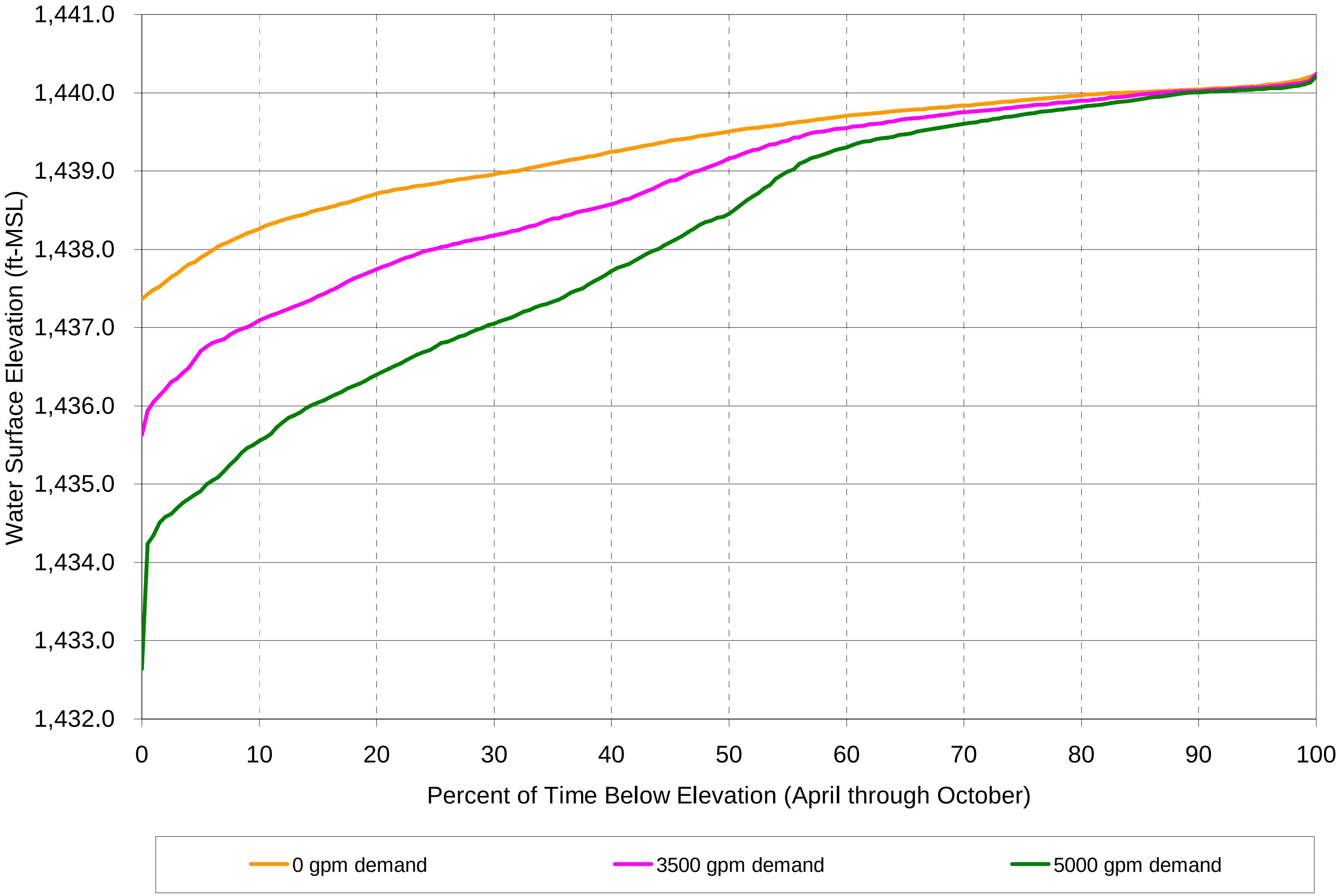


Figure 37: Modeled water levels in Colby Lake and Whitewater Reservoir for 50-year low-flow conditions, Scenario 2b, including zero-demand case and various make-up water demands

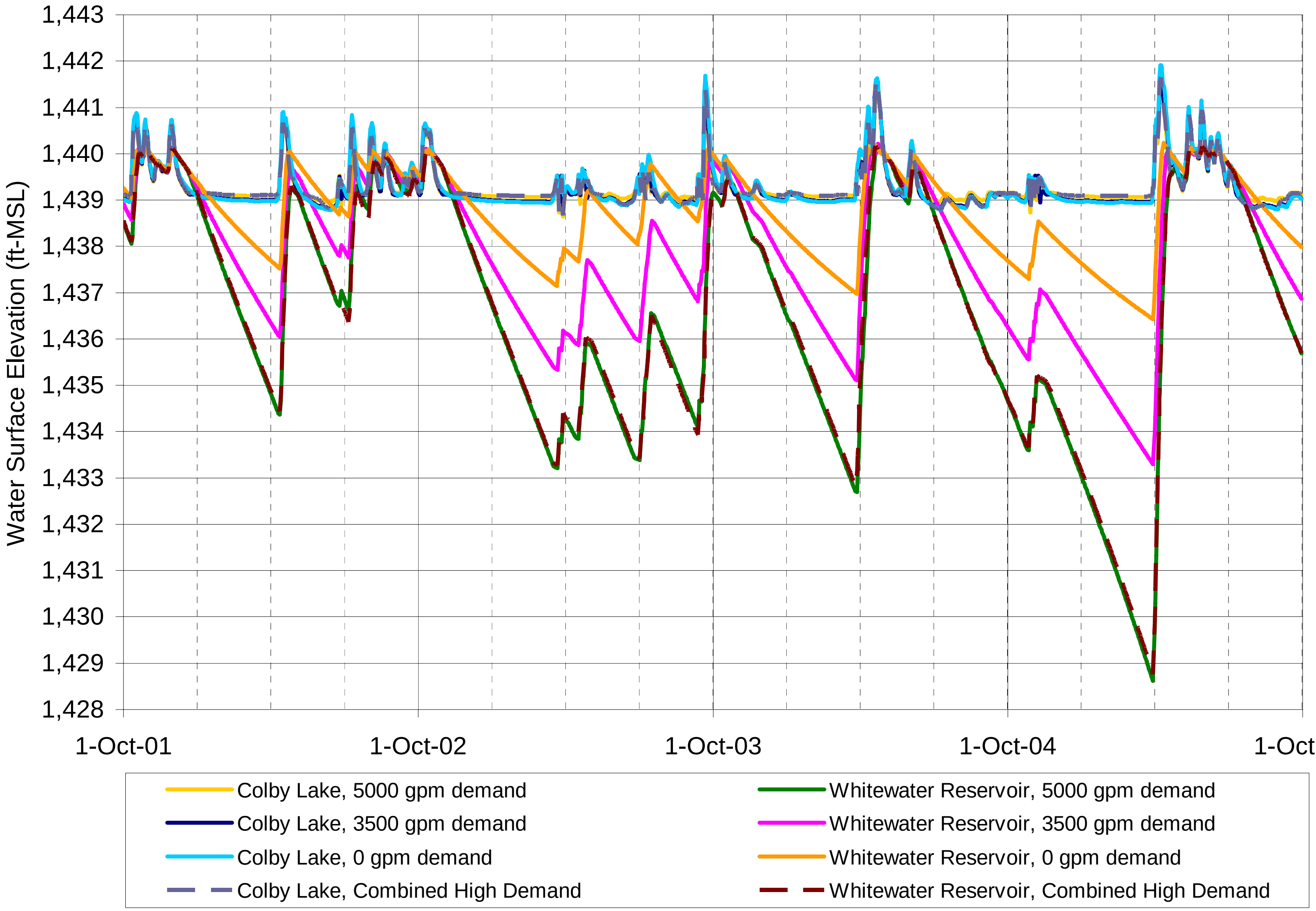


Figure 38: Elevation-duration curves for Colby Lake under 50-year low-flow conditions, Scenario 2b, including zero-demand case and various make-up water demands

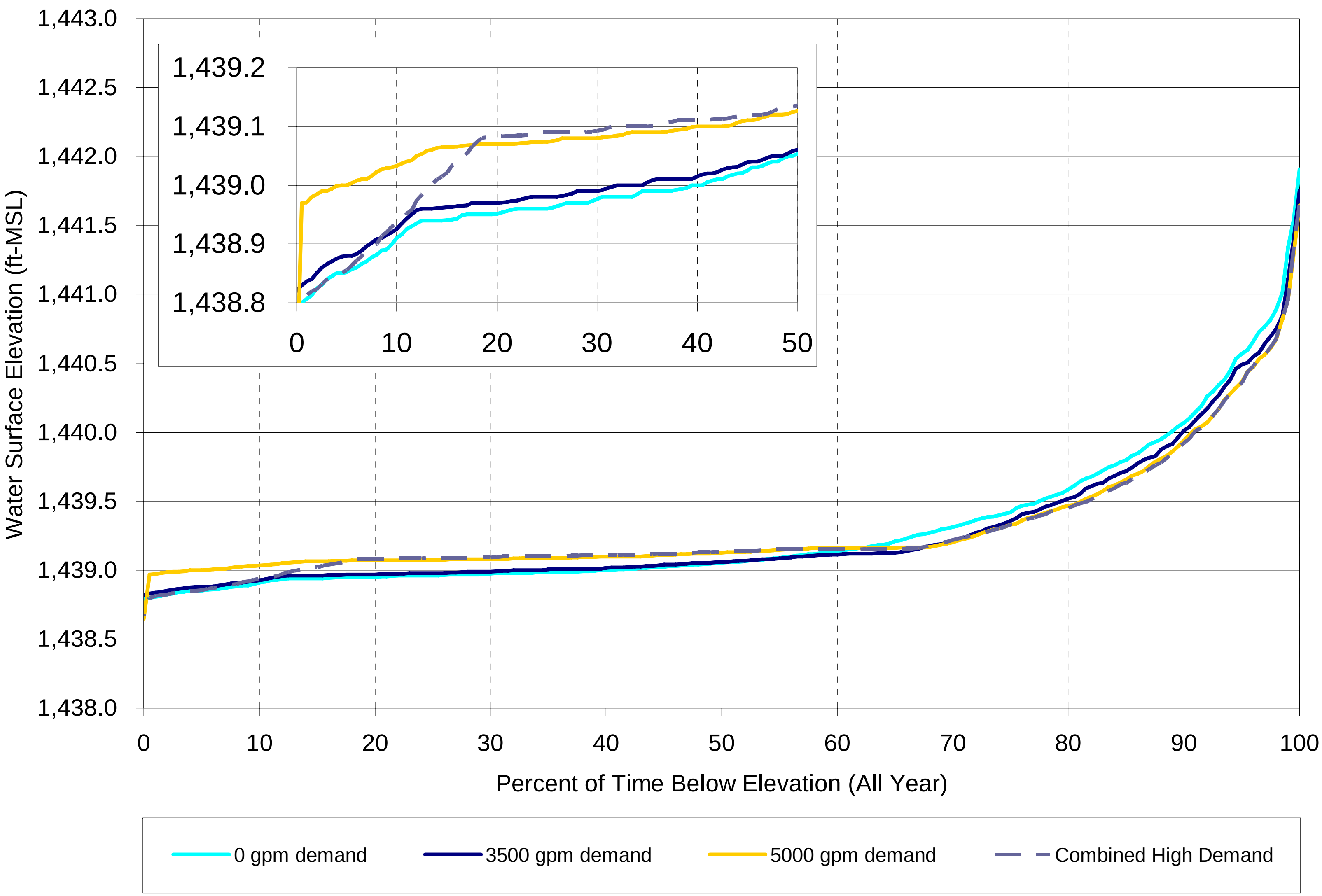


Figure 39: Elevation-duration curves for Whitewater Reservoir under 50-year low-flow conditions, Scenario 2b, including zero-demand case and various make-up water demands

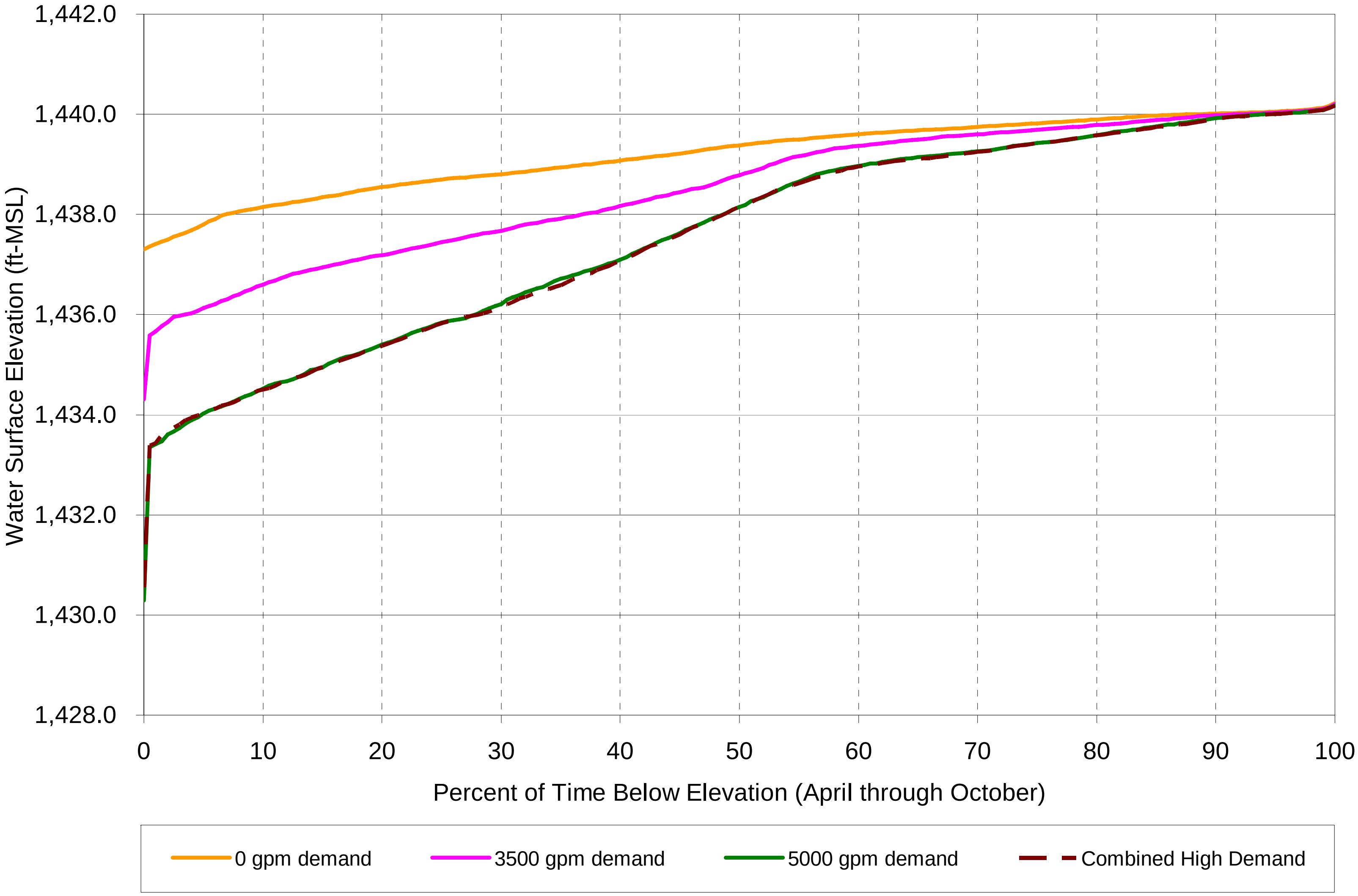


Figure 40: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for average flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute

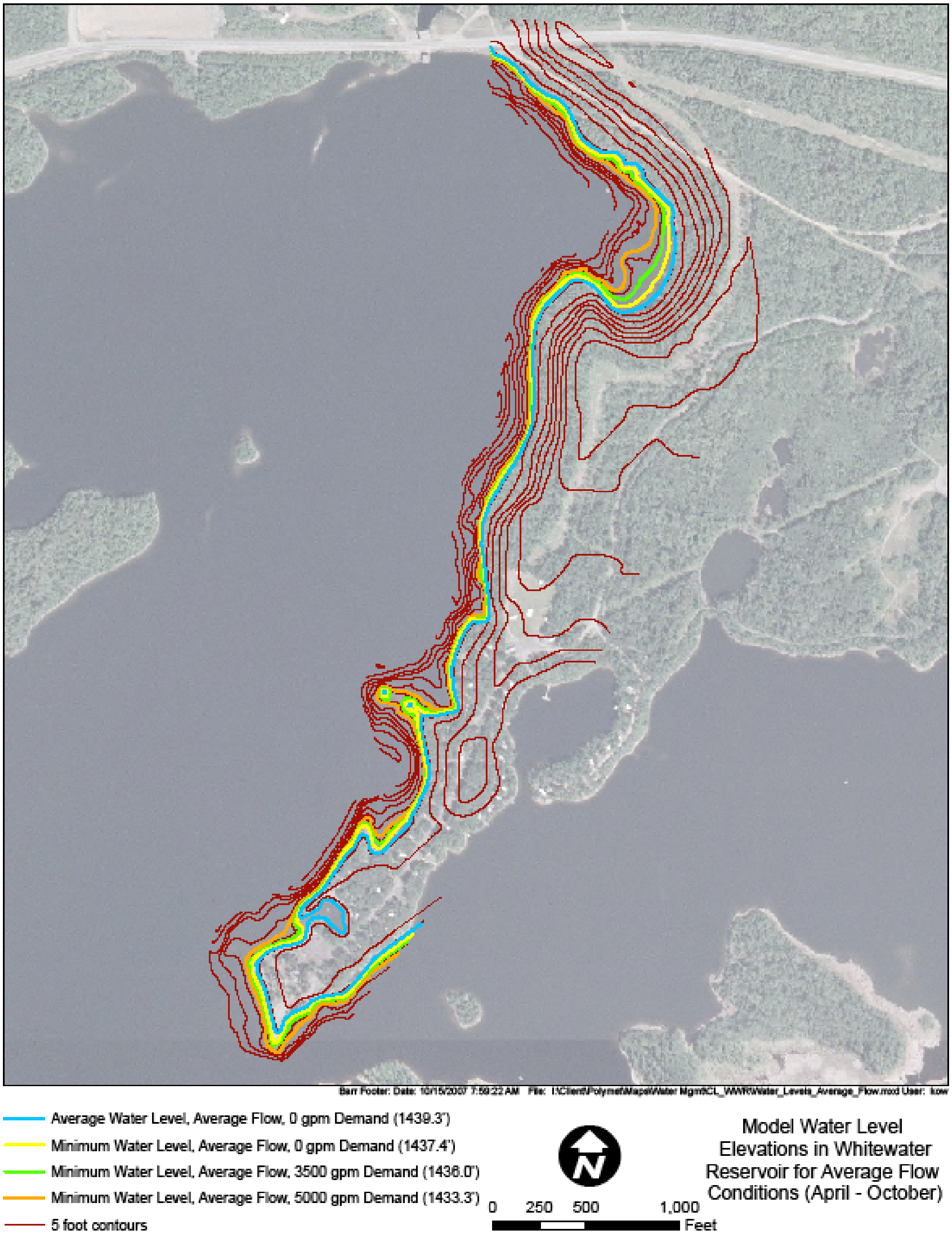


Figure 41: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 2001-2005 flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute

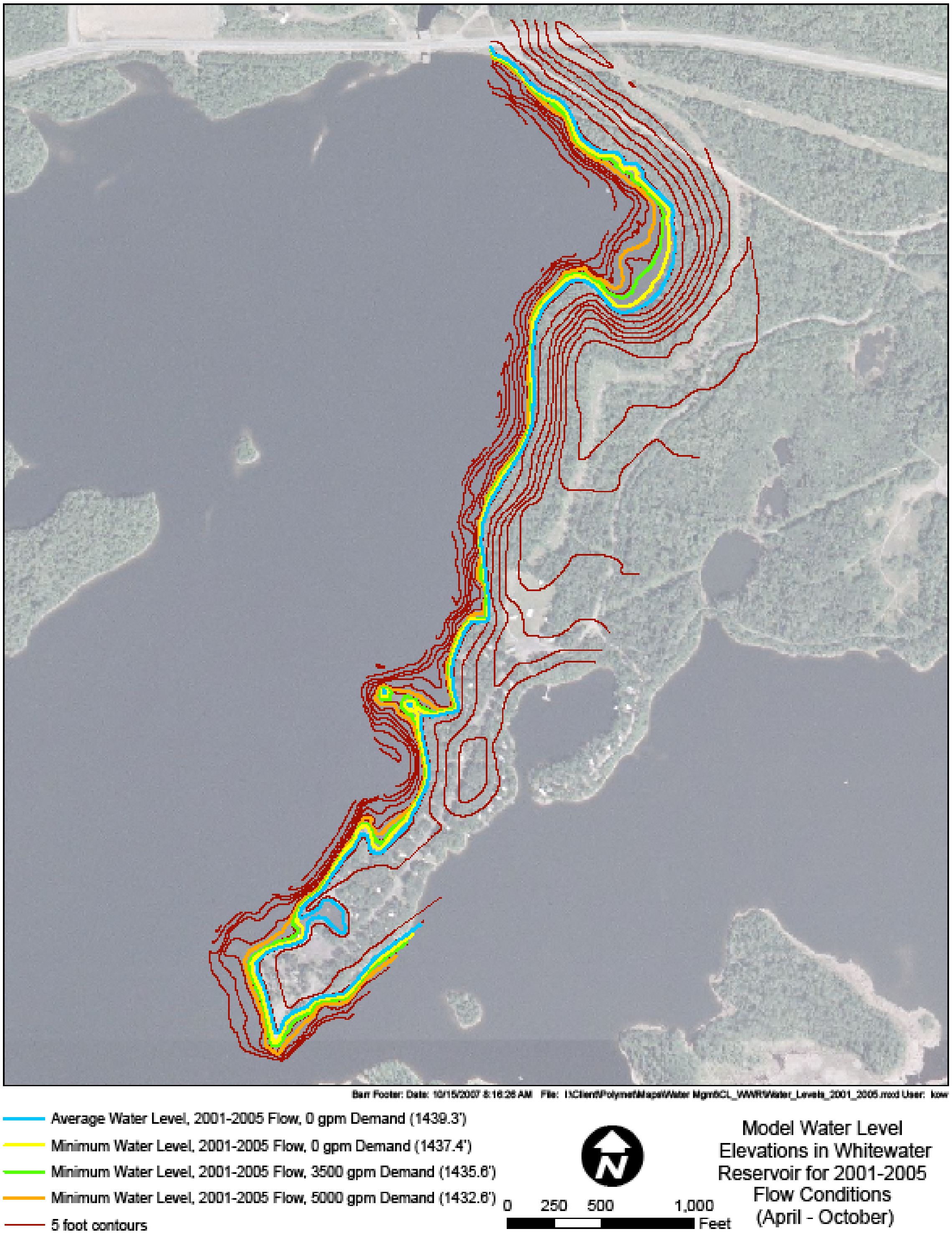
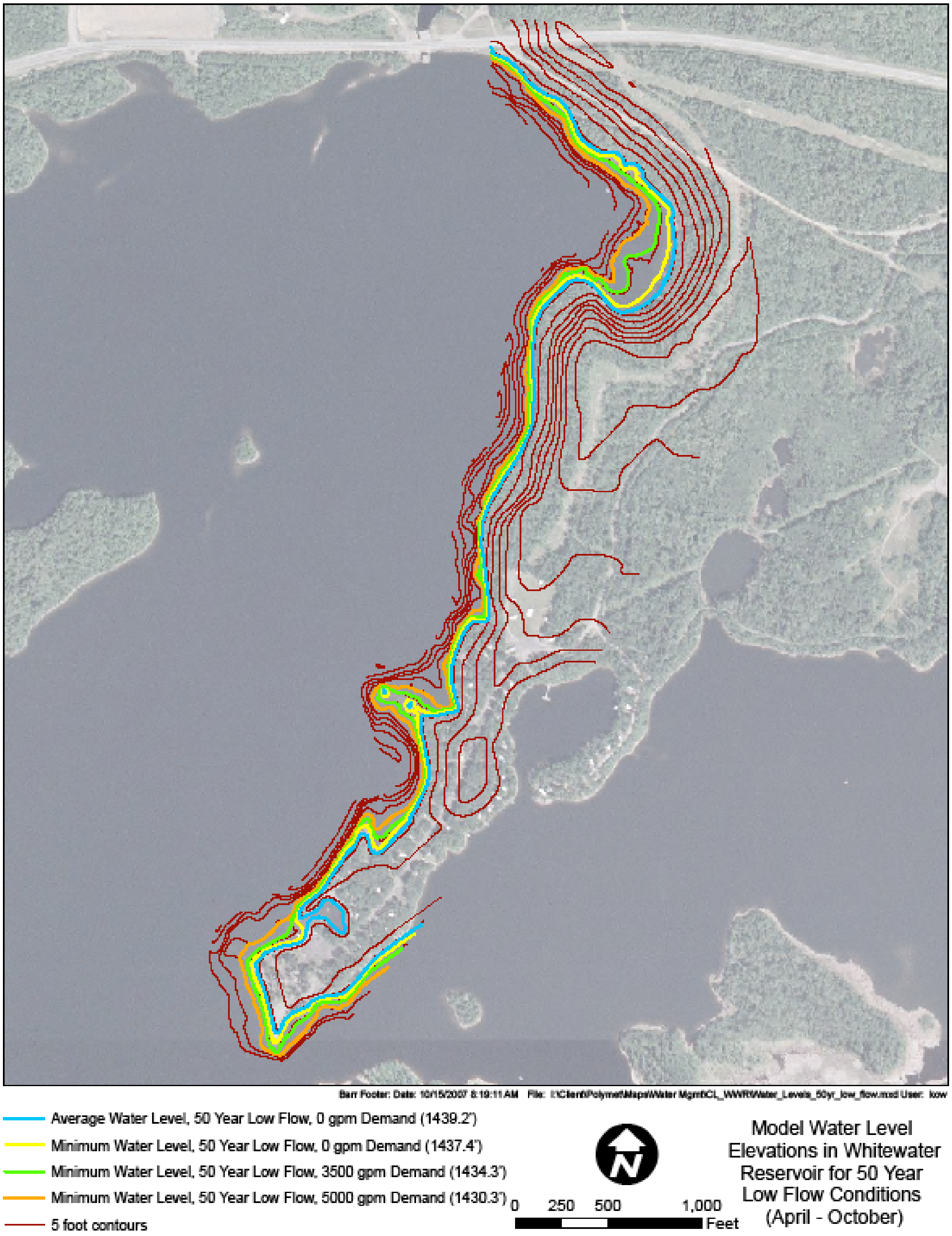


Figure 42: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 50-year low-flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute



Appendices

Appendix A

Barr Memorandum on “Hydrologic parameters used in the RVA (Richter et al., 1998)”

External Memorandum

To: Project File

From: Greg Williams, Miguel Wong

Subject: Hydrologic parameters used in the RVA (Richter et al., 1998)

Date: September 12, 2008

Introduction

This memo has been prepared in response to an initial request by the Minnesota Department of Natural Resources (MnDNR) and Knight Piesold during the meeting held in Barr's Minneapolis Office on July 9, 2007, and follow-up email communications, which resulted in agreement on the methodology proposed to deal with reductions in baseflow for different stages of the Mine Site development and closure (email from Jim Kunkel, Knight Piesold dated August 5, 2008).

This memo provides the results of calculating the hydrologic parameters used in the Range of Variability Approach (RVA) proposed by Richter et al. (1998) for the seven scenarios modeled with the hydrologic/hydraulic model of the Partridge River watershed (for more details, see RS73A and RS73B). The hydrologic parameters were calculated based on the results of the hydrologic/hydraulic model (XP-SWMM) with corrections to baseflow reductions estimated by groundwater modeling (MODFLOW; for more details see Appendix B of RS22).

The seven scenarios modeled are:

- Current Existing Conditions; that is, without NorthMet.
- Year 1; that is, by the end of the first year of mining operations.
- Year 5; that is, by the end of the fifth year of mining operations.
- Year 10; that is, by the end of the tenth year of mining operations.
- Year 15; that is, by the end of the fifteenth year of mining operations.

- Year 20; that is, by the end of the twentieth year of mining operations.
- Mine Facilities Off; that is, a hypothetical high-impact scenario in which all runoff from the footprint of the mine facilities, including reclaimed stockpiles, is collected and diverted to a different watershed.

The results are reported at the following locations along the Partridge River (see Figure 1):

- Surface water monitoring station SW-001. This location on the north branch of the Partridge River is upstream of all Mine Site facilities (but downstream of the Peter Mitchell Pit discharge), and its catchment area is 6.2 square miles.
- Surface water monitoring station SW-002. This location on the north branch of the Partridge River is northeast of the Mine Site, and its catchment area is 13.3 square miles.
- Surface water monitoring station SW-003. This location on the north branch of the Partridge River is east of the Mine Site, and its catchment area is 15.2 square miles.
- Surface water monitoring station SW-004. This location on the north branch of the Partridge River is immediately upstream of the confluence with the south branch, downstream of 64 percent of the proposed Mine Site facilities by the end of Year 20, and its catchment area is 23.0 square miles.
- Surface water monitoring station SW-004a. This location on the Partridge River is immediately downstream of the confluence of the north and south branches, downstream of 99 percent of the proposed Mine Site facilities by the end of Year 20, and its catchment area is 54.4 square miles.
- Surface water monitoring station SW-005. This location on the Partridge River is at the railway crossing, downstream of 100 percent of the proposed Mine Site facilities by the end of Year 20, and its catchment area is 98.7 square miles. The Mine Site (4.7 square miles) represents less than 5 percent of this watershed.
- USGS gaging station #04015475. This location on the Partridge River is upstream of Colby Lake, and its catchment area is 103.4 square miles.

Baseflow Corrections

The impacts to Partridge River flows were estimated in the hydrologic/hydraulic model by reducing the tributary area to account for the re-routing of Process Water from the Mine Site to the Tailings Basin and by adjusting watershed parameters to reflect the revised conditions. With this method, groundwater flows to the Partridge River are reduced by nearly the same percentage as surface water flows because the same reduction in tributary area is applied. In contrast, the groundwater model presented in Appendix B of RS22 estimates changes in groundwater discharge to streams (including the Partridge River) based on changes in groundwater flow directions and fluxes that result from mining activities, primarily pit dewatering.

The (groundwater) MODFLOW model was used to predict baseflow reductions to the Partridge River during “average” low-flow conditions. To do this, the MODFLOW model was calibrated to average water levels in wetland piezometers and a prediction of baseflow at the surface water monitoring station SW-004. The minimum 30-day low-flow for this station (predicted using the XP-SWMM model under Current Existing Conditions) was assumed to be a proxy for the average groundwater contribution to the stream. During low flow periods, it was assumed that all of the streamflow (i.e., the combination of groundwater contribution and surface runoff) is baseflow.

As indicated above, the MODFLOW model was used to predict baseflow reductions as a result of pit dewatering. The groundwater model estimates a cone of depression in the water table that extends beyond the surface watersheds that are affected by mining activity and, in some areas, beyond the boundaries of the Mine Site. As a result, the reduction in baseflow estimated by the groundwater model is greater than the reduction in baseflow estimated by the hydrologic/hydraulic model. More specifically, the MODFLOW model was designed to predict groundwater inflow rates to the pits and aquifer drawdowns that result from pit dewatering. The MODFLOW model was not designed to predict streamflow. However, the MODFLOW model can be used to predict relative changes to the amount of groundwater flow to and from the streams.

Since the groundwater model was not designed to estimate absolute groundwater flows to the Partridge River, the results of the model were only used to estimate the percent reduction in baseflow to the Partridge River from Current Existing Conditions. Flows estimated by the XP-SWMM model could not be separated into baseflow and surface flow components, so a correction was applied to estimated flows to account for the reduction in baseflow. A different correction was applied to low flow statistics than for the remaining statistics under high flow and average flow conditions.

For low flow statistics (Min 1-day flow, Min 3-day flow, Min 7-day flow, Min 30-day flow, and Min 90-day flow) the percent reduction for a given scenario estimated by the groundwater model (Table 1) was applied to the Current Existing Conditions flows at each reporting location. The description of the methodology is explained in further details below.

Table 1: Reduction in baseflow relative to Current Existing Conditions estimated by the groundwater model (MODFLOW)

Location	**Year 1**	**Year 5**	**Year 10**	**Year 15**	**Year 20**	**Mine Facilities Off**
SW-001	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
SW-002	3.7%	7.7%	12.6%	13.9%	21.8%	21.8%
SW-003	3.4%	6.9%	11.3%	12.0%	19.5%	19.5%
SW-004	2.0%	5.2%	8.7%	8.8%	14.8%	14.8%
SW-004a	0.8%	2.2%	3.9%	3.8%	6.5%	6.5%
SW-005	0.4%	1.1%	2.0%	1.9%	3.3%	3.3%
USGS gage	0.4%	1.0%	1.8%	1.8%	3.0%	3.0%

Surface water monitoring station SW-001 is located at the headwaters of the Partridge River, and Table 1 below shows that the reduction in baseflow at this location is zero. Headwater portions of streams typically fluctuate between gaining and loosing water to the groundwater system seasonally, especially in a wetland dominated environment like Hundred Mile Swamp. Thus, during periods with high groundwater levels the stream will gain water and during periods with low groundwater levels the stream will loose water. This seasonal reversal of flow between groundwater and the stream can not accurately be captured by the XP-SWMM model, which has a simplified subroutine for the calculation of groundwater flux or the groundwater model, which is steady-state. As such, the furthest upstream that baseflow reductions were predicted with some confidence are at surface water monitoring station SW-002, where the stream is likely a gaining stream for most of the year. The baseflow reductions predicted using the groundwater model at SW-002 takes into account aquifer drawdowns in the entire watershed up-gradient of the monitoring location; that is, it includes average annual gains or losses at SW-001. In general, when considering water quantity and water quality impacts, it is important to note that SW-002 is located downstream of 5.2 percent of the Mine Site facilities, whereas SW-001 is located upstream of all Mine Site facilities.

The percent reduction for SW-002 in Table 1 was used to calculate baseflows at this location for different stages of the Mine Site development and closure. The same methodology was not applied for locations downstream of SW-002 because this might result in “artificial” gains of baseflow at SW-003 and downstream. Instead, the percent reductions in Table 1 were first divided by incremental watershed area (e.g., the sub-watershed between SW-002 and SW-003), then applied to the Current Existing Conditions incremental baseflows for the incremental watershed area, and finally integrated to obtain the absolute reduction in baseflow (i.e., cubic feet per second instead of percentages) at SW-003 and SW-004. In the case of SW-004a, the absolute reduction in baseflow accounted for the contribution from the south branch of the Partridge River (not affected by the Mine Site) as well as for the Mine Site area that drains to this location as opposed to SW-004 (i.e., a factor of 99/64 applied to the incremental percent reduction between SW-003 and SW-004). No additional baseflow reductions happen downstream of SW-004a. The absolute reductions in baseflow for all locations of analysis and stages of Mine Site development are presented in Table 2 below.

Table 2: Reduction in average groundwater flow (cfs) relative to Current Existing Conditions applied to average and high flow statistics.

Location	Year 1	Year 5	Year 10	Year 15	Year 20	Mine Facilities Off
SW-001	0	0	0	0	0	0
SW-002	0.02	0.04	0.07	0.08	0.12	0.12
SW-003	0.02	0.05	0.08	0.08	0.13	0.13
SW-004	0.02	0.05	0.09	0.09	0.15	0.15
SW-004a	0.02	0.05	0.09	0.09	0.16	0.16
SW-005	0.02	0.05	0.09	0.09	0.16	0.16
USGS gage	0.02	0.05	0.09	0.09	0.16	0.16

It is important to mention that during average and high flow conditions, streamflow is composed of baseflow and surface runoff. The baseflow contribution to the streamflow is negligible during these periods, in particular for high flow conditions, so the results from the XP-SWMM model were considered appropriate and sufficient to characterize streamflows for periods when there is more than just baseflow conditions. To compute the statistics for average and high flow conditions, however, the streamflow obtained from the XP-SWMM model was reduced by the predicted reduction in baseflow by the MODFLOW model.

For average and high flow statistics (Mean annual flow, Mean monthly flows, Max 1-day flow, Max 3-day flow, Max 7-day flow, Max 30-day flow, and Max 90-day flow) an absolute baseflow reduction was applied based on the average existing conditions groundwater flow estimated by the hydrologic/hydraulic model and the reduction of baseflows from existing conditions predicted by the groundwater model (Table 2). For example, at SW-002 the hydrologic/hydraulic model estimated an average groundwater flow of 0.56 cubic feet per second -cfs- during existing conditions; the groundwater model estimated a 12.6 percent reduction in baseflow at this location in Year 10 of mine operation. A correction of 12.6 percent of 0.56 cfs, or 0.07 cfs, was subtracted from all average and high flow statistics from hydrologic/hydraulic model results to estimate the corrected statistics in

Table 2d. During months where the monthly average flows represented low flow conditions, the percent correction reflected in Table 1 was applied instead of the absolute correction in Table 2. These months were defined as when the average monthly flow was less than the average groundwater flow estimated by the hydrologic/hydraulic model.

References

Richter, B.D., Baumgartner, J.V., Braun, D.P., and Powell, J. (1998). A spatial assessment of hydrologic alteration within a river network. *Regulated Rivers: Research & Management*, 14, 329-340.

Table 1a: Summary of modeled hydrologic parameters for **Existing Conditions** at surface water monitoring location **SW-001**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	0.8	1.5	2.9	13.9	19.4	9.6	0.9	8.5	0.9	0.3
Mean Nov. flow	cfs	0.4	4.4	1.7	3.2	7.8	3.2	1.1	2.7	1.3	0.4
Mean Dec. flow	cfs	0.3	1.3	0.8	1.2	3.4	1.7	1.4	1.1	0.7	0.4
Mean Jan. flow	cfs	0.3	0.7	0.6	0.7	1.1	0.8	0.8	0.7	0.5	0.4
Mean Feb. flow	cfs	0.3	0.5	3.9	0.6	0.7	0.8	0.6	0.5	0.5	0.4
Mean Mar. flow	cfs	1.9	1.3	6.0	1.5	2.2	1.3	6.5	4.0	10.9	2.6
Mean Apr. flow	cfs	14.9	5.6	15.4	11.8	10.3	20.7	11.2	17.7	3.2	7.2
Mean May flow	cfs	6.1	0.5	4.7	9.8	2.3	4.8	6.6	8.6	11.2	1.6
Mean Jun. flow	cfs	0.9	0.3	1.6	0.9	0.2	5.4	9.5	0.7	3.5	0.2
Mean Jul. flow	cfs	0.3	0.2	1.4	4.1	0.5	0.4	3.9	0.3	3.7	0.2
Mean Aug. flow	cfs	0.2	0.5	0.2	1.2	1.2	0.2	1.1	0.3	0.8	10.2
Mean Sep. flow	cfs	0.4	2.1	0.4	4.1	2.2	0.2	6.7	1.2	0.7	4.7
Max. 1-day flow	cfs	50.4	20.2	36.1	36.4	45.2	49.1	34.7	45.4	42.4	36.3
Max. 3-day flow	cfs	44.0	16.0	28.4	32.0	40.3	45.8	28.1	38.8	33.6	29.1
Max. 7-day flow	cfs	37.6	11.2	25.8	27.2	39.5	37.4	21.8	31.7	29.8	22.5
Max. 30-day flow	cfs	19.0	6.1	15.8	16.2	19.9	21.5	11.5	17.9	14.3	12.8
Max. 90-day flow	cfs	7.8	2.5	9.7	9.5	11.0	10.4	9.4	10.3	9.0	5.1
Min. 1-day flow	cfs	0.04	0.04	0.06	0.06	0.05	0.04	0.07	0.04	0.06	0.05
Min. 3-day flow	cfs	0.05	0.04	0.06	0.07	0.05	0.05	0.07	0.04	0.07	0.05
Min. 7-day flow	cfs	0.06	0.04	0.06	0.14	0.05	0.05	0.07	0.05	0.12	0.05
Min. 30-day flow	cfs	0.14	0.14	0.10	0.54	0.14	0.06	0.26	0.23	0.42	0.10
Min. 90-day flow	cfs	0.28	0.21	0.56	0.68	0.35	0.15	0.57	0.40	0.40	0.36
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.025	0.028	0.018	0.032	0.012	0.012	0.017	0.012	0.038	0.020
Julian date of max. flow	day	25-Apr	1-Nov	23-Apr	17-Apr	7-Oct	15-Apr	23-Apr	2-Apr	23-May	25-Aug
Julian date of min. flow	day	26-Jul	8-Jul	16-Aug	2-Jul	17-Jun	25-Aug	15-Oct	15-Jul	10-Jul	24-Jul
Number of high pulses[1]	#/year	9	9	5	11	8	6	14	3	5	5
Number of low pulses[2]	#/year	16	17	11	5	8	6	1	12	9	10
Mean high pulse duration	days	5.2	5.9	22.2	11.7	15.8	18.5	9.9	34.0	16.4	14.8
Mean low pulse duration	days	13.0	6.9	6.5	4.4	9.0	12.2	21.0	5.0	5.3	16.0
Mean rate of flow increase[3]	cfs/day	1.9	1.2	2.2	3.8	2.5	1.9	3.0	1.6	2.2	1.3
Mean rate of flow decrease[4]	cfs/day	0.4	0.3	0.5	0.7	0.6	0.5	0.6	0.5	0.5	0.5
Number of flow reversals	#/year	65	78	89	69	76	61	70	86	63	75

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 1b: Summary of modeled hydrologic parameters for **Mine Year 1** at surface water monitoring location **SW-001**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	0.8	1.5	2.9	13.9	19.4	9.6	0.9	8.5	0.9	0.3
Mean Nov. flow	cfs	0.4	4.4	1.7	3.2	7.8	3.2	1.1	2.7	1.3	0.4
Mean Dec. flow	cfs	0.3	1.3	0.8	1.2	3.4	1.7	1.4	1.1	0.7	0.4
Mean Jan. flow	cfs	0.3	0.7	0.6	0.7	1.1	0.8	0.8	0.7	0.5	0.4
Mean Feb. flow	cfs	0.3	0.5	3.9	0.6	0.7	0.8	0.6	0.5	0.5	0.4
Mean Mar. flow	cfs	1.9	1.3	6.0	1.5	2.2	1.3	6.5	4.0	10.9	2.6
Mean Apr. flow	cfs	14.9	5.6	15.4	11.8	10.3	20.7	11.1	17.7	3.2	7.2
Mean May flow	cfs	6.1	0.5	4.7	9.8	2.3	4.8	6.6	8.6	11.2	1.6
Mean Jun. flow	cfs	0.9	0.3	1.6	0.9	0.2	5.4	9.5	0.7	3.6	0.2
Mean Jul. flow	cfs	0.3	0.2	1.4	4.1	0.5	0.4	3.9	0.3	3.7	0.2
Mean Aug. flow	cfs	0.2	0.5	0.2	1.2	1.2	0.2	1.1	0.3	0.8	10.2
Mean Sep. flow	cfs	0.4	2.1	0.4	4.1	2.2	0.2	6.7	1.2	0.7	4.7
Max. 1-day flow	cfs	50.4	20.3	36.1	36.4	45.2	49.1	34.7	45.4	42.4	36.3
Max. 3-day flow	cfs	44.0	16.0	28.4	32.0	40.3	45.8	28.1	38.8	33.6	29.1
Max. 7-day flow	cfs	37.6	11.2	25.8	27.2	39.5	37.4	21.8	31.7	29.8	22.5
Max. 30-day flow	cfs	19.0	6.1	15.8	16.2	19.9	21.5	11.5	17.9	14.3	12.7
Max. 90-day flow	cfs	7.8	2.5	9.7	9.5	11.0	10.4	9.4	10.3	9.0	5.1
Min. 1-day flow	cfs	0.04	0.04	0.06	0.06	0.05	0.04	0.07	0.04	0.06	0.05
Min. 3-day flow	cfs	0.05	0.04	0.06	0.08	0.05	0.05	0.07	0.04	0.07	0.05
Min. 7-day flow	cfs	0.06	0.04	0.06	0.14	0.05	0.05	0.07	0.05	0.12	0.05
Min. 30-day flow	cfs	0.14	0.14	0.10	0.54	0.14	0.06	0.26	0.23	0.42	0.10
Min. 90-day flow	cfs	0.28	0.21	0.56	0.68	0.35	0.15	0.57	0.40	0.40	0.36
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.025	0.029	0.018	0.032	0.012	0.012	0.017	0.012	0.038	0.020
Julian date of max. flow	day	25-Apr	1-Nov	23-Apr	17-Apr	7-Oct	15-Apr	23-Apr	2-Apr	23-May	25-Aug
Julian date of min. flow	day	26-Jul	8-Jul	16-Aug	2-Jul	18-Jun	25-Aug	15-Oct	15-Jul	10-Jul	24-Jul
Number of high pulses[1]	#/year	9	9	5	11	8	6	14	3	5	5
Number of low pulses[2]	#/year	16	17	11	6	8	6	1	12	9	10
Mean high pulse duration	days	5.2	5.9	22.2	11.7	15.8	18.5	9.9	34.0	16.2	14.8
Mean low pulse duration	days	13.0	6.9	6.5	3.5	9.0	12.2	21.0	5.0	5.3	16.1
Mean rate of flow increase[3]	cfs/day	2.0	1.1	2.2	3.8	2.6	1.9	3.0	1.6	2.2	1.3
Mean rate of flow decrease[4]	cfs/day	0.4	0.3	0.5	0.7	0.6	0.5	0.6	0.5	0.5	0.5
Number of flow reversals	#/year	64	80	88	69	77	65	70	88	63	74

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 1c: Summary of modeled hydrologic parameters for **Mine Year 5** at surface water monitoring location **SW-001**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	0.8	1.5	2.9	13.9	19.4	9.6	0.9	8.5	0.9	0.3
Mean Nov. flow	cfs	0.4	4.4	1.7	3.2	7.8	3.2	1.1	2.7	1.3	0.4
Mean Dec. flow	cfs	0.3	1.3	0.8	1.2	3.4	1.7	1.4	1.1	0.7	0.4
Mean Jan. flow	cfs	0.3	0.7	0.6	0.7	1.1	0.8	0.8	0.7	0.5	0.4
Mean Feb. flow	cfs	0.3	0.5	3.9	0.6	0.7	0.8	0.6	0.5	0.5	0.4
Mean Mar. flow	cfs	1.9	1.3	6.0	1.5	2.2	1.3	6.5	4.0	10.9	2.6
Mean Apr. flow	cfs	14.9	5.6	15.4	11.8	10.3	20.7	11.1	17.7	3.2	7.2
Mean May flow	cfs	6.1	0.5	4.7	9.8	2.3	4.8	6.6	8.6	11.2	1.6
Mean Jun. flow	cfs	0.9	0.3	1.6	0.9	0.2	5.4	9.5	0.7	3.6	0.2
Mean Jul. flow	cfs	0.3	0.2	1.4	4.1	0.5	0.4	3.9	0.3	3.7	0.2
Mean Aug. flow	cfs	0.2	0.5	0.2	1.2	1.2	0.2	1.1	0.3	0.8	10.2
Mean Sep. flow	cfs	0.4	2.1	0.4	4.1	2.2	0.2	6.7	1.2	0.7	4.7
Max. 1-day flow	cfs	50.4	20.3	36.1	36.4	45.2	49.1	34.7	45.4	42.4	36.3
Max. 3-day flow	cfs	44.0	16.0	28.4	32.0	40.3	45.8	28.1	38.8	33.6	29.1
Max. 7-day flow	cfs	37.6	11.2	25.8	27.2	39.5	37.4	21.8	31.7	29.8	22.5
Max. 30-day flow	cfs	19.0	6.1	15.8	16.2	19.9	21.5	11.5	17.9	14.3	12.8
Max. 90-day flow	cfs	7.8	2.5	9.7	9.5	11.0	10.4	9.4	10.3	9.0	5.1
Min. 1-day flow	cfs	0.04	0.04	0.06	0.06	0.05	0.04	0.07	0.04	0.06	0.05
Min. 3-day flow	cfs	0.05	0.04	0.06	0.08	0.05	0.05	0.07	0.04	0.07	0.05
Min. 7-day flow	cfs	0.06	0.05	0.06	0.14	0.05	0.05	0.07	0.05	0.12	0.05
Min. 30-day flow	cfs	0.14	0.14	0.10	0.54	0.14	0.06	0.26	0.23	0.42	0.10
Min. 90-day flow	cfs	0.28	0.21	0.56	0.68	0.35	0.15	0.57	0.40	0.40	0.36
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.025	0.029	0.018	0.032	0.012	0.012	0.017	0.012	0.038	0.020
Julian date of max. flow	day	25-Apr	1-Nov	23-Apr	17-Apr	7-Oct	15-Apr	23-Apr	2-Apr	23-May	25-Aug
Julian date of min. flow	day	26-Jul	8-Jul	16-Aug	2-Jul	18-Jun	25-Aug	15-Oct	15-Jul	10-Jul	24-Jul
Number of high pulses[1]	#/year	9	9	5	11	8	6	14	3	5	5
Number of low pulses[2]	#/year	16	17	11	6	8	6	1	12	9	10
Mean high pulse duration	days	5.2	5.9	22.2	11.7	15.8	18.5	9.9	34.0	16.2	14.8
Mean low pulse duration	days	13.0	6.9	6.5	3.5	9.0	12.2	21.0	5.0	5.3	16.1
Mean rate of flow increase[3]	cfs/day	2.0	1.1	2.2	3.7	2.6	1.9	3.0	1.6	2.2	1.3
Mean rate of flow decrease[4]	cfs/day	0.4	0.3	0.6	0.7	0.6	0.5	0.6	0.5	0.5	0.5
Number of flow reversals	#/year	64	80	89	70	76	65	70	87	63	74

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 1d: Summary of modeled hydrologic parameters for **Mine Year 10** at surface water monitoring location **SW-001**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	0.8	1.5	2.9	13.9	19.4	9.6	0.9	8.5	0.9	0.3
Mean Nov. flow	cfs	0.4	4.4	1.7	3.2	7.8	3.2	1.1	2.7	1.3	0.4
Mean Dec. flow	cfs	0.3	1.3	0.8	1.2	3.4	1.7	1.4	1.1	0.7	0.4
Mean Jan. flow	cfs	0.3	0.7	0.6	0.7	1.1	0.8	0.8	0.7	0.5	0.4
Mean Feb. flow	cfs	0.3	0.5	3.9	0.6	0.7	0.8	0.6	0.5	0.5	0.4
Mean Mar. flow	cfs	1.9	1.3	6.0	1.5	2.2	1.3	6.5	4.0	10.9	2.6
Mean Apr. flow	cfs	14.9	5.6	15.4	11.8	10.3	20.7	11.1	17.7	3.2	7.2
Mean May flow	cfs	6.1	0.5	4.7	9.8	2.3	4.8	6.6	8.6	11.2	1.6
Mean Jun. flow	cfs	0.9	0.3	1.6	0.9	0.2	5.4	9.5	0.7	3.6	0.2
Mean Jul. flow	cfs	0.3	0.2	1.4	4.1	0.5	0.4	3.9	0.3	3.7	0.2
Mean Aug. flow	cfs	0.2	0.5	0.2	1.2	1.2	0.2	1.1	0.3	0.8	10.2
Mean Sep. flow	cfs	0.4	2.1	0.4	4.1	2.2	0.2	6.7	1.2	0.7	4.7
Max. 1-day flow	cfs	50.4	20.3	36.1	36.4	45.2	49.1	34.7	45.4	42.4	36.3
Max. 3-day flow	cfs	44.0	16.0	28.4	32.0	40.4	45.8	28.1	38.8	33.6	29.1
Max. 7-day flow	cfs	37.6	11.2	25.8	27.2	39.5	37.4	21.8	31.7	29.8	22.5
Max. 30-day flow	cfs	19.0	6.1	15.8	16.2	19.9	21.5	11.5	17.9	14.3	12.8
Max. 90-day flow	cfs	7.8	2.5	9.7	9.5	11.0	10.4	9.4	10.3	9.0	5.1
Min. 1-day flow	cfs	0.04	0.04	0.06	0.06	0.05	0.04	0.07	0.04	0.06	0.05
Min. 3-day flow	cfs	0.05	0.04	0.06	0.08	0.05	0.05	0.07	0.04	0.07	0.05
Min. 7-day flow	cfs	0.06	0.05	0.06	0.14	0.05	0.05	0.07	0.05	0.12	0.05
Min. 30-day flow	cfs	0.14	0.15	0.10	0.54	0.14	0.06	0.26	0.23	0.42	0.10
Min. 90-day flow	cfs	0.28	0.21	0.56	0.68	0.35	0.15	0.57	0.40	0.40	0.36
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.025	0.029	0.018	0.032	0.012	0.012	0.017	0.012	0.038	0.020
Julian date of max. flow	day	25-Apr	1-Nov	23-Apr	17-Apr	7-Oct	15-Apr	23-Apr	2-Apr	23-May	25-Aug
Julian date of min. flow	day	26-Jul	8-Jul	16-Aug	2-Jul	18-Jun	25-Aug	15-Oct	15-Jul	10-Jul	24-Jul
Number of high pulses[1]	#/year	9	9	5	11	8	6	14	3	5	5
Number of low pulses[2]	#/year	16	17	11	6	8	6	1	12	9	10
Mean high pulse duration	days	5.2	5.9	22.2	11.7	15.8	18.5	9.9	34.0	16.2	14.8
Mean low pulse duration	days	13.0	6.9	6.5	3.5	9.0	12.2	21.0	5.0	5.3	16.1
Mean rate of flow increase[3]	cfs/day	2.0	1.1	2.2	3.8	2.6	1.9	3.0	1.6	2.2	1.3
Mean rate of flow decrease[4]	cfs/day	0.4	0.3	0.5	0.7	0.6	0.5	0.6	0.5	0.5	0.5
Number of flow reversals	#/year	64	79	89	69	76	65	70	87	63	74

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 1e: Summary of modeled hydrologic parameters for **Mine Year 15** at surface water monitoring location **SW-001**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	0.8	1.5	2.9	13.9	19.4	9.6	0.9	8.5	0.9	0.3
Mean Nov. flow	cfs	0.4	4.4	1.7	3.2	7.8	3.2	1.1	2.7	1.3	0.4
Mean Dec. flow	cfs	0.3	1.3	0.8	1.2	3.4	1.7	1.4	1.1	0.7	0.4
Mean Jan. flow	cfs	0.3	0.7	0.6	0.7	1.1	0.8	0.8	0.7	0.5	0.4
Mean Feb. flow	cfs	0.3	0.5	3.9	0.6	0.7	0.8	0.6	0.5	0.5	0.4
Mean Mar. flow	cfs	1.9	1.3	6.0	1.5	2.2	1.3	6.5	4.0	10.9	2.6
Mean Apr. flow	cfs	14.9	5.6	15.4	11.8	10.3	20.7	11.1	17.7	3.2	7.2
Mean May flow	cfs	6.1	0.5	4.7	9.8	2.3	4.8	6.6	8.6	11.2	1.6
Mean Jun. flow	cfs	0.9	0.3	1.6	0.9	0.2	5.4	9.5	0.7	3.6	0.2
Mean Jul. flow	cfs	0.3	0.2	1.4	4.1	0.5	0.4	3.9	0.3	3.7	0.2
Mean Aug. flow	cfs	0.2	0.5	0.2	1.2	1.2	0.2	1.1	0.3	0.8	10.2
Mean Sep. flow	cfs	0.4	2.1	0.4	4.1	2.2	0.2	6.7	1.2	0.7	4.7
Max. 1-day flow	cfs	50.4	20.2	36.1	36.4	45.2	49.1	34.7	45.4	42.4	36.3
Max. 3-day flow	cfs	44.0	16.0	28.4	32.0	40.3	45.8	28.1	38.8	33.6	29.1
Max. 7-day flow	cfs	37.6	11.2	25.8	27.2	39.5	37.4	21.8	31.7	29.8	22.5
Max. 30-day flow	cfs	19.0	6.1	15.8	16.2	19.9	21.5	11.5	17.9	14.3	12.8
Max. 90-day flow	cfs	7.8	2.5	9.7	9.5	11.0	10.4	9.4	10.3	9.0	5.1
Min. 1-day flow	cfs	0.04	0.04	0.06	0.06	0.05	0.04	0.07	0.04	0.06	0.05
Min. 3-day flow	cfs	0.05	0.04	0.06	0.08	0.05	0.05	0.07	0.04	0.07	0.05
Min. 7-day flow	cfs	0.06	0.05	0.06	0.14	0.05	0.05	0.07	0.05	0.12	0.05
Min. 30-day flow	cfs	0.14	0.15	0.10	0.54	0.14	0.06	0.26	0.23	0.42	0.10
Min. 90-day flow	cfs	0.28	0.21	0.56	0.68	0.35	0.15	0.57	0.40	0.40	0.36
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.025	0.029	0.018	0.032	0.012	0.012	0.017	0.012	0.038	0.020
Julian date of max. flow	day	25-Apr	1-Nov	23-Apr	17-Apr	7-Oct	15-Apr	23-Apr	2-Apr	23-May	25-Aug
Julian date of min. flow	day	26-Jul	8-Jul	16-Aug	2-Jul	18-Jun	25-Aug	15-Oct	15-Jul	10-Jul	24-Jul
Number of high pulses[1]	#/year	9	9	5	11	8	6	14	3	5	5
Number of low pulses[2]	#/year	16	17	11	6	8	6	1	12	9	10
Mean high pulse duration	days	5.2	5.9	22.2	11.7	15.8	18.5	9.9	34.0	16.2	14.8
Mean low pulse duration	days	13.0	6.9	6.5	3.5	9.0	12.2	21.0	5.0	5.3	16.1
Mean rate of flow increase[3]	cfs/day	2.0	1.1	2.2	3.8	2.6	1.9	3.0	1.6	2.2	1.3
Mean rate of flow decrease[4]	cfs/day	0.4	0.3	0.5	0.7	0.6	0.5	0.6	0.5	0.5	0.5
Number of flow reversals	#/year	64	79	89	69	76	65	70	87	63	74

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 1f: Summary of modeled hydrologic parameters for **Mine Year 20** at surface water monitoring location **SW-001**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	0.8	1.5	2.9	13.9	19.4	9.6	0.9	8.5	0.9	0.3
Mean Nov. flow	cfs	0.4	4.4	1.7	3.2	7.8	3.2	1.1	2.7	1.3	0.4
Mean Dec. flow	cfs	0.3	1.3	0.8	1.2	3.4	1.7	1.4	1.1	0.7	0.4
Mean Jan. flow	cfs	0.3	0.7	0.6	0.7	1.1	0.8	0.8	0.7	0.5	0.4
Mean Feb. flow	cfs	0.3	0.5	3.9	0.6	0.7	0.8	0.6	0.5	0.5	0.4
Mean Mar. flow	cfs	1.9	1.3	6.0	1.5	2.2	1.3	6.5	4.0	10.9	2.6
Mean Apr. flow	cfs	14.9	5.6	15.4	11.8	10.3	20.7	11.1	17.7	3.2	7.2
Mean May flow	cfs	6.1	0.5	4.7	9.8	2.3	4.8	6.6	8.6	11.2	1.6
Mean Jun. flow	cfs	0.9	0.3	1.6	0.9	0.2	5.4	9.5	0.7	3.6	0.2
Mean Jul. flow	cfs	0.3	0.2	1.4	4.1	0.5	0.4	3.9	0.3	3.7	0.2
Mean Aug. flow	cfs	0.2	0.5	0.2	1.2	1.2	0.2	1.1	0.3	0.8	10.2
Mean Sep. flow	cfs	0.4	2.1	0.4	4.1	2.2	0.2	6.7	1.2	0.7	4.7
Max. 1-day flow	cfs	50.4	20.3	36.1	36.4	45.2	49.1	34.7	45.4	42.4	36.3
Max. 3-day flow	cfs	44.0	16.0	28.4	32.0	40.3	45.8	28.1	38.8	33.6	29.1
Max. 7-day flow	cfs	37.6	11.2	25.8	27.2	39.5	37.4	21.8	31.7	29.8	22.5
Max. 30-day flow	cfs	19.0	6.1	15.8	16.2	19.9	21.5	11.5	17.9	14.3	12.8
Max. 90-day flow	cfs	7.8	2.5	9.7	9.5	11.0	10.4	9.4	10.3	9.0	5.1
Min. 1-day flow	cfs	0.04	0.04	0.06	0.06	0.05	0.04	0.07	0.04	0.06	0.05
Min. 3-day flow	cfs	0.05	0.04	0.06	0.08	0.05	0.05	0.07	0.04	0.07	0.05
Min. 7-day flow	cfs	0.06	0.05	0.06	0.14	0.05	0.05	0.07	0.05	0.12	0.05
Min. 30-day flow	cfs	0.14	0.15	0.10	0.54	0.14	0.06	0.26	0.23	0.42	0.10
Min. 90-day flow	cfs	0.28	0.21	0.56	0.68	0.35	0.15	0.57	0.40	0.40	0.36
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.025	0.029	0.018	0.032	0.012	0.012	0.017	0.012	0.038	0.020
Julian date of max. flow	day	25-Apr	1-Nov	23-Apr	17-Apr	7-Oct	15-Apr	23-Apr	2-Apr	23-May	25-Aug
Julian date of min. flow	day	26-Jul	8-Jul	16-Aug	2-Jul	18-Jun	25-Aug	15-Oct	15-Jul	10-Jul	24-Jul
Number of high pulses[1]	#/year	9	9	5	11	8	6	14	3	5	5
Number of low pulses[2]	#/year	16	17	11	6	8	6	1	12	9	10
Mean high pulse duration	days	5.2	5.9	22.2	11.7	15.8	18.5	9.9	34.0	16.2	14.8
Mean low pulse duration	days	13.0	6.9	6.5	3.5	9.0	12.2	21.0	5.0	5.3	16.1
Mean rate of flow increase[3]	cfs/day	2.0	1.1	2.2	3.8	2.6	1.9	3.0	1.6	2.2	1.3
Mean rate of flow decrease[4]	cfs/day	0.4	0.3	0.5	0.7	0.6	0.5	0.6	0.5	0.5	0.5
Number of flow reversals	#/year	64	79	89	69	76	65	70	87	63	74

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 1g: Summary of modeled hydrologic parameters for **Mine Facilities Off** at surface water monitoring location **SW-001**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	0.8	1.5	2.9	13.9	19.4	9.6	0.9	8.5	0.9	0.3
Mean Nov. flow	cfs	0.4	4.4	1.7	3.2	7.8	3.2	1.1	2.7	1.3	0.4
Mean Dec. flow	cfs	0.3	1.3	0.8	1.2	3.4	1.7	1.4	1.1	0.7	0.4
Mean Jan. flow	cfs	0.3	0.7	0.6	0.7	1.1	0.8	0.8	0.7	0.5	0.4
Mean Feb. flow	cfs	0.3	0.5	3.9	0.6	0.7	0.8	0.6	0.5	0.5	0.4
Mean Mar. flow	cfs	1.9	1.3	6.0	1.5	2.2	1.3	6.5	4.0	10.9	2.6
Mean Apr. flow	cfs	14.9	5.6	15.4	11.8	10.3	20.7	11.1	17.7	3.2	7.2
Mean May flow	cfs	6.1	0.5	4.7	9.8	2.3	4.8	6.6	8.6	11.2	1.6
Mean Jun. flow	cfs	0.9	0.3	1.6	0.9	0.2	5.4	9.5	0.7	3.6	0.2
Mean Jul. flow	cfs	0.3	0.2	1.4	4.1	0.5	0.4	3.9	0.3	3.7	0.2
Mean Aug. flow	cfs	0.2	0.5	0.2	1.2	1.2	0.2	1.1	0.3	0.8	10.2
Mean Sep. flow	cfs	0.4	2.1	0.4	4.1	2.2	0.2	6.7	1.2	0.7	4.7
Max. 1-day flow	cfs	50.4	20.3	36.1	36.4	45.2	49.1	34.7	45.4	42.4	36.3
Max. 3-day flow	cfs	44.0	16.0	28.4	32.0	40.4	45.8	28.1	38.8	33.6	29.1
Max. 7-day flow	cfs	37.6	11.2	25.8	27.2	39.5	37.4	21.8	31.7	29.8	22.5
Max. 30-day flow	cfs	19.0	6.1	15.8	16.2	19.9	21.5	11.5	17.9	14.3	12.8
Max. 90-day flow	cfs	7.8	2.5	9.7	9.5	11.0	10.4	9.4	10.3	9.0	5.1
Min. 1-day flow	cfs	0.04	0.04	0.06	0.06	0.05	0.04	0.07	0.04	0.06	0.05
Min. 3-day flow	cfs	0.05	0.04	0.06	0.08	0.05	0.05	0.07	0.04	0.07	0.05
Min. 7-day flow	cfs	0.06	0.05	0.06	0.14	0.05	0.05	0.07	0.05	0.12	0.05
Min. 30-day flow	cfs	0.14	0.15	0.10	0.54	0.14	0.06	0.26	0.23	0.42	0.10
Min. 90-day flow	cfs	0.28	0.21	0.56	0.68	0.35	0.15	0.57	0.40	0.40	0.36
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.025	0.029	0.018	0.032	0.012	0.012	0.017	0.012	0.038	0.020
Julian date of max. flow	day	25-Apr	1-Nov	23-Apr	17-Apr	7-Oct	15-Apr	23-Apr	2-Apr	23-May	25-Aug
Julian date of min. flow	day	26-Jul	8-Jul	16-Aug	2-Jul	18-Jun	25-Aug	15-Oct	15-Jul	10-Jul	24-Jul
Number of high pulses[1]	#/year	9	9	5	11	8	6	14	3	5	5
Number of low pulses[2]	#/year	16	17	11	6	8	6	1	12	9	10
Mean high pulse duration	days	5.2	5.9	22.2	11.7	15.8	18.5	9.9	34.0	16.2	14.8
Mean low pulse duration	days	13.0	6.9	6.5	3.5	9.0	12.2	21.0	5.0	5.3	16.1
Mean rate of flow increase[3]	cfs/day	2.0	1.1	2.2	3.8	2.6	1.9	3.0	1.6	2.2	1.3
Mean rate of flow decrease[4]	cfs/day	0.4	0.3	0.5	0.7	0.6	0.5	0.6	0.5	0.5	0.5
Number of flow reversals	#/year	64	79	89	69	76	65	70	87	63	74

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 2a: Summary of modeled hydrologic parameters for **Existing Conditions** at surface water monitoring location **SW-002**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.4	3.4	6.9	35.0	42.9	22.5	2.3	17.3	2.8	0.6
Mean Nov. flow	cfs	0.9	13.1	2.6	4.7	16.3	6.1	2.5	4.8	3.2	1.5
Mean Dec. flow	cfs	0.7	2.1	1.4	1.9	5.6	2.5	3.8	1.7	1.3	0.9
Mean Jan. flow	cfs	0.7	1.2	1.2	1.3	1.8	1.4	1.4	1.2	1.0	0.8
Mean Feb. flow	cfs	0.7	1.0	10.9	1.1	1.2	1.6	1.1	1.0	1.0	0.9
Mean Mar. flow	cfs	7.7	3.0	12.9	3.5	5.4	3.2	17.5	9.7	26.8	7.2
Mean Apr. flow	cfs	42.0	12.9	35.2	28.2	24.1	47.4	24.3	39.8	4.8	15.1
Mean May flow	cfs	9.5	0.8	7.3	19.0	3.4	7.7	10.7	16.4	27.5	3.2
Mean Jun. flow	cfs	2.5	0.9	4.5	1.4	0.5	12.8	24.2	1.3	5.1	0.6
Mean Jul. flow	cfs	0.8	0.6	3.7	11.7	1.4	0.7	5.9	1.0	10.5	0.5
Mean Aug. flow	cfs	0.7	1.5	0.6	3.8	4.2	0.7	2.7	0.9	1.3	25.2
Mean Sep. flow	cfs	1.2	5.9	0.9	10.0	5.1	0.6	17.7	3.5	2.4	7.8
Max. 1-day flow	cfs	142.5	72.8	87.4	117.0	125.6	133.8	99.6	128.7	122.1	94.5
Max. 3-day flow	cfs	133.4	60.6	74.2	104.9	116.9	119.8	81.4	116.7	101.5	79.9
Max. 7-day flow	cfs	125.5	40.2	63.5	77.1	101.4	96.2	57.1	88.7	88.7	54.5
Max. 30-day flow	cfs	48.8	15.2	35.3	37.9	44.1	49.1	25.3	39.8	32.4	29.5
Max. 90-day flow	cfs	20.0	6.4	21.6	21.4	24.2	23.1	20.9	22.3	20.7	11.4
Min. 1-day flow	cfs	0.26	0.22	0.28	0.32	0.24	0.25	0.31	0.22	0.28	0.23
Min. 3-day flow	cfs	0.28	0.22	0.30	0.36	0.25	0.25	0.31	0.23	0.30	0.23
Min. 7-day flow	cfs	0.28	0.24	0.31	0.46	0.25	0.26	0.31	0.28	0.39	0.25
Min. 30-day flow	cfs	0.54	0.52	0.44	1.03	0.44	0.30	0.67	0.80	0.93	0.43
Min. 90-day flow	cfs	0.71	0.68	1.33	1.22	0.97	0.54	1.38	1.04	1.12	0.86
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.048	0.063	0.042	0.045	0.026	0.029	0.033	0.034	0.052	0.046
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	9-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	6	4	9	7	4	12	3	6	6
Number of low pulses[2]	#/year	14	14	8	3	8	6	3	11	9	9
Mean high pulse duration	days	12.5	9.0	25.0	13.9	18.6	25.3	11.3	31.7	14.0	12.2
Mean low pulse duration	days	14.8	6.7	8.3	7.3	8.5	11.8	9.3	4.4	5.8	20.1
Mean rate of flow increase[3]	cfs/day	3.3	2.3	3.0	6.1	4.1	3.9	6.3	3.6	5.0	2.8
Mean rate of flow decrease[4]	cfs/day	1.0	0.7	1.1	1.6	1.3	1.2	1.8	1.3	1.4	0.9
Number of flow reversals	#/year	59	62	77	56	60	55	55	71	60	64

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 2b: Summary of modeled hydrologic parameters for **Mine Year 1** at surface water monitoring location **SW-002**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.3	3.2	6.7	33.8	41.6	21.8	2.2	16.8	2.6	0.6
Mean Nov. flow	cfs	0.8	12.6	2.6	4.6	15.9	5.9	2.5	4.7	3.1	1.4
Mean Dec. flow	cfs	0.7	2.0	1.4	1.8	5.5	2.4	3.6	1.7	1.2	0.8
Mean Jan. flow	cfs	0.7	1.2	1.1	1.2	1.7	1.3	1.4	1.1	1.0	0.8
Mean Feb. flow	cfs	0.7	1.0	10.5	1.0	1.2	1.5	1.0	1.0	1.0	0.8
Mean Mar. flow	cfs	7.4	2.9	12.5	3.4	5.2	3.1	16.8	9.3	25.9	7.0
Mean Apr. flow	cfs	40.3	12.5	34.0	27.3	23.4	45.9	23.6	38.6	4.7	14.7
Mean May flow	cfs	9.4	0.8	7.2	18.5	3.4	7.6	10.5	16.0	26.6	3.1
Mean Jun. flow	cfs	2.4	0.8	4.3	1.4	0.4	12.4	23.3	1.2	5.0	0.6
Mean Jul. flow	cfs	0.8	0.6	3.6	11.0	1.3	0.7	5.8	0.9	10.1	0.5
Mean Aug. flow	cfs	0.6	1.4	0.5	3.4	4.1	0.7	2.6	0.8	1.2	24.7
Mean Sep. flow	cfs	1.1	5.6	0.9	9.6	5.0	0.6	17.1	3.3	2.3	7.9
Max. 1-day flow	cfs	135.9	69.3	84.4	111.9	120.4	129.0	95.5	123.4	116.7	91.0
Max. 3-day flow	cfs	126.9	57.7	71.4	100.4	112.3	115.4	78.1	111.8	97.3	77.0
Max. 7-day flow	cfs	119.8	38.3	61.3	74.0	97.7	92.6	55.0	85.2	85.1	52.8
Max. 30-day flow	cfs	47.1	14.6	34.2	36.6	42.8	47.6	24.4	38.6	31.5	28.9
Max. 90-day flow	cfs	19.3	6.1	20.9	20.7	23.5	22.4	20.3	21.7	20.1	11.2
Min. 1-day flow	cfs	0.25	0.21	0.27	0.30	0.24	0.24	0.30	0.22	0.27	0.22
Min. 3-day flow	cfs	0.27	0.21	0.29	0.35	0.24	0.24	0.30	0.22	0.29	0.22
Min. 7-day flow	cfs	0.27	0.23	0.29	0.44	0.24	0.25	0.30	0.27	0.37	0.24
Min. 30-day flow	cfs	0.52	0.50	0.43	0.99	0.43	0.29	0.65	0.77	0.90	0.42
Min. 90-day flow	cfs	0.68	0.66	1.28	1.17	0.93	0.52	1.33	1.00	1.08	0.83
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.048	0.063	0.042	0.045	0.026	0.029	0.033	0.033	0.052	0.046
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	9-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	6	4	8	7	5	12	3	6	6
Number of low pulses[2]	#/year	14	14	8	3	7	6	3	11	9	10
Mean high pulse duration	days	12.3	8.8	25.0	15.5	18.6	20.8	11.3	31.3	13.8	12.5
Mean low pulse duration	days	14.8	6.7	8.4	7.7	9.7	11.8	9.3	4.4	5.9	18.0
Mean rate of flow increase[3]	cfs/day	3.2	2.3	2.9	5.9	4.0	3.8	6.1	3.5	4.9	2.7
Mean rate of flow decrease[4]	cfs/day	0.9	0.6	1.0	1.5	1.2	1.2	1.8	1.2	1.3	0.9
Number of flow reversals	#/year	59	64	77	58	60	55	54	71	60	69

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 2c: Summary of modeled hydrologic parameters for **Mine Year 5** at surface water monitoring location **SW-002**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.2	3.2	6.7	33.7	41.5	21.7	2.2	16.7	2.6	0.6
Mean Nov. flow	cfs	0.8	12.5	2.5	4.6	15.8	5.8	2.4	4.6	3.0	1.3
Mean Dec. flow	cfs	0.7	2.0	1.3	1.8	5.5	2.4	3.6	1.6	1.2	0.8
Mean Jan. flow	cfs	0.6	1.2	1.1	1.2	1.7	1.3	1.3	1.1	1.0	0.8
Mean Feb. flow	cfs	0.6	0.9	10.4	1.0	1.1	1.5	1.0	0.9	1.0	0.8
Mean Mar. flow	cfs	7.3	2.9	12.4	3.4	5.2	3.1	16.7	9.3	25.7	6.9
Mean Apr. flow	cfs	40.1	12.4	33.9	27.2	23.2	45.7	23.5	38.5	4.7	14.6
Mean May flow	cfs	9.4	0.8	7.1	18.4	3.3	7.5	10.4	15.9	26.4	3.1
Mean Jun. flow	cfs	2.4	0.8	4.3	1.3	0.4	12.3	23.2	1.2	5.0	0.6
Mean Jul. flow	cfs	0.7	0.6	3.5	11.0	1.3	0.7	5.8	0.9	10.0	0.5
Mean Aug. flow	cfs	0.6	1.4	0.5	3.4	4.1	0.7	2.6	0.8	1.2	24.3
Mean Sep. flow	cfs	1.1	5.6	0.8	9.6	5.0	0.5	17.0	3.3	2.2	7.6
Max. 1-day flow	cfs	135.1	68.7	84.0	111.0	119.4	128.4	94.9	122.8	115.7	90.5
Max. 3-day flow	cfs	126.0	57.4	70.9	99.7	111.6	114.7	77.6	111.1	96.7	76.5
Max. 7-day flow	cfs	119.1	38.1	61.0	73.6	97.2	92.1	54.7	84.7	84.7	52.5
Max. 30-day flow	cfs	46.9	14.5	34.1	36.4	42.6	47.3	24.3	38.5	31.3	28.5
Max. 90-day flow	cfs	19.2	6.1	20.8	20.7	23.4	22.3	20.2	21.6	20.0	11.0
Min. 1-day flow	cfs	0.24	0.20	0.26	0.29	0.23	0.23	0.28	0.21	0.26	0.21
Min. 3-day flow	cfs	0.26	0.20	0.28	0.33	0.23	0.23	0.28	0.21	0.28	0.21
Min. 7-day flow	cfs	0.26	0.22	0.28	0.42	0.23	0.24	0.29	0.26	0.36	0.23
Min. 30-day flow	cfs	0.50	0.48	0.41	0.95	0.41	0.27	0.62	0.74	0.86	0.40
Min. 90-day flow	cfs	0.65	0.63	1.23	1.12	0.89	0.50	1.27	0.96	1.04	0.80
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.047	0.061	0.040	0.043	0.025	0.028	0.031	0.032	0.050	0.044
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	9-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	6	4	8	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	14	8	3	7	6	3	11	9	10
Mean high pulse duration	days	12.3	9.0	25.0	15.4	16.4	20.8	11.3	31.3	13.8	12.3
Mean low pulse duration	days	14.9	6.7	8.4	7.7	9.7	11.8	9.3	4.4	5.9	18.0
Mean rate of flow increase[3]	cfs/day	3.2	2.3	2.9	5.9	4.0	3.8	6.1	3.5	4.9	2.7
Mean rate of flow decrease[4]	cfs/day	0.9	0.6	1.0	1.5	1.2	1.2	1.7	1.2	1.3	0.9
Number of flow reversals	#/year	59	64	77	58	60	55	55	71	60	67

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 2d: Summary of modeled hydrologic parameters for **Mine Year 10** at surface water monitoring location **SW-002**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.2	3.1	6.7	33.5	41.3	21.6	2.2	16.6	2.5	0.5
Mean Nov. flow	cfs	0.8	12.4	2.5	4.6	15.7	5.8	2.3	4.6	3.0	1.3
Mean Dec. flow	cfs	0.6	1.9	1.3	1.8	5.4	2.3	3.5	1.6	1.1	0.7
Mean Jan. flow	cfs	0.6	1.1	1.0	1.1	1.6	1.3	1.3	1.1	0.9	0.7
Mean Feb. flow	cfs	0.6	0.9	10.3	0.9	1.1	1.4	0.9	0.9	0.9	0.7
Mean Mar. flow	cfs	7.2	2.8	12.3	3.3	5.1	3.0	16.6	9.2	25.6	6.8
Mean Apr. flow	cfs	39.9	12.3	33.7	27.0	23.1	45.5	23.4	38.3	4.6	14.6
Mean May flow	cfs	9.3	0.7	7.1	18.3	3.3	7.5	10.4	15.9	26.3	3.0
Mean Jun. flow	cfs	2.3	0.8	4.2	1.3	0.4	12.2	23.1	1.2	4.9	0.5
Mean Jul. flow	cfs	0.7	0.5	3.5	10.9	1.2	0.6	5.7	0.8	9.9	0.5
Mean Aug. flow	cfs	0.6	1.3	0.5	3.3	4.0	0.6	2.5	0.8	1.1	24.2
Mean Sep. flow	cfs	1.1	5.5	0.8	9.6	4.9	0.5	16.9	3.2	2.0	7.6
Max. 1-day flow	cfs	134.2	68.0	83.3	110.3	118.7	127.7	94.2	122.1	114.8	89.9
Max. 3-day flow	cfs	125.2	56.9	70.5	99.1	111.0	114.2	77.1	110.5	96.0	76.1
Max. 7-day flow	cfs	118.3	37.8	60.7	73.2	96.7	91.7	54.4	84.2	84.2	52.3
Max. 30-day flow	cfs	46.6	14.4	33.9	36.2	42.5	47.1	24.1	38.3	31.1	28.4
Max. 90-day flow	cfs	19.1	6.0	20.7	20.6	23.3	22.1	20.1	21.5	19.9	10.9
Min. 1-day flow	cfs	0.23	0.19	0.25	0.28	0.21	0.22	0.27	0.20	0.24	0.20
Min. 3-day flow	cfs	0.24	0.19	0.26	0.32	0.21	0.22	0.27	0.20	0.26	0.20
Min. 7-day flow	cfs	0.24	0.21	0.27	0.40	0.22	0.23	0.27	0.24	0.34	0.22
Min. 30-day flow	cfs	0.47	0.45	0.39	0.90	0.39	0.26	0.59	0.70	0.81	0.38
Min. 90-day flow	cfs	0.62	0.60	1.16	1.06	0.85	0.47	1.21	0.91	0.98	0.75
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.045	0.059	0.039	0.041	0.024	0.027	0.030	0.031	0.048	0.043
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	9-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	6	4	8	7	5	12	3	6	6
Number of low pulses[2]	#/year	14	14	8	3	7	6	3	11	10	10
Mean high pulse duration	days	12.3	9.0	25.0	15.4	18.9	20.8	11.3	31.3	13.7	12.3
Mean low pulse duration	days	14.9	6.7	8.4	7.7	9.7	11.8	9.3	4.4	5.3	18.0
Mean rate of flow increase[3]	cfs/day	3.2	2.2	2.9	5.8	4.0	3.8	6.0	3.4	4.9	2.6
Mean rate of flow decrease[4]	cfs/day	0.9	0.6	1.0	1.5	1.2	1.1	1.7	1.2	1.3	0.9
Number of flow reversals	#/year	59	64	77	58	60	55	55	71	60	69

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 2e: Summary of modeled hydrologic parameters for **Mine Year 15** at surface water monitoring location **SW-002**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.1	3.1	6.6	33.5	41.3	21.6	2.1	16.6	2.5	0.5
Mean Nov. flow	cfs	0.8	12.4	2.5	4.5	15.7	5.8	2.3	4.6	3.0	1.3
Mean Dec. flow	cfs	0.6	1.9	1.3	1.8	5.4	2.3	3.5	1.6	1.1	0.7
Mean Jan. flow	cfs	0.6	1.1	1.0	1.1	1.6	1.2	1.3	1.1	0.9	0.7
Mean Feb. flow	cfs	0.6	0.9	10.3	0.9	1.1	1.4	0.9	0.9	0.9	0.7
Mean Mar. flow	cfs	7.2	2.8	12.3	3.3	5.1	3.0	16.6	9.2	25.6	6.8
Mean Apr. flow	cfs	39.9	12.3	33.7	27.0	23.1	45.5	23.4	38.3	4.6	14.5
Mean May flow	cfs	9.3	0.7	7.1	18.3	3.3	7.5	10.3	15.9	26.3	3.0
Mean Jun. flow	cfs	2.3	0.8	4.2	1.3	0.4	12.2	23.1	1.1	4.9	0.5
Mean Jul. flow	cfs	0.7	0.5	3.4	10.9	1.2	0.6	5.7	0.8	9.9	0.5
Mean Aug. flow	cfs	0.6	1.3	0.5	3.3	4.0	0.6	2.5	0.8	1.1	24.2
Mean Sep. flow	cfs	1.0	5.5	0.8	9.6	4.9	0.5	16.8	3.2	2.0	7.6
Max. 1-day flow	cfs	134.2	68.0	83.4	110.3	118.7	127.7	94.1	122.2	114.8	89.9
Max. 3-day flow	cfs	125.2	56.9	70.5	99.1	111.0	114.2	77.1	110.5	96.0	76.1
Max. 7-day flow	cfs	118.4	37.8	60.7	73.2	96.7	91.7	54.4	84.2	84.2	52.3
Max. 30-day flow	cfs	46.6	14.4	33.9	36.2	42.5	47.1	24.1	38.3	31.1	28.4
Max. 90-day flow	cfs	19.1	6.0	20.7	20.6	23.3	22.1	20.1	21.5	19.9	10.9
Min. 1-day flow	cfs	0.23	0.19	0.25	0.27	0.21	0.22	0.26	0.19	0.24	0.20
Min. 3-day flow	cfs	0.24	0.19	0.26	0.31	0.21	0.22	0.27	0.20	0.26	0.20
Min. 7-day flow	cfs	0.24	0.21	0.26	0.39	0.21	0.22	0.27	0.24	0.33	0.21
Min. 30-day flow	cfs	0.47	0.44	0.38	0.89	0.38	0.26	0.58	0.69	0.80	0.37
Min. 90-day flow	cfs	0.61	0.59	1.14	1.05	0.83	0.46	1.19	0.90	0.97	0.74
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.044	0.058	0.038	0.041	0.024	0.026	0.030	0.030	0.048	0.042
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	9-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	6	4	8	7	5	12	3	6	6
Number of low pulses[2]	#/year	14	14	8	3	7	6	3	11	10	10
Mean high pulse duration	days	12.3	9.0	25.0	15.4	18.9	20.8	11.3	31.3	13.7	12.3
Mean low pulse duration	days	14.9	6.7	8.4	7.7	9.7	11.8	9.3	4.4	5.3	18.0
Mean rate of flow increase[3]	cfs/day	3.2	2.2	2.9	5.8	4.0	3.8	6.0	3.4	4.9	2.6
Mean rate of flow decrease[4]	cfs/day	0.9	0.6	1.0	1.5	1.2	1.1	1.7	1.2	1.3	0.9
Number of flow reversals	#/year	59	64	77	58	60	55	55	71	60	69

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 2f: Summary of modeled hydrologic parameters for **Mine Year 20** at surface water monitoring location **SW-002**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.1	3.1	6.6	33.4	41.2	21.5	2.1	16.6	2.5	0.5
Mean Nov. flow	cfs	0.7	12.3	2.4	4.5	15.6	5.7	2.3	4.5	2.9	1.2
Mean Dec. flow	cfs	0.6	1.9	1.2	1.7	5.3	2.3	3.4	1.5	1.1	0.7
Mean Jan. flow	cfs	0.5	1.0	1.0	1.0	1.6	1.2	1.2	1.0	0.8	0.6
Mean Feb. flow	cfs	0.5	0.8	10.3	0.9	1.0	1.4	0.9	0.8	0.8	0.7
Mean Mar. flow	cfs	7.1	2.8	12.2	3.3	5.0	2.9	16.5	9.1	25.5	6.8
Mean Apr. flow	cfs	39.8	12.2	33.6	27.0	23.0	45.4	23.3	38.2	4.6	14.5
Mean May flow	cfs	9.3	0.6	7.0	18.3	3.2	7.4	10.3	15.8	26.2	3.0
Mean Jun. flow	cfs	2.3	0.7	4.2	1.2	0.4	12.1	23.0	1.1	4.9	0.5
Mean Jul. flow	cfs	0.6	0.5	3.4	10.8	1.1	0.6	5.6	0.8	9.9	0.4
Mean Aug. flow	cfs	0.5	1.3	0.4	3.2	3.9	0.6	2.4	0.7	1.1	24.1
Mean Sep. flow	cfs	1.0	5.4	0.7	9.5	4.8	0.5	16.8	3.1	2.0	7.5
Max. 1-day flow	cfs	134.1	67.9	83.3	110.2	118.7	127.6	94.1	122.1	114.8	89.9
Max. 3-day flow	cfs	125.2	56.8	70.4	99.0	111.0	114.1	77.0	110.4	95.9	76.0
Max. 7-day flow	cfs	118.3	37.7	60.7	73.2	96.6	91.6	54.3	84.2	84.1	52.2
Max. 30-day flow	cfs	46.6	14.3	33.8	36.2	42.4	47.1	24.1	38.2	31.1	28.3
Max. 90-day flow	cfs	19.0	5.9	20.6	20.5	23.2	22.1	20.0	21.4	19.8	10.8
Min. 1-day flow	cfs	0.21	0.17	0.22	0.25	0.19	0.20	0.24	0.17	0.22	0.18
Min. 3-day flow	cfs	0.22	0.17	0.24	0.28	0.19	0.20	0.24	0.18	0.23	0.18
Min. 7-day flow	cfs	0.22	0.19	0.24	0.36	0.19	0.20	0.24	0.22	0.30	0.19
Min. 30-day flow	cfs	0.42	0.40	0.35	0.81	0.35	0.23	0.53	0.62	0.73	0.34
Min. 90-day flow	cfs	0.55	0.53	1.04	0.95	0.76	0.42	1.08	0.82	0.88	0.67
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.041	0.054	0.035	0.037	0.022	0.024	0.027	0.028	0.044	0.039
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	9-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	6	4	8	7	5	12	3	6	6
Number of low pulses[2]	#/year	14	14	8	3	7	6	3	11	10	10
Mean high pulse duration	days	12.3	9.0	25.0	15.4	18.9	20.8	11.3	31.3	13.7	12.3
Mean low pulse duration	days	14.9	6.7	8.4	7.7	9.7	11.8	9.3	4.4	5.3	18.0
Mean rate of flow increase[3]	cfs/day	3.2	2.2	2.9	5.8	4.0	3.8	6.0	3.4	4.9	2.6
Mean rate of flow decrease[4]	cfs/day	0.9	0.6	1.0	1.5	1.2	1.1	1.7	1.2	1.3	0.9
Number of flow reversals	#/year	59	64	77	58	60	55	55	71	60	69

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 2g: Summary of modeled hydrologic parameters for **Mine Facilities off** at surface water monitoring location **SW-002**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.1	3.1	6.6	33.4	41.2	21.5	2.1	16.6	2.5	0.5
Mean Nov. flow	cfs	0.7	12.3	2.4	4.5	15.6	5.7	2.3	4.5	2.9	1.2
Mean Dec. flow	cfs	0.6	1.9	1.2	1.7	5.3	2.3	3.4	1.5	1.1	0.7
Mean Jan. flow	cfs	0.5	1.0	1.0	1.1	1.6	1.2	1.2	1.0	0.8	0.6
Mean Feb. flow	cfs	0.5	0.8	10.3	0.9	1.0	1.4	0.9	0.8	0.8	0.7
Mean Mar. flow	cfs	7.1	2.8	12.2	3.3	5.0	2.9	16.5	9.1	25.5	6.8
Mean Apr. flow	cfs	39.8	12.2	33.7	27.0	23.0	45.4	23.3	38.2	4.6	14.5
Mean May flow	cfs	9.3	0.6	7.0	18.3	3.2	7.4	10.3	15.8	26.2	3.0
Mean Jun. flow	cfs	2.3	0.7	4.2	1.2	0.4	12.1	23.0	1.1	4.9	0.5
Mean Jul. flow	cfs	0.6	0.5	3.4	10.8	1.1	0.6	5.6	0.8	9.9	0.4
Mean Aug. flow	cfs	0.5	1.3	0.4	3.2	3.9	0.6	2.4	0.7	1.1	24.1
Mean Sep. flow	cfs	1.0	5.4	0.7	9.5	4.8	0.5	16.8	3.1	2.0	7.5
Max. 1-day flow	cfs	134.1	67.9	83.3	110.2	118.7	127.7	94.1	122.1	114.8	89.9
Max. 3-day flow	cfs	125.2	56.8	70.4	99.0	111.0	114.1	77.0	110.4	96.0	76.0
Max. 7-day flow	cfs	118.3	37.7	60.7	73.2	96.6	91.6	54.3	84.2	84.1	52.2
Max. 30-day flow	cfs	46.6	14.3	33.8	36.2	42.4	47.1	24.1	38.2	31.1	28.3
Max. 90-day flow	cfs	19.0	5.9	20.6	20.5	23.2	22.1	20.0	21.4	19.8	10.8
Min. 1-day flow	cfs	0.21	0.17	0.22	0.25	0.19	0.20	0.24	0.17	0.22	0.18
Min. 3-day flow	cfs	0.22	0.17	0.24	0.28	0.19	0.20	0.24	0.18	0.23	0.18
Min. 7-day flow	cfs	0.22	0.19	0.24	0.36	0.19	0.20	0.24	0.22	0.30	0.19
Min. 30-day flow	cfs	0.42	0.40	0.35	0.81	0.35	0.23	0.53	0.62	0.73	0.34
Min. 90-day flow	cfs	0.55	0.53	1.04	0.95	0.76	0.42	1.08	0.82	0.88	0.67
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.041	0.054	0.035	0.037	0.022	0.024	0.027	0.028	0.044	0.039
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	9-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	6	4	8	7	5	12	3	6	6
Number of low pulses[2]	#/year	14	14	8	3	7	6	3	11	10	10
Mean high pulse duration	days	12.3	9.0	25.0	15.4	18.9	20.8	11.3	31.3	13.7	12.3
Mean low pulse duration	days	14.9	6.7	8.4	7.7	9.7	11.8	9.3	4.4	5.3	18.0
Mean rate of flow increase[3]	cfs/day	3.2	2.2	2.9	5.8	4.0	3.8	6.0	3.4	4.9	2.6
Mean rate of flow decrease[4]	cfs/day	0.9	0.6	1.0	1.5	1.2	1.1	1.7	1.2	1.3	0.9
Number of flow reversals	#/year	59	64	77	58	60	55	55	71	60	69

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 3a: Summary of modeled hydrologic parameters for **Existing Conditions** at surface water monitoring location **SW-003**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.8	4.9	8.1	40.9	49.3	26.2	2.5	19.4	3.4	0.7
Mean Nov. flow	cfs	1.0	14.6	2.8	4.9	18.5	6.7	3.0	5.3	3.7	1.8
Mean Dec. flow	cfs	0.9	2.2	1.5	2.0	6.0	2.6	4.4	1.9	1.4	1.0
Mean Jan. flow	cfs	0.8	1.4	1.3	1.4	1.9	1.5	1.6	1.3	1.2	1.0
Mean Feb. flow	cfs	0.8	1.1	13.1	1.2	1.4	1.8	1.2	1.1	1.2	1.0
Mean Mar. flow	cfs	9.5	3.9	15.0	4.4	6.3	3.8	20.8	12.4	31.3	8.7
Mean Apr. flow	cfs	50.4	14.5	40.6	32.7	28.2	55.2	27.9	45.8	4.9	17.2
Mean May flow	cfs	10.2	0.9	7.6	21.7	3.6	8.1	12.8	17.7	32.5	3.7
Mean Jun. flow	cfs	3.1	1.1	5.8	1.5	0.6	15.3	28.1	1.5	5.4	0.7
Mean Jul. flow	cfs	1.0	0.7	4.0	14.1	1.5	0.8	6.2	1.2	12.7	0.7
Mean Aug. flow	cfs	0.8	1.7	0.7	4.5	5.2	0.8	3.2	1.1	1.4	29.8
Mean Sep. flow	cfs	1.4	7.2	1.2	11.8	6.1	0.7	21.3	4.0	2.8	8.5
Max. 1-day flow	cfs	181.3	84.9	99.4	136.2	147.5	157.2	116.5	158.9	141.0	106.9
Max. 3-day flow	cfs	164.6	75.0	86.3	124.3	136.1	139.1	99.5	140.0	122.2	93.3
Max. 7-day flow	cfs	152.8	50.0	74.1	91.8	118.4	112.8	67.5	104.9	105.5	64.4
Max. 30-day flow	cfs	57.6	17.9	40.8	44.3	50.7	56.7	29.9	45.8	37.8	34.4
Max. 90-day flow	cfs	23.7	7.4	24.9	24.7	27.8	26.8	24.3	25.7	24.0	13.2
Min. 1-day flow	cfs	0.35	0.29	0.37	0.42	0.32	0.33	0.39	0.29	0.36	0.30
Min. 3-day flow	cfs	0.37	0.29	0.39	0.46	0.32	0.33	0.39	0.30	0.38	0.30
Min. 7-day flow	cfs	0.37	0.31	0.40	0.55	0.33	0.34	0.39	0.36	0.49	0.33
Min. 30-day flow	cfs	0.66	0.63	0.56	1.16	0.55	0.38	0.81	0.95	1.06	0.55
Min. 90-day flow	cfs	0.83	0.81	1.50	1.35	1.12	0.65	1.54	1.21	1.25	0.99
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.054	0.070	0.048	0.047	0.030	0.033	0.035	0.038	0.057	0.053
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	5	8	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	15	7	4	6	6	3	11	9	9
Mean high pulse duration	days	12.8	10.6	20.2	15.5	16.5	20.0	11.2	31.0	13.8	12.7
Mean low pulse duration	days	14.8	6.1	9.4	6.0	11.3	11.7	9.0	4.2	5.8	20.1
Mean rate of flow increase[3]	cfs/day	4.1	2.9	3.8	7.5	5.3	5.1	8.0	4.4	5.6	3.3
Mean rate of flow decrease[4]	cfs/day	1.2	0.8	1.3	1.9	1.6	1.4	2.2	1.6	1.6	1.1
Number of flow reversals	#/year	61	64	75	53	58	51	55	73	62	61

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 3b: Summary of modeled hydrologic parameters for **Mine Year 1** at surface water monitoring location **SW-003**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.8	4.7	7.9	39.8	48.2	25.5	2.5	18.9	3.3	0.7
Mean Nov. flow	cfs	1.0	14.1	2.7	4.8	18.1	6.6	2.9	5.2	3.6	1.7
Mean Dec. flow	cfs	0.8	2.2	1.5	2.0	5.9	2.6	4.2	1.8	1.4	1.0
Mean Jan. flow	cfs	0.8	1.3	1.3	1.3	1.9	1.5	1.5	1.3	1.1	0.9
Mean Feb. flow	cfs	0.8	1.1	12.7	1.1	1.3	1.7	1.1	1.1	1.1	0.9
Mean Mar. flow	cfs	9.2	3.8	14.6	4.3	6.1	3.7	20.2	12.1	30.5	8.4
Mean Apr. flow	cfs	48.9	14.1	39.6	31.8	27.5	53.8	27.3	44.7	4.9	16.8
Mean May flow	cfs	10.2	0.9	7.5	21.3	3.5	8.0	12.7	17.4	31.7	3.7
Mean Jun. flow	cfs	3.0	1.0	5.7	1.5	0.6	14.9	27.2	1.4	5.4	0.7
Mean Jul. flow	cfs	0.9	0.7	3.9	13.5	1.5	0.8	6.1	1.1	12.4	0.6
Mean Aug. flow	cfs	0.8	1.6	0.7	4.2	4.9	0.8	3.2	1.0	1.3	29.5
Mean Sep. flow	cfs	1.3	6.9	1.1	11.5	6.0	0.7	20.8	3.7	2.7	8.6
Max. 1-day flow	cfs	175.1	81.1	95.8	130.7	143.0	152.4	111.9	154.1	134.8	102.7
Max. 3-day flow	cfs	158.4	72.2	84.0	119.8	131.3	134.6	96.3	135.3	117.8	90.4
Max. 7-day flow	cfs	147.4	48.3	72.0	88.9	115.0	109.4	65.5	101.6	102.0	62.9
Max. 30-day flow	cfs	56.1	17.4	39.8	43.1	49.5	55.3	29.0	44.7	36.9	34.0
Max. 90-day flow	cfs	23.1	7.2	24.3	24.1	27.1	26.1	23.7	25.1	23.4	13.1
Min. 1-day flow	cfs	0.34	0.28	0.36	0.40	0.31	0.32	0.37	0.28	0.35	0.29
Min. 3-day flow	cfs	0.35	0.28	0.38	0.45	0.31	0.32	0.38	0.29	0.37	0.29
Min. 7-day flow	cfs	0.36	0.31	0.39	0.54	0.32	0.33	0.38	0.35	0.48	0.32
Min. 30-day flow	cfs	0.64	0.61	0.54	1.12	0.53	0.37	0.79	0.92	1.03	0.53
Min. 90-day flow	cfs	0.81	0.79	1.46	1.31	1.09	0.63	1.49	1.17	1.21	0.96
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.054	0.070	0.048	0.047	0.030	0.033	0.035	0.038	0.057	0.052
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	5	7	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	15	7	3	6	6	3	11	8	9
Mean high pulse duration	days	12.8	10.4	20.4	17.4	16.6	20.0	11.3	31.0	13.8	12.7
Mean low pulse duration	days	14.8	6.2	9.4	7.7	11.5	11.7	9.0	4.2	6.6	20.0
Mean rate of flow increase[3]	cfs/day	4.1	2.8	3.7	7.5	5.1	4.9	7.7	4.3	5.6	3.3
Mean rate of flow decrease[4]	cfs/day	1.2	0.7	1.2	1.8	1.5	1.4	2.1	1.5	1.5	1.1
Number of flow reversals	#/year	61	64	75	56	58	51	55	73	62	61

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 3c: Summary of modeled hydrologic parameters for **Mine Year 5** at surface water monitoring location **SW-003**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.7	4.6	7.9	39.5	47.9	25.4	2.4	18.8	3.2	0.7
Mean Nov. flow	cfs	1.0	13.9	2.7	4.8	18.0	6.5	2.8	5.1	3.6	1.6
Mean Dec. flow	cfs	0.8	2.1	1.5	2.0	5.9	2.5	4.2	1.8	1.3	0.9
Mean Jan. flow	cfs	0.8	1.3	1.2	1.3	1.8	1.4	1.5	1.2	1.1	0.9
Mean Feb. flow	cfs	0.8	1.1	12.6	1.1	1.3	1.7	1.1	1.1	1.1	0.9
Mean Mar. flow	cfs	9.1	3.8	14.5	4.2	6.1	3.7	20.1	12.0	30.2	8.3
Mean Apr. flow	cfs	48.5	14.0	39.4	31.6	27.3	53.5	27.1	44.4	4.9	16.7
Mean May flow	cfs	10.1	0.9	7.4	21.2	3.5	7.9	12.6	17.3	31.4	3.6
Mean Jun. flow	cfs	3.0	1.0	5.6	1.5	0.5	14.8	27.1	1.4	5.3	0.7
Mean Jul. flow	cfs	0.9	0.7	3.9	13.4	1.5	0.8	6.1	1.1	12.3	0.6
Mean Aug. flow	cfs	0.8	1.6	0.6	4.1	4.9	0.8	3.2	1.0	1.3	28.9
Mean Sep. flow	cfs	1.3	6.8	1.1	11.5	5.9	0.7	20.6	3.7	2.7	8.3
Max. 1-day flow	cfs	173.6	80.4	95.3	129.5	142.1	151.4	111.2	152.9	133.6	102.0
Max. 3-day flow	cfs	156.9	71.6	83.4	118.9	130.4	133.7	95.6	134.2	116.9	89.8
Max. 7-day flow	cfs	146.2	47.8	71.6	88.2	114.2	108.7	65.0	100.8	101.2	62.5
Max. 30-day flow	cfs	55.7	17.2	39.5	42.8	49.2	55.0	28.9	44.4	36.7	33.5
Max. 90-day flow	cfs	22.9	7.1	24.1	24.0	27.0	26.0	23.6	25.0	23.3	12.8
Min. 1-day flow	cfs	0.33	0.27	0.35	0.39	0.30	0.31	0.36	0.28	0.34	0.28
Min. 3-day flow	cfs	0.34	0.28	0.37	0.43	0.31	0.31	0.37	0.28	0.36	0.29
Min. 7-day flow	cfs	0.35	0.30	0.38	0.52	0.31	0.32	0.37	0.34	0.47	0.31
Min. 30-day flow	cfs	0.62	0.60	0.52	1.09	0.52	0.36	0.77	0.89	1.00	0.52
Min. 90-day flow	cfs	0.78	0.77	1.42	1.27	1.06	0.61	1.45	1.14	1.18	0.93
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.053	0.069	0.047	0.046	0.030	0.032	0.034	0.037	0.056	0.051
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	5	8	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	15	7	3	6	6	3	11	8	10
Mean high pulse duration	days	12.8	10.4	20.4	15.4	16.6	20.0	11.3	31.0	13.8	12.5
Mean low pulse duration	days	14.8	6.2	9.4	7.7	11.5	11.7	9.0	4.2	6.8	17.9
Mean rate of flow increase[3]	cfs/day	4.1	2.8	3.7	7.5	5.1	4.8	7.7	4.2	5.5	3.2
Mean rate of flow decrease[4]	cfs/day	1.2	0.7	1.2	1.8	1.5	1.4	2.1	1.5	1.5	1.1
Number of flow reversals	#/year	61	64	75	56	58	51	55	73	62	61

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 3d: Summary of modeled hydrologic parameters for **Mine Year 10** at surface water monitoring location **SW-003**

Statistic	Units	Water Year									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.7	4.5	7.8	39.2	47.5	25.2	2.4	18.7	3.2	0.7
Mean Nov. flow	cfs	0.9	13.8	2.7	4.7	17.8	6.4	2.8	5.1	3.5	1.6
Mean Dec. flow	cfs	0.8	2.1	1.4	1.9	5.8	2.5	4.1	1.7	1.3	0.9
Mean Jan. flow	cfs	0.7	1.3	1.2	1.3	1.8	1.4	1.4	1.2	1.0	0.8
Mean Feb. flow	cfs	0.7	1.0	12.4	1.1	1.2	1.6	1.1	1.0	1.1	0.9
Mean Mar. flow	cfs	9.0	3.7	14.4	4.2	6.0	3.6	19.9	11.8	30.0	8.2
Mean Apr. flow	cfs	48.1	13.8	39.1	31.3	27.0	53.0	26.9	44.1	4.8	16.6
Mean May flow	cfs	10.0	0.8	7.4	21.0	3.5	7.9	12.5	17.2	31.1	3.6
Mean Jun. flow	cfs	2.9	0.9	5.5	1.4	0.5	14.6	26.8	1.3	5.3	0.7
Mean Jul. flow	cfs	0.8	0.7	3.8	13.2	1.4	0.7	6.0	1.0	12.1	0.6
Mean Aug. flow	cfs	0.7	1.6	0.6	4.0	4.9	0.8	3.1	0.9	1.3	28.7
Mean Sep. flow	cfs	1.2	6.7	1.1	11.4	5.8	0.6	20.4	3.6	2.5	8.3
Max. 1-day flow	cfs	171.6	79.6	94.4	128.3	140.5	150.1	110.1	151.3	132.5	101.4
Max. 3-day flow	cfs	155.2	70.7	82.6	117.7	129.3	132.6	94.5	132.9	115.8	89.0
Max. 7-day flow	cfs	144.7	47.3	70.9	87.4	113.2	107.7	64.4	99.9	100.3	62.0
Max. 30-day flow	cfs	55.2	17.1	39.2	42.4	48.9	54.5	28.6	44.1	36.4	33.2
Max. 90-day flow	cfs	22.7	7.0	23.9	23.8	26.8	25.7	23.4	24.8	23.1	12.7
Min. 1-day flow	cfs	0.32	0.26	0.34	0.38	0.29	0.30	0.35	0.27	0.33	0.27
Min. 3-day flow	cfs	0.33	0.27	0.35	0.42	0.29	0.30	0.35	0.27	0.35	0.28
Min. 7-day flow	cfs	0.33	0.29	0.36	0.50	0.30	0.31	0.35	0.33	0.45	0.30
Min. 30-day flow	cfs	0.60	0.57	0.50	1.05	0.50	0.35	0.74	0.86	0.96	0.50
Min. 90-day flow	cfs	0.75	0.74	1.37	1.22	1.02	0.59	1.40	1.10	1.14	0.90
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.051	0.068	0.045	0.045	0.029	0.031	0.033	0.036	0.055	0.050
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	5	7	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	15	7	3	6	6	3	11	9	9
Mean high pulse duration	days	12.8	10.4	20.4	17.4	16.6	20.0	11.3	31.0	13.8	12.5
Mean low pulse duration	days	14.8	6.2	9.4	7.7	11.5	11.7	9.0	4.2	5.9	20.0
Mean rate of flow increase[3]	cfs/day	3.9	2.7	3.6	7.4	5.1	4.8	7.6	4.2	5.6	3.1
Mean rate of flow decrease[4]	cfs/day	1.2	0.7	1.2	1.8	1.5	1.3	2.1	1.5	1.5	1.1
Number of flow reversals	#/year	61	64	75	55	58	51	55	73	62	60

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 3e: Summary of modeled hydrologic parameters for **Mine Year 15** at surface water monitoring location **SW-003**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.7	4.6	7.8	39.2	47.5	25.2	2.4	18.7	3.2	0.7
Mean Nov. flow	cfs	0.9	13.8	2.7	4.7	17.8	6.5	2.8	5.1	3.5	1.6
Mean Dec. flow	cfs	0.8	2.1	1.4	1.9	5.8	2.5	4.1	1.7	1.3	0.9
Mean Jan. flow	cfs	0.7	1.3	1.2	1.3	1.8	1.4	1.4	1.2	1.1	0.9
Mean Feb. flow	cfs	0.8	1.0	12.4	1.1	1.2	1.6	1.1	1.0	1.1	0.9
Mean Mar. flow	cfs	9.0	3.7	14.4	4.2	6.0	3.6	19.9	11.8	30.0	8.2
Mean Apr. flow	cfs	48.1	13.9	39.1	31.3	27.1	53.1	26.9	44.1	4.8	16.6
Mean May flow	cfs	10.1	0.8	7.4	21.0	3.5	7.9	12.5	17.2	31.2	3.6
Mean Jun. flow	cfs	2.9	1.0	5.6	1.4	0.5	14.6	26.8	1.3	5.3	0.7
Mean Jul. flow	cfs	0.8	0.7	3.9	13.2	1.4	0.8	6.0	1.0	12.1	0.6
Mean Aug. flow	cfs	0.7	1.6	0.6	4.2	4.9	0.8	3.1	0.9	1.3	28.7
Mean Sep. flow	cfs	1.2	6.7	1.1	11.4	5.9	0.6	20.4	3.6	2.5	8.3
Max. 1-day flow	cfs	171.7	79.6	94.5	128.4	140.6	150.2	110.1	151.3	132.5	101.4
Max. 3-day flow	cfs	155.3	70.8	82.6	117.8	129.3	132.7	94.5	132.9	115.8	89.0
Max. 7-day flow	cfs	144.7	47.3	71.0	87.4	113.2	107.8	64.5	99.9	100.3	62.0
Max. 30-day flow	cfs	55.2	17.1	39.2	42.4	48.9	54.6	28.6	44.1	36.4	33.2
Max. 90-day flow	cfs	22.7	7.0	23.9	23.8	26.8	25.7	23.4	24.8	23.1	12.7
Min. 1-day flow	cfs	0.32	0.26	0.34	0.38	0.29	0.30	0.35	0.27	0.33	0.28
Min. 3-day flow	cfs	0.33	0.27	0.36	0.42	0.30	0.30	0.35	0.27	0.35	0.28
Min. 7-day flow	cfs	0.34	0.29	0.37	0.51	0.30	0.31	0.36	0.33	0.45	0.30
Min. 30-day flow	cfs	0.60	0.58	0.51	1.05	0.50	0.35	0.74	0.87	0.97	0.50
Min. 90-day flow	cfs	0.76	0.74	1.37	1.23	1.03	0.59	1.40	1.10	1.14	0.90
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.052	0.068	0.045	0.045	0.029	0.031	0.034	0.037	0.055	0.050
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	5	8	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	15	7	3	6	6	3	11	8	9
Mean high pulse duration	days	12.8	10.4	20.4	15.5	16.6	20.0	11.3	31.0	13.8	12.3
Mean low pulse duration	days	14.8	6.2	9.4	7.7	11.5	11.7	9.0	4.2	6.6	20.0
Mean rate of flow increase[3]	cfs/day	4.1	2.8	3.6	7.4	5.0	4.8	7.6	4.2	5.5	3.1
Mean rate of flow decrease[4]	cfs/day	1.2	0.7	1.2	1.8	1.5	1.3	2.1	1.5	1.5	1.1
Number of flow reversals	#/year	61	64	75	56	58	51	55	73	62	60

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 3f: Summary of modeled hydrologic parameters for **Mine Year 20** at surface water monitoring location **SW-003**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.6	4.5	7.8	39.2	47.5	25.1	2.4	18.6	3.1	0.6
Mean Nov. flow	cfs	0.9	13.8	2.6	4.7	17.8	6.4	2.7	5.0	3.5	1.5
Mean Dec. flow	cfs	0.7	2.0	1.4	1.9	5.8	2.4	4.0	1.7	1.2	0.8
Mean Jan. flow	cfs	0.7	1.2	1.1	1.2	1.7	1.3	1.4	1.1	1.0	0.8
Mean Feb. flow	cfs	0.7	1.0	12.4	1.0	1.2	1.6	1.0	1.0	1.0	0.8
Mean Mar. flow	cfs	8.9	3.7	14.4	4.1	6.0	3.5	19.8	11.8	29.9	8.2
Mean Apr. flow	cfs	48.0	13.8	39.0	31.3	27.0	53.0	26.9	44.0	4.8	16.5
Mean May flow	cfs	10.0	0.8	7.3	21.0	3.4	7.8	12.4	17.1	31.1	3.5
Mean Jun. flow	cfs	2.9	0.9	5.5	1.4	0.5	14.6	26.8	1.3	5.2	0.6
Mean Jul. flow	cfs	0.8	0.6	3.8	13.2	1.4	0.7	6.0	1.0	12.1	0.6
Mean Aug. flow	cfs	0.7	1.5	0.6	4.0	4.8	0.7	3.0	0.9	1.2	28.7
Mean Sep. flow	cfs	1.2	6.7	1.0	11.4	5.8	0.6	20.3	3.5	2.5	8.2
Max. 1-day flow	cfs	171.6	79.5	94.4	128.3	140.6	150.1	110.0	151.3	132.3	101.2
Max. 3-day flow	cfs	155.3	70.7	82.6	117.7	129.3	132.6	94.5	132.9	115.8	89.0
Max. 7-day flow	cfs	144.7	47.3	70.9	87.4	113.2	107.7	64.4	99.9	100.3	62.0
Max. 30-day flow	cfs	55.2	17.0	39.2	42.4	48.8	54.5	28.5	44.0	36.4	33.2
Max. 90-day flow	cfs	22.6	7.0	23.9	23.8	26.7	25.7	23.3	24.7	23.0	12.7
Min. 1-day flow	cfs	0.30	0.25	0.31	0.35	0.27	0.28	0.33	0.25	0.31	0.25
Min. 3-day flow	cfs	0.31	0.25	0.33	0.39	0.27	0.28	0.33	0.25	0.32	0.26
Min. 7-day flow	cfs	0.31	0.27	0.34	0.47	0.28	0.29	0.33	0.31	0.42	0.28
Min. 30-day flow	cfs	0.55	0.53	0.47	0.98	0.46	0.32	0.69	0.80	0.89	0.46
Min. 90-day flow	cfs	0.70	0.69	1.27	1.14	0.95	0.55	1.30	1.02	1.06	0.83
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.048	0.064	0.042	0.042	0.027	0.029	0.031	0.034	0.051	0.047
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	5	8	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	15	7	4	6	6	3	11	8	9
Mean high pulse duration	days	12.8	10.6	20.4	15.4	16.6	20.0	11.3	31.0	13.8	12.3
Mean low pulse duration	days	14.8	6.2	9.6	6.0	11.2	11.7	9.0	4.2	6.6	20.0
Mean rate of flow increase[3]	cfs/day	4.0	2.8	3.7	7.5	5.0	4.8	7.6	4.2	5.5	3.1
Mean rate of flow decrease[4]	cfs/day	1.2	0.7	1.2	1.8	1.5	1.3	2.1	1.5	1.5	1.1
Number of flow reversals	#/year	61	64	73	56	58	51	55	73	62	61

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 3g: Summary of modeled hydrologic parameters for **Mine Facilities off** at surface water monitoring location **SW-003**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	2.5	4.4	7.7	39.1	47.4	25.0	2.3	18.6	3.1	0.6
Mean Nov. flow	cfs	0.8	13.7	2.6	4.6	17.7	6.3	2.6	5.0	3.4	1.5
Mean Dec. flow	cfs	0.7	2.0	1.3	1.8	5.7	2.4	4.0	1.6	1.2	0.8
Mean Jan. flow	cfs	0.6	1.1	1.1	1.1	1.7	1.3	1.3	1.1	0.9	0.7
Mean Feb. flow	cfs	0.6	0.9	12.3	1.0	1.1	1.5	1.0	0.9	0.9	0.8
Mean Mar. flow	cfs	8.9	3.6	14.3	4.0	5.9	3.5	19.7	11.7	29.9	8.1
Mean Apr. flow	cfs	48.0	13.7	38.9	31.2	26.9	52.9	26.8	43.9	4.7	16.5
Mean May flow	cfs	9.9	0.7	7.3	20.9	3.4	7.8	12.4	17.1	31.0	3.5
Mean Jun. flow	cfs	2.8	0.8	5.4	1.3	0.4	14.5	26.7	1.2	5.2	0.6
Mean Jul. flow	cfs	0.7	0.6	3.7	13.1	1.3	0.6	5.9	0.9	12.0	0.5
Mean Aug. flow	cfs	0.6	1.4	0.5	3.9	4.8	0.7	3.0	0.8	1.2	28.6
Mean Sep. flow	cfs	1.1	6.6	1.0	11.3	5.7	0.5	20.3	3.5	2.4	8.1
Max. 1-day flow	cfs	171.4	79.4	94.3	128.2	140.4	150.0	109.9	151.1	132.3	101.2
Max. 3-day flow	cfs	155.1	70.6	82.4	117.6	129.1	132.5	94.3	132.8	115.6	88.9
Max. 7-day flow	cfs	144.6	47.2	70.8	87.2	113.0	107.6	64.3	99.8	100.2	61.9
Max. 30-day flow	cfs	55.1	16.9	39.1	42.3	48.8	54.4	28.5	43.9	36.3	33.1
Max. 90-day flow	cfs	22.5	6.9	23.8	23.7	26.6	25.6	23.3	24.7	23.0	12.6
Min. 1-day flow	cfs	0.27	0.23	0.29	0.33	0.25	0.26	0.30	0.23	0.28	0.24
Min. 3-day flow	cfs	0.29	0.23	0.31	0.36	0.25	0.26	0.30	0.23	0.30	0.24
Min. 7-day flow	cfs	0.29	0.25	0.31	0.43	0.26	0.26	0.30	0.28	0.39	0.26
Min. 30-day flow	cfs	0.51	0.49	0.43	0.90	0.43	0.30	0.63	0.74	0.83	0.43
Min. 90-day flow	cfs	0.65	0.64	1.18	1.05	0.88	0.51	1.20	0.94	0.98	0.77
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.045	0.060	0.040	0.039	0.025	0.027	0.029	0.032	0.048	0.044
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	30-Jul	2-Aug	19-Jun	3-Jul	30-Jun	2-Sep	15-Oct	31-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	5	7	8	5	12	3	6	6
Number of low pulses[2]	#/year	14	15	7	3	6	6	3	11	9	9
Mean high pulse duration	days	12.8	10.4	20.4	17.4	16.6	20.0	11.3	31.0	13.8	12.5
Mean low pulse duration	days	14.8	6.2	9.4	7.7	11.5	11.7	9.0	4.2	5.9	20.0
Mean rate of flow increase[3]	cfs/day	3.9	2.8	3.6	7.4	5.1	4.8	7.6	4.2	5.6	3.1
Mean rate of flow decrease[4]	cfs/day	1.2	0.7	1.2	1.8	1.5	1.3	2.1	1.5	1.5	1.1
Number of flow reversals	#/year	61	64	75	56	58	51	55	73	61	61

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 4a: Summary of modeled hydrologic parameters for **Existing Conditions** at surface water monitoring location **SW-004**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	4.9	6.1	13.0	65.9	76.2	41.5	3.5	29.2	6.4	1.3
Mean Nov. flow	cfs	1.7	26.1	3.7	5.9	28.0	9.6	4.1	7.4	6.0	3.5
Mean Dec. flow	cfs	1.4	3.0	2.1	2.7	8.2	3.5	7.6	2.5	2.0	1.7
Mean Jan. flow	cfs	1.3	1.9	1.9	1.9	2.5	2.1	2.2	1.9	1.7	1.5
Mean Feb. flow	cfs	1.3	1.7	21.4	1.7	1.9	2.4	1.7	1.7	1.7	1.5
Mean Mar. flow	cfs	16.9	5.3	21.9	6.4	9.7	5.8	33.6	17.2	49.8	14.0
Mean Apr. flow	cfs	84.6	23.3	63.8	52.1	44.6	85.2	43.1	70.4	6.2	26.4
Mean May flow	cfs	13.7	1.2	10.7	32.7	4.6	12.1	16.3	29.1	52.3	5.8
Mean Jun. flow	cfs	4.9	1.4	7.9	2.0	0.8	24.6	46.8	1.9	6.9	0.9
Mean Jul. flow	cfs	1.3	1.1	8.0	23.6	2.2	1.2	9.2	1.6	21.0	0.8
Mean Aug. flow	cfs	1.3	2.1	1.0	7.2	8.1	1.2	5.2	1.5	1.9	48.0
Mean Sep. flow	cfs	2.2	12.1	1.4	19.3	10.2	1.0	34.8	6.1	4.4	11.6
Max. 1-day flow	cfs	283.4	140.4	142.4	217.9	232.5	242.7	173.8	240.9	225.0	165.4
Max. 3-day flow	cfs	281.3	123.2	136.2	197.3	216.5	221.5	158.9	224.7	194.5	145.5
Max. 7-day flow	cfs	260.1	87.7	114.9	150.3	191.1	180.7	109.2	172.8	174.5	103.3
Max. 30-day flow	cfs	94.4	29.7	63.8	71.2	78.5	88.8	47.8	70.4	59.4	54.3
Max. 90-day flow	cfs	38.8	11.9	38.5	38.6	42.8	41.6	38.1	39.5	37.9	20.5
Min. 1-day flow	cfs	0.53	0.41	0.58	0.67	0.43	0.43	0.75	0.54	0.49	0.39
Min. 3-day flow	cfs	0.54	0.41	0.58	0.70	0.44	0.44	0.76	0.56	0.54	0.42
Min. 7-day flow	cfs	0.56	0.45	0.59	0.76	0.46	0.45	0.77	0.61	0.69	0.47
Min. 30-day flow	cfs	1.14	0.87	0.85	1.68	0.80	0.63	1.17	1.41	1.42	0.76
Min. 90-day flow	cfs	1.35	1.12	2.09	1.89	1.55	0.97	2.16	1.67	1.81	1.53
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.049	0.064	0.046	0.041	0.028	0.028	0.044	0.043	0.051	0.048
Julian date of max. flow	day	23-Apr	2-Nov	25-Apr	18-Apr	9-Oct	15-Apr	27-Jun	3-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	5-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	4	7	8	4	13	4	6	6
Number of low pulses[2]	#/year	6	8	5	3	4	3	3	8	6	6
Mean high pulse duration	days	13.0	10.6	24.8	17.6	16.8	23.8	10.4	23.0	14.3	12.2
Mean low pulse duration	days	31.5	14.3	13.0	7.0	18.3	24.7	9.0	5.6	8.7	30.5
Mean rate of flow increase[3]	cfs/day	4.6	3.6	4.8	10.1	7.0	6.6	10.4	5.5	8.9	4.3
Mean rate of flow decrease[4]	cfs/day	1.9	1.1	1.8	3.0	2.4	2.2	3.5	2.3	2.3	1.8
Number of flow reversals	#/year	46	53	59	42	46	39	49	55	44	50

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 4b: Summary of modeled hydrologic parameters for **Mine Year 1** at surface water monitoring location **SW-004**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	4.6	5.6	12.4	62.5	72.6	39.4	3.4	28.0	6.0	1.3
Mean Nov. flow	cfs	1.6	24.8	3.6	5.8	26.7	9.2	4.0	7.2	5.7	3.3
Mean Dec. flow	cfs	1.4	2.9	2.1	2.6	7.9	3.4	7.2	2.4	1.9	1.6
Mean Jan. flow	cfs	1.3	1.9	1.8	1.8	2.4	2.0	2.2	1.8	1.6	1.4
Mean Feb. flow	cfs	1.3	1.6	20.2	1.6	1.8	2.3	1.6	1.6	1.6	1.4
Mean Mar. flow	cfs	15.9	5.0	20.9	6.1	9.2	5.5	31.8	16.1	47.2	13.2
Mean Apr. flow	cfs	79.9	22.2	60.7	49.5	42.4	81.0	41.2	66.9	6.1	25.2
Mean May flow	cfs	13.5	1.2	10.5	31.3	4.5	11.8	15.7	28.2	49.5	5.5
Mean Jun. flow	cfs	4.7	1.4	7.4	1.9	0.8	23.3	44.3	1.8	6.8	0.9
Mean Jul. flow	cfs	1.3	1.1	7.7	22.1	2.1	1.2	9.0	1.5	19.8	0.8
Mean Aug. flow	cfs	1.3	1.9	1.0	6.6	7.6	1.2	5.0	1.4	1.9	45.9
Mean Sep. flow	cfs	2.1	11.5	1.4	18.3	9.8	1.0	32.9	5.4	4.2	11.4
Max. 1-day flow	cfs	267.4	129.3	135.0	202.5	218.8	227.5	161.1	226.7	210.4	153.9
Max. 3-day flow	cfs	263.2	115.2	128.1	184.3	203.8	208.5	148.9	211.3	181.2	136.2
Max. 7-day flow	cfs	244.8	82.1	108.5	141.3	181.0	170.8	103.1	162.8	163.7	98.0
Max. 30-day flow	cfs	89.5	28.1	60.7	67.5	74.8	84.5	45.0	66.9	56.4	52.0
Max. 90-day flow	cfs	36.8	11.3	36.7	36.7	40.8	39.6	36.3	37.6	36.0	19.7
Min. 1-day flow	cfs	0.53	0.41	0.57	0.66	0.43	0.43	0.75	0.53	0.49	0.39
Min. 3-day flow	cfs	0.53	0.41	0.58	0.70	0.43	0.43	0.75	0.56	0.54	0.42
Min. 7-day flow	cfs	0.55	0.44	0.59	0.75	0.45	0.44	0.76	0.60	0.68	0.46
Min. 30-day flow	cfs	1.12	0.86	0.84	1.67	0.79	0.63	1.16	1.40	1.40	0.76
Min. 90-day flow	cfs	1.33	1.11	2.07	1.87	1.54	0.96	2.14	1.65	1.80	1.52
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.052	0.066	0.048	0.043	0.029	0.029	0.046	0.044	0.053	0.050
Julian date of max. flow	day	22-Apr	2-Nov	25-Apr	18-Apr	9-Oct	15-Apr	28-Jun	3-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	5-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	4	7	8	4	13	4	6	6
Number of low pulses[2]	#/year	8	9	5	3	4	3	3	8	6	6
Mean high pulse duration	days	13.0	10.6	24.8	17.7	16.6	24.0	10.4	23.0	14.2	12.2
Mean low pulse duration	days	23.9	12.4	13.0	7.3	18.3	24.3	9.0	5.5	8.8	30.5
Mean rate of flow increase[3]	cfs/day	4.4	3.4	4.5	9.1	6.5	6.2	9.6	5.0	8.1	3.9
Mean rate of flow decrease[4]	cfs/day	1.8	1.0	1.7	2.8	2.2	2.0	3.2	2.2	2.1	1.6
Number of flow reversals	#/year	46	51	57	44	46	37	47	59	44	47

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 4c: Summary of modeled hydrologic parameters for **Mine Year 5** at surface water monitoring location **SW-004**

Statistic	Units	Water Year									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	4.6	5.4	12.3	61.9	71.9	39.0	3.4	27.7	5.9	1.3
Mean Nov. flow	cfs	1.6	24.5	3.6	5.8	26.4	9.1	3.8	7.1	5.6	3.2
Mean Dec. flow	cfs	1.3	2.8	2.0	2.5	7.9	3.3	7.1	2.4	1.9	1.5
Mean Jan. flow	cfs	1.3	1.8	1.7	1.8	2.4	2.0	2.1	1.7	1.6	1.4
Mean Feb. flow	cfs	1.3	1.5	19.9	1.6	1.8	2.3	1.6	1.5	1.6	1.4
Mean Mar. flow	cfs	15.7	4.9	20.7	6.0	9.1	5.4	31.4	15.8	46.7	13.0
Mean Apr. flow	cfs	78.9	22.0	60.0	49.0	41.9	80.2	40.7	66.2	6.1	24.9
Mean May flow	cfs	13.3	1.2	10.4	30.9	4.4	11.7	15.5	28.0	48.9	5.4
Mean Jun. flow	cfs	4.6	1.3	7.3	1.9	0.8	23.0	43.8	1.8	6.8	0.8
Mean Jul. flow	cfs	1.3	1.0	7.6	21.8	2.0	1.1	8.9	1.5	19.5	0.8
Mean Aug. flow	cfs	1.3	1.9	1.0	6.5	7.5	1.1	4.9	1.4	1.8	45.1
Mean Sep. flow	cfs	2.0	11.1	1.4	18.2	9.6	1.0	32.5	5.4	4.0	11.1
Max. 1-day flow	cfs	263.6	127.2	133.8	199.5	215.8	224.4	159.3	223.9	206.1	151.6
Max. 3-day flow	cfs	259.7	113.8	126.4	181.7	200.9	206.0	147.0	208.7	178.9	134.5
Max. 7-day flow	cfs	241.7	81.0	107.2	139.6	178.9	168.8	102.0	160.9	161.6	96.9
Max. 30-day flow	cfs	88.5	27.7	60.0	66.8	74.0	83.6	44.5	66.2	55.8	51.2
Max. 90-day flow	cfs	36.4	11.1	36.3	36.4	40.4	39.2	35.9	37.2	35.6	19.3
Min. 1-day flow	cfs	0.51	0.40	0.56	0.64	0.42	0.42	0.73	0.52	0.48	0.38
Min. 3-day flow	cfs	0.52	0.40	0.56	0.68	0.42	0.42	0.73	0.54	0.52	0.41
Min. 7-day flow	cfs	0.54	0.43	0.57	0.73	0.44	0.43	0.74	0.59	0.66	0.45
Min. 30-day flow	cfs	1.09	0.84	0.82	1.62	0.77	0.61	1.13	1.36	1.36	0.74
Min. 90-day flow	cfs	1.30	1.08	2.02	1.82	1.50	0.94	2.08	1.61	1.75	1.48
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.051	0.066	0.047	0.042	0.028	0.029	0.046	0.044	0.052	0.049
Julian date of max. flow	day	22-Apr	2-Nov	25-Apr	18-Apr	9-Oct	15-Apr	28-Jun	3-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	5-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	4	7	8	4	13	4	6	6
Number of low pulses[2]	#/year	7	9	5	3	4	3	3	9	6	6
Mean high pulse duration	days	13.0	10.4	24.8	17.7	16.8	24.0	10.3	22.8	14.2	12.5
Mean low pulse duration	days	27.1	12.6	13.0	7.3	18.3	24.3	9.0	4.8	8.8	30.7
Mean rate of flow increase[3]	cfs/day	4.3	3.3	4.4	8.9	6.4	6.0	9.5	5.1	7.9	3.8
Mean rate of flow decrease[4]	cfs/day	1.8	1.0	1.6	2.7	2.2	2.0	3.2	2.1	2.1	1.6
Number of flow reversals	#/year	46	51	57	44	46	37	47	55	44	48

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 4d: Summary of modeled hydrologic parameters for **Mine Year 10** at surface water monitoring location **SW-004**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	4.5	5.4	12.2	61.3	71.3	38.6	3.3	27.4	5.8	1.2
Mean Nov. flow	cfs	1.5	24.3	3.5	5.7	26.2	9.0	3.8	7.0	5.6	3.2
Mean Dec. flow	cfs	1.3	2.8	2.0	2.5	7.8	3.3	7.0	2.3	1.8	1.5
Mean Jan. flow	cfs	1.3	1.8	1.7	1.8	2.3	1.9	2.1	1.7	1.5	1.3
Mean Feb. flow	cfs	1.3	1.5	19.7	1.5	1.7	2.2	1.5	1.5	1.5	1.3
Mean Mar. flow	cfs	15.5	4.8	20.4	5.9	8.9	5.3	31.0	15.6	46.2	12.9
Mean Apr. flow	cfs	78.1	21.8	59.5	48.5	41.5	79.4	40.4	65.6	6.1	24.7
Mean May flow	cfs	13.3	1.2	10.4	30.6	4.4	11.7	15.3	27.8	48.4	5.4
Mean Jun. flow	cfs	4.5	1.3	7.1	1.8	0.8	22.7	43.3	1.7	6.7	0.8
Mean Jul. flow	cfs	1.2	1.0	7.5	21.5	2.0	1.1	8.9	1.5	19.3	0.8
Mean Aug. flow	cfs	1.2	1.9	1.0	6.4	7.4	1.1	4.8	1.3	1.8	44.6
Mean Sep. flow	cfs	2.0	11.0	1.3	18.0	9.5	0.9	32.2	5.3	3.8	11.0
Max. 1-day flow	cfs	260.8	124.8	132.2	196.3	212.9	221.7	157.1	221.2	202.4	149.3
Max. 3-day flow	cfs	256.9	112.2	124.8	179.2	198.6	203.7	144.8	206.4	176.7	132.7
Max. 7-day flow	cfs	239.1	80.0	106.1	138.0	177.1	167.1	100.9	159.1	159.6	95.9
Max. 30-day flow	cfs	87.6	27.4	59.5	66.1	73.4	82.9	44.0	65.6	55.3	50.7
Max. 90-day flow	cfs	36.0	11.0	35.9	36.0	40.0	38.8	35.5	36.9	35.2	19.1
Min. 1-day flow	cfs	0.50	0.39	0.54	0.63	0.41	0.41	0.71	0.50	0.46	0.37
Min. 3-day flow	cfs	0.50	0.39	0.55	0.66	0.41	0.41	0.71	0.53	0.51	0.39
Min. 7-day flow	cfs	0.52	0.42	0.56	0.71	0.43	0.42	0.72	0.57	0.64	0.44
Min. 30-day flow	cfs	1.06	0.82	0.80	1.58	0.75	0.59	1.10	1.32	1.33	0.72
Min. 90-day flow	cfs	1.26	1.05	1.96	1.77	1.45	0.91	2.02	1.56	1.70	1.44
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.050	0.065	0.046	0.041	0.028	0.028	0.045	0.043	0.051	0.048
Julian date of max. flow	day	23-Apr	2-Nov	25-Apr	18-Apr	9-Oct	15-Apr	28-Jun	3-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	5-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	4	7	8	4	13	4	6	6
Number of low pulses[2]	#/year	6	9	5	3	4	3	3	9	6	6
Mean high pulse duration	days	13.0	10.6	24.8	17.7	16.8	24.0	10.3	22.8	14.2	12.3
Mean low pulse duration	days	31.7	12.7	13.0	7.3	18.3	24.3	8.7	4.8	8.8	31.0
Mean rate of flow increase[3]	cfs/day	4.2	3.2	4.4	8.8	6.3	5.9	9.3	5.0	7.5	3.8
Mean rate of flow decrease[4]	cfs/day	1.7	1.0	1.6	2.7	2.2	1.9	3.1	2.1	2.0	1.6
Number of flow reversals	#/year	46	51	51	44	48	37	47	52	44	47

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 4e: Summary of modeled hydrologic parameters for **Mine Year 15** at surface water monitoring location **SW-004**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	4.5	5.3	12.1	61.0	71.0	38.4	3.3	27.3	5.7	1.2
Mean Nov. flow	cfs	1.5	24.2	3.6	5.7	26.1	9.0	3.8	7.0	5.5	3.2
Mean Dec. flow	cfs	1.3	2.8	2.0	2.5	7.8	3.3	7.0	2.3	1.8	1.5
Mean Jan. flow	cfs	1.3	1.8	1.7	1.8	2.4	1.9	2.1	1.7	1.5	1.3
Mean Feb. flow	cfs	1.3	1.5	19.6	1.5	1.7	2.2	1.6	1.5	1.5	1.3
Mean Mar. flow	cfs	15.4	4.8	20.3	5.8	8.9	5.3	30.9	15.5	46.0	12.8
Mean Apr. flow	cfs	77.7	21.7	59.2	48.3	41.3	79.0	40.2	65.3	6.1	24.6
Mean May flow	cfs	13.2	1.2	10.4	30.5	4.4	11.7	15.2	27.8	48.2	5.3
Mean Jun. flow	cfs	4.5	1.3	7.1	1.8	0.8	22.6	43.1	1.7	6.7	0.8
Mean Jul. flow	cfs	1.2	1.0	7.5	21.4	2.0	1.1	8.9	1.5	19.2	0.8
Mean Aug. flow	cfs	1.3	1.9	1.0	6.5	7.4	1.1	4.8	1.4	1.8	44.4
Mean Sep. flow	cfs	2.0	11.0	1.3	17.9	9.5	1.0	32.0	5.3	3.8	11.0
Max. 1-day flow	cfs	259.2	123.6	131.9	195.0	211.8	220.0	156.7	220.3	199.7	148.1
Max. 3-day flow	cfs	255.4	111.7	124.1	177.9	197.8	202.6	144.0	205.4	175.6	131.8
Max. 7-day flow	cfs	237.8	79.5	105.5	137.1	176.2	166.3	100.4	158.3	158.5	95.4
Max. 30-day flow	cfs	87.2	27.3	59.2	65.8	73.1	82.5	43.7	65.3	55.0	50.4
Max. 90-day flow	cfs	35.9	11.0	35.8	35.9	39.8	38.7	35.4	36.7	35.1	19.1
Min. 1-day flow	cfs	0.50	0.39	0.55	0.63	0.41	0.41	0.71	0.51	0.47	0.37
Min. 3-day flow	cfs	0.51	0.39	0.55	0.67	0.42	0.41	0.72	0.53	0.51	0.40
Min. 7-day flow	cfs	0.53	0.42	0.56	0.72	0.43	0.42	0.73	0.57	0.65	0.44
Min. 30-day flow	cfs	1.07	0.82	0.81	1.59	0.76	0.60	1.11	1.33	1.34	0.72
Min. 90-day flow	cfs	1.27	1.06	1.98	1.78	1.47	0.92	2.04	1.57	1.72	1.45
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.051	0.066	0.047	0.042	0.028	0.029	0.045	0.044	0.052	0.049
Julian date of max. flow	day	23-Apr	2-Nov	25-Apr	18-Apr	9-Oct	15-Apr	28-Jun	3-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	5-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	4	7	8	4	13	4	6	6
Number of low pulses[2]	#/year	7	9	5	3	4	3	3	9	6	6
Mean high pulse duration	days	13.0	10.6	24.8	17.7	16.6	24.0	10.3	22.8	14.2	12.5
Mean low pulse duration	days	27.3	12.8	12.8	7.3	18.3	24.3	8.7	4.8	8.8	30.8
Mean rate of flow increase[3]	cfs/day	4.1	3.2	4.4	8.7	6.2	5.9	9.3	4.9	7.4	3.7
Mean rate of flow decrease[4]	cfs/day	1.7	1.0	1.6	2.6	2.2	1.9	3.1	2.1	2.0	1.5
Number of flow reversals	#/year	46	53	51	44	48	37	47	53	44	48

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 4f: Summary of modeled hydrologic parameters for **Mine Year 20** at surface water monitoring location **SW-004**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	4.4	5.3	12.1	61.0	70.9	38.4	3.3	27.3	5.6	1.2
Mean Nov. flow	cfs	1.5	24.2	3.5	5.7	26.1	8.9	3.7	6.9	5.5	3.2
Mean Dec. flow	cfs	1.2	2.7	1.9	2.4	7.7	3.2	6.9	2.3	1.8	1.4
Mean Jan. flow	cfs	1.2	1.7	1.6	1.7	2.3	1.8	2.0	1.6	1.5	1.2
Mean Feb. flow	cfs	1.2	1.4	19.5	1.5	1.7	2.2	1.5	1.4	1.5	1.3
Mean Mar. flow	cfs	15.3	4.7	20.3	5.8	8.9	5.3	30.8	15.4	46.0	12.7
Mean Apr. flow	cfs	77.7	21.7	59.2	48.3	41.3	79.0	40.2	65.3	6.0	24.6
Mean May flow	cfs	13.2	1.1	10.3	30.5	4.4	11.6	15.2	27.7	48.2	5.3
Mean Jun. flow	cfs	4.4	1.3	6.8	1.8	0.7	22.6	43.1	1.7	6.6	0.8
Mean Jul. flow	cfs	1.2	1.0	7.5	21.4	2.0	1.1	8.8	1.4	19.2	0.7
Mean Aug. flow	cfs	1.2	1.9	0.9	6.3	7.3	1.1	4.8	1.3	1.7	44.5
Mean Sep. flow	cfs	1.9	10.9	1.2	17.8	9.4	0.9	32.0	5.3	3.8	11.0
Max. 1-day flow	cfs	259.2	123.7	131.9	195.0	211.8	220.1	156.6	220.3	199.9	148.2
Max. 3-day flow	cfs	255.5	111.7	124.1	178.0	197.8	202.6	144.0	205.4	175.7	131.9
Max. 7-day flow	cfs	237.8	79.5	105.5	137.2	176.2	166.3	100.4	158.4	158.6	95.4
Max. 30-day flow	cfs	87.2	27.2	59.2	65.8	73.1	82.4	43.7	65.3	55.0	50.5
Max. 90-day flow	cfs	35.8	10.9	35.7	35.8	39.8	38.6	35.3	36.7	35.0	19.1
Min. 1-day flow	cfs	0.48	0.37	0.52	0.60	0.39	0.39	0.67	0.48	0.44	0.35
Min. 3-day flow	cfs	0.48	0.37	0.52	0.63	0.39	0.39	0.68	0.50	0.49	0.38
Min. 7-day flow	cfs	0.50	0.40	0.53	0.68	0.41	0.40	0.69	0.54	0.61	0.42
Min. 30-day flow	cfs	1.02	0.78	0.76	1.51	0.72	0.57	1.05	1.26	1.27	0.68
Min. 90-day flow	cfs	1.21	1.01	1.87	1.69	1.39	0.87	1.93	1.49	1.63	1.37
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.048	0.063	0.045	0.040	0.027	0.027	0.043	0.041	0.049	0.047
Julian date of max. flow	day	23-Apr	2-Nov	25-Apr	18-Apr	9-Oct	15-Apr	28-Jun	3-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	5-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	4	7	8	4	13	4	6	6
Number of low pulses[2]	#/year	7	9	5	3	4	3	3	10	6	6
Mean high pulse duration	days	13.0	10.6	24.8	17.7	16.6	24.0	10.3	22.8	14.2	12.5
Mean low pulse duration	days	27.3	12.8	12.8	7.3	18.3	24.0	8.3	4.5	9.2	30.7
Mean rate of flow increase[3]	cfs/day	4.2	3.3	4.3	8.8	6.2	5.8	9.3	4.9	7.3	3.7
Mean rate of flow decrease[4]	cfs/day	1.7	1.0	1.6	2.6	2.2	1.9	3.1	2.1	2.0	1.5
Number of flow reversals	#/year	49	54	55	44	48	37	47	53	42	48

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 4g: Summary of modeled hydrologic parameters for **Mine Facilities off** at surface water monitoring location **SW-004**

Statistic	Units	Water Year									
		1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Mean Oct. flow	cfs	4.2	5.0	11.9	60.7	70.7	38.2	3.1	27.1	5.5	1.0
Mean Nov. flow	cfs	1.3	24.0	3.3	5.5	25.9	8.8	3.6	6.8	5.3	2.9
Mean Dec. flow	cfs	1.1	2.6	1.7	2.3	7.6	3.1	6.8	2.1	1.6	1.3
Mean Jan. flow	cfs	1.0	1.5	1.5	1.5	2.1	1.7	1.8	1.5	1.3	1.1
Mean Feb. flow	cfs	1.0	1.3	19.3	1.3	1.5	2.0	1.3	1.3	1.3	1.1
Mean Mar. flow	cfs	15.2	4.5	20.1	5.6	8.7	5.1	30.6	15.2	45.8	12.6
Mean Apr. flow	cfs	77.4	21.5	58.9	48.0	41.0	78.7	39.9	65.0	5.8	24.4
Mean May flow	cfs	13.0	1.0	10.2	30.2	4.2	11.4	15.0	27.5	47.9	5.1
Mean Jun. flow	cfs	4.3	1.1	6.8	1.6	0.6	22.3	42.8	1.5	6.5	0.7
Mean Jul. flow	cfs	1.0	0.8	7.3	21.2	1.8	0.9	8.6	1.2	18.9	0.6
Mean Aug. flow	cfs	1.0	1.6	0.8	6.1	7.1	0.9	4.6	1.1	1.6	44.1
Mean Sep. flow	cfs	1.7	10.7	1.1	17.6	9.2	0.8	31.7	5.0	3.6	10.8
Max. 1-day flow	cfs	258.6	123.2	131.6	194.5	211.3	219.5	156.3	219.9	199.1	147.7
Max. 3-day flow	cfs	254.9	111.4	123.7	177.4	197.3	202.1	143.6	204.9	175.2	131.4
Max. 7-day flow	cfs	237.3	79.2	105.1	136.8	175.7	165.9	100.1	157.9	158.1	95.0
Max. 30-day flow	cfs	86.9	27.1	58.9	65.5	72.8	82.2	43.4	65.0	54.7	50.2
Max. 90-day flow	cfs	35.6	10.7	35.5	35.6	39.6	38.4	35.1	36.5	34.8	18.8
Min. 1-day flow	cfs	0.42	0.32	0.45	0.52	0.34	0.34	0.59	0.42	0.39	0.31
Min. 3-day flow	cfs	0.42	0.32	0.46	0.55	0.34	0.34	0.59	0.44	0.43	0.33
Min. 7-day flow	cfs	0.43	0.35	0.46	0.59	0.36	0.35	0.60	0.48	0.54	0.37
Min. 30-day flow	cfs	0.89	0.68	0.67	1.32	0.63	0.49	0.92	1.10	1.11	0.60
Min. 90-day flow	cfs	1.05	0.88	1.64	1.47	1.21	0.76	1.69	1.30	1.42	1.20
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.043	0.056	0.039	0.035	0.024	0.024	0.038	0.037	0.044	0.042
Julian date of max. flow	day	23-Apr	2-Nov	25-Apr	18-Apr	9-Oct	15-Apr	28-Jun	3-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	5-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	4	5	4	7	8	4	13	3	6	6
Number of low pulses[2]	#/year	7	9	5	3	4	3	3	9	7	6
Mean high pulse duration	days	13.0	10.4	24.8	17.7	16.8	24.0	10.3	30.7	14.2	12.3
Mean low pulse duration	days	27.3	12.6	13.0	7.3	18.0	24.3	9.0	4.8	7.7	30.7
Mean rate of flow increase[3]	cfs/day	4.1	3.2	4.4	8.9	6.3	5.8	9.3	4.8	7.7	3.6
Mean rate of flow decrease[4]	cfs/day	1.7	1.0	1.6	2.6	2.2	1.9	3.1	2.1	2.0	1.6
Number of flow reversals	#/year	46	53	51	42	46	37	47	55	42	48

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 5a: Summary of modeled hydrologic parameters for **Existing Conditions** at surface water monitoring location **SW-004a**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	11.5	17.6	30.4	154.2	178.0	95.1	7.3	64.0	16.0	3.2
Mean Nov. flow	cfs	3.8	60.1	6.2	10.1	64.3	19.7	8.2	16.8	14.4	8.7
Mean Dec. flow	cfs	3.2	5.1	4.0	5.8	17.7	6.7	18.4	5.8	3.9	3.7
Mean Jan. flow	cfs	3.1	3.7	3.8	4.5	6.1	4.5	4.2	4.5	3.6	3.3
Mean Feb. flow	cfs	3.1	3.5	51.8	4.1	4.7	5.5	3.5	4.0	3.6	3.4
Mean Mar. flow	cfs	42.9	12.6	49.3	15.4	23.0	14.0	81.0	46.3	117.0	33.7
Mean Apr. flow	cfs	208.9	52.3	150.9	124.9	106.5	204.5	98.4	168.4	11.3	58.7
Mean May flow	cfs	24.8	2.5	20.3	73.7	8.5	23.6	37.0	59.5	127.8	13.3
Mean Jun. flow	cfs	11.6	3.7	20.9	4.1	2.0	59.0	111.0	4.2	12.0	2.3
Mean Jul. flow	cfs	3.5	2.7	17.7	59.0	5.1	2.5	16.9	4.1	52.3	2.1
Mean Aug. flow	cfs	3.1	5.4	2.6	17.8	20.0	3.0	12.2	3.8	3.6	113.9
Mean Sep. flow	cfs	4.9	29.1	3.8	45.7	22.8	2.5	84.5	15.0	10.5	21.1
Max. 1-day flow	cfs	856.7	396.5	388.6	598.2	624.7	674.4	525.2	679.4	658.2	441.9
Max. 3-day flow	cfs	772.1	329.1	338.2	531.3	567.3	586.1	411.2	596.8	535.1	365.2
Max. 7-day flow	cfs	678.8	222.7	284.9	378.8	472.6	459.2	269.5	433.8	447.0	246.4
Max. 30-day flow	cfs	226.2	71.0	150.9	172.1	183.5	211.7	116.5	168.4	140.9	124.7
Max. 90-day flow	cfs	93.1	27.9	89.1	90.2	99.5	98.5	88.6	92.7	89.1	46.7
Min. 1-day flow	cfs	1.35	1.01	1.49	1.53	1.10	1.12	1.84	1.32	1.24	1.01
Min. 3-day flow	cfs	1.37	1.01	1.51	1.59	1.11	1.13	1.85	1.43	1.34	1.09
Min. 7-day flow	cfs	1.40	1.09	1.53	1.77	1.16	1.15	1.87	1.49	1.65	1.18
Min. 30-day flow	cfs	2.73	2.24	2.17	3.83	1.98	1.61	2.95	3.46	2.95	1.98
Min. 90-day flow	cfs	3.14	2.83	3.99	4.41	3.59	2.42	4.49	4.03	3.70	3.38
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.052	0.066	0.051	0.041	0.030	0.031	0.047	0.045	0.052	0.053
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	4-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	7	4	9	8	4	13	6	6	7
Number of low pulses[2]	#/year	11	11	6	5	6	4	3	8	7	6
Mean high pulse duration	days	10.4	7.9	23.5	13.3	17.0	23.3	9.9	15.7	14.2	11.0
Mean low pulse duration	days	16.8	11.0	10.5	4.8	11.5	19.0	10.3	4.6	10.9	28.0
Mean rate of flow increase[3]	cfs/day	16.8	12.5	13.5	27.2	19.8	18.6	29.8	17.3	23.2	12.4
Mean rate of flow decrease[4]	cfs/day	4.9	2.9	4.8	7.9	6.2	5.8	9.5	6.1	7.0	4.5
Number of flow reversals	#/year	65	76	75	52	64	51	51	69	48	64

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 5b: Summary of modeled hydrologic parameters for **Mine Year 1** at surface water monitoring location **SW-004a**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	11.1	16.7	29.5	149.6	173.0	92.2	7.1	62.3	15.5	3.1
Mean Nov. flow	cfs	3.7	58.4	6.1	10.0	62.6	19.1	8.2	16.4	13.9	8.4
Mean Dec. flow	cfs	3.2	5.0	3.8	5.7	17.4	6.5	17.9	5.7	3.8	3.6
Mean Jan. flow	cfs	3.1	3.6	3.7	4.4	6.0	4.4	4.1	4.4	3.5	3.3
Mean Feb. flow	cfs	3.1	3.3	50.2	3.9	4.6	5.4	3.4	3.9	3.5	3.2
Mean Mar. flow	cfs	41.5	12.1	47.7	14.9	22.3	13.6	78.4	44.6	113.4	32.6
Mean Apr. flow	cfs	202.3	50.9	146.6	121.3	103.3	198.6	95.7	163.6	11.2	57.1
Mean May flow	cfs	24.4	2.5	19.9	71.7	8.4	23.2	35.8	58.4	123.8	12.9
Mean Jun. flow	cfs	11.2	3.5	20.0	3.9	2.0	57.1	107.6	4.0	11.8	2.3
Mean Jul. flow	cfs	3.4	2.7	17.3	56.9	4.9	2.4	16.6	4.0	50.6	2.1
Mean Aug. flow	cfs	3.0	5.2	2.6	17.0	19.2	3.0	11.8	3.6	3.4	110.7
Mean Sep. flow	cfs	4.8	28.2	3.6	44.2	22.2	2.5	81.8	14.2	10.2	20.8
Max. 1-day flow	cfs	824.1	381.3	372.8	577.9	600.5	652.9	505.2	653.8	636.7	426.4
Max. 3-day flow	cfs	746.4	316.3	327.3	513.0	549.1	568.5	395.4	576.7	515.6	352.7
Max. 7-day flow	cfs	655.9	215.2	275.9	366.3	457.6	445.5	261.0	420.2	431.8	238.9
Max. 30-day flow	cfs	219.4	68.7	146.6	166.8	178.3	205.7	112.6	163.6	136.7	121.2
Max. 90-day flow	cfs	90.2	27.0	86.5	87.5	96.7	95.6	85.9	90.0	86.4	45.5
Min. 1-day flow	cfs	1.34	1.00	1.48	1.52	1.09	1.11	1.83	1.31	1.23	1.01
Min. 3-day flow	cfs	1.36	1.00	1.49	1.58	1.10	1.12	1.84	1.41	1.33	1.09
Min. 7-day flow	cfs	1.39	1.08	1.52	1.75	1.15	1.14	1.85	1.48	1.64	1.17
Min. 30-day flow	cfs	2.71	2.22	2.15	3.80	1.96	1.60	2.93	3.43	2.93	1.97
Min. 90-day flow	cfs	3.12	2.80	3.96	4.38	3.56	2.40	4.45	4.00	3.67	3.35
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.053	0.068	0.052	0.041	0.031	0.032	0.048	0.046	0.053	0.054
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	4-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	7	4	9	8	4	13	6	6	6
Number of low pulses[2]	#/year	11	11	6	5	6	4	3	8	7	6
Mean high pulse duration	days	10.4	7.9	23.5	13.3	17.0	23.3	9.8	15.7	14.3	12.7
Mean low pulse duration	days	16.8	11.1	10.5	5.0	11.5	19.0	10.3	4.6	10.6	28.3
Mean rate of flow increase[3]	cfs/day	16.2	12.1	13.0	26.1	19.0	17.9	28.7	16.5	22.6	11.8
Mean rate of flow decrease[4]	cfs/day	4.8	2.8	4.6	7.6	6.0	5.6	9.1	5.9	6.7	4.4
Number of flow reversals	#/year	63	74	73	52	64	51	51	69	46	66

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 5c: Summary of modeled hydrologic parameters for **Mine Year 5** at surface water monitoring location **SW-004a**

Statistic	Units	Water Year									
		1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Mean Oct. flow	cfs	11.0	16.2	29.2	147.6	170.8	91.0	7.0	61.6	15.2	3.1
Mean Nov. flow	cfs	3.6	57.8	6.0	9.9	61.8	18.8	7.9	16.2	13.7	8.3
Mean Dec. flow	cfs	3.1	4.9	3.7	5.6	17.2	6.4	17.6	5.6	3.7	3.5
Mean Jan. flow	cfs	3.1	3.5	3.6	4.3	5.9	4.3	4.0	4.3	3.4	3.2
Mean Feb. flow	cfs	3.1	3.3	49.4	3.9	4.5	5.3	3.3	3.8	3.4	3.3
Mean Mar. flow	cfs	40.8	11.8	47.0	14.6	22.0	13.3	77.2	43.7	111.8	32.1
Mean Apr. flow	cfs	199.4	50.3	144.7	119.7	102.0	195.9	94.5	161.5	11.1	56.3
Mean May flow	cfs	24.2	2.4	19.8	70.7	8.3	23.1	35.1	58.0	122.1	12.7
Mean Jun. flow	cfs	11.0	3.4	19.6	3.8	2.0	56.3	106.3	3.9	11.7	2.2
Mean Jul. flow	cfs	3.3	2.7	17.2	56.1	4.8	2.4	16.5	3.9	49.8	2.1
Mean Aug. flow	cfs	3.0	5.1	2.5	16.8	18.9	2.9	11.6	3.5	3.4	108.9
Mean Sep. flow	cfs	4.7	27.4	3.5	43.7	21.9	2.4	80.6	14.0	9.9	20.3
Max. 1-day flow	cfs	809.3	375.2	365.7	569.9	588.1	643.6	495.7	642.3	626.5	420.5
Max. 3-day flow	cfs	735.1	310.5	323.0	505.3	541.0	561.3	387.9	568.0	506.5	347.7
Max. 7-day flow	cfs	646.0	212.3	272.1	361.0	451.1	439.7	257.3	414.7	425.2	235.7
Max. 30-day flow	cfs	216.4	67.8	144.7	164.6	176.1	203.1	111.0	161.5	134.9	119.3
Max. 90-day flow	cfs	89.0	26.6	85.4	86.4	95.5	94.4	84.8	88.9	85.3	44.7
Min. 1-day flow	cfs	1.32	0.99	1.46	1.50	1.07	1.09	1.80	1.29	1.21	0.99
Min. 3-day flow	cfs	1.34	0.99	1.47	1.55	1.09	1.10	1.81	1.39	1.31	1.07
Min. 7-day flow	cfs	1.37	1.06	1.49	1.73	1.13	1.12	1.83	1.46	1.61	1.15
Min. 30-day flow	cfs	2.67	2.19	2.12	3.74	1.93	1.57	2.89	3.38	2.88	1.94
Min. 90-day flow	cfs	3.07	2.76	3.90	4.31	3.51	2.36	4.39	3.94	3.62	3.30
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.053	0.068	0.052	0.041	0.031	0.032	0.048	0.046	0.053	0.054
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	4-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	7	4	9	8	4	13	6	6	7
Number of low pulses[2]	#/year	10	11	6	5	6	4	3	8	7	6
Mean high pulse duration	days	10.4	7.9	23.5	13.3	17.3	23.0	9.8	15.5	14.2	11.0
Mean low pulse duration	days	18.6	11.0	10.5	5.0	11.5	19.0	10.3	4.6	10.6	28.3
Mean rate of flow increase[3]	cfs/day	15.1	11.9	12.8	25.7	18.8	17.5	27.9	15.5	22.4	11.7
Mean rate of flow decrease[4]	cfs/day	4.7	2.7	4.5	7.5	5.8	5.5	9.0	5.9	6.5	4.3
Number of flow reversals	#/year	63	74	71	50	62	51	51	65	48	65

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 5d: Summary of modeled hydrologic parameters for **Mine Year 10** at surface water monitoring location **SW-004a**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	10.8	15.8	28.8	146.0	169.2	90.0	6.9	61.0	15.0	3.0
Mean Nov. flow	cfs	3.5	57.4	5.9	9.8	61.2	18.6	7.8	16.0	13.5	8.1
Mean Dec. flow	cfs	3.1	4.8	3.7	5.5	17.1	6.4	17.4	5.5	3.6	3.4
Mean Jan. flow	cfs	3.0	3.4	3.5	4.2	5.8	4.2	3.9	4.2	3.3	3.2
Mean Feb. flow	cfs	3.0	3.2	48.8	3.8	4.4	5.2	3.3	3.7	3.3	3.2
Mean Mar. flow	cfs	40.3	11.5	46.4	14.3	21.7	13.1	76.3	42.9	110.6	31.7
Mean Apr. flow	cfs	197.3	49.8	143.3	118.5	100.9	193.9	93.5	159.9	11.0	55.8
Mean May flow	cfs	24.0	2.4	19.7	70.0	8.2	23.0	34.5	57.7	120.7	12.5
Mean Jun. flow	cfs	10.8	3.4	19.2	3.8	2.0	55.6	105.2	3.8	11.6	2.2
Mean Jul. flow	cfs	3.2	2.6	17.0	55.4	4.7	2.4	16.4	3.8	49.2	2.0
Mean Aug. flow	cfs	2.9	5.0	2.5	16.5	18.7	2.9	11.4	3.4	3.3	107.6
Mean Sep. flow	cfs	4.6	27.2	3.4	43.2	21.6	2.4	79.6	13.8	9.6	20.1
Max. 1-day flow	cfs	798.4	370.7	361.9	563.9	580.1	636.9	489.1	633.8	620.1	416.6
Max. 3-day flow	cfs	726.8	306.2	320.4	499.4	535.4	556.1	382.7	561.4	500.2	343.8
Max. 7-day flow	cfs	638.7	210.1	269.3	356.9	446.3	435.3	254.6	410.6	420.4	233.1
Max. 30-day flow	cfs	214.1	67.0	143.3	163.0	174.4	201.1	109.7	159.9	133.4	118.0
Max. 90-day flow	cfs	88.0	26.3	84.5	85.6	94.6	93.4	83.9	88.0	84.4	44.2
Min. 1-day flow	cfs	1.30	0.97	1.44	1.47	1.05	1.08	1.77	1.27	1.19	0.97
Min. 3-day flow	cfs	1.31	0.97	1.45	1.53	1.07	1.08	1.78	1.37	1.29	1.05
Min. 7-day flow	cfs	1.35	1.05	1.47	1.70	1.11	1.10	1.80	1.44	1.59	1.13
Min. 30-day flow	cfs	2.62	2.15	2.08	3.68	1.90	1.55	2.84	3.32	2.84	1.90
Min. 90-day flow	cfs	3.02	2.72	3.84	4.24	3.45	2.32	4.31	3.87	3.56	3.25
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.053	0.068	0.052	0.041	0.031	0.032	0.047	0.046	0.053	0.054
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	4-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	7	4	9	8	4	13	6	6	7
Number of low pulses[2]	#/year	10	11	6	5	5	4	3	8	7	6
Mean high pulse duration	days	10.4	7.9	23.3	13.3	17.4	23.0	9.8	15.7	14.2	11.0
Mean low pulse duration	days	18.6	11.1	10.5	5.0	13.6	19.0	10.3	4.6	10.6	28.3
Mean rate of flow increase[3]	cfs/day	15.0	11.5	12.6	25.4	18.0	16.9	27.8	15.2	22.1	11.5
Mean rate of flow decrease[4]	cfs/day	4.7	2.7	4.5	7.4	5.8	5.4	8.8	5.9	6.4	4.2
Number of flow reversals	#/year	61	72	71	50	56	47	50	65	48	59

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 5e: Summary of modeled hydrologic parameters for **Mine Year 15** at surface water monitoring location **SW-004a**

Statistic	Units	Water Year									
		1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Mean Oct. flow	cfs	10.7	15.7	28.7	145.4	168.6	89.6	6.9	60.8	14.8	3.0
Mean Nov. flow	cfs	3.5	57.3	5.9	9.8	61.0	18.6	7.7	16.0	13.4	8.2
Mean Dec. flow	cfs	3.1	4.8	3.7	5.5	17.0	6.4	17.3	5.5	3.6	3.4
Mean Jan. flow	cfs	3.0	3.4	3.5	4.2	5.8	4.2	3.9	4.2	3.3	3.2
Mean Feb. flow	cfs	3.0	3.2	48.6	3.8	4.4	5.1	3.2	3.7	3.3	3.2
Mean Mar. flow	cfs	40.1	11.4	46.2	14.2	21.6	13.1	76.0	42.6	110.2	31.5
Mean Apr. flow	cfs	196.4	49.7	142.7	118.1	100.5	193.1	93.2	159.3	11.0	55.6
Mean May flow	cfs	23.9	2.4	19.7	69.7	8.2	23.0	34.3	57.6	120.2	12.5
Mean Jun. flow	cfs	10.8	3.4	19.0	3.8	2.0	55.4	104.8	3.8	11.5	2.2
Mean Jul. flow	cfs	3.2	2.6	17.0	55.1	4.7	2.4	16.4	3.8	49.0	2.0
Mean Aug. flow	cfs	2.9	5.0	2.5	16.6	18.6	2.9	11.3	3.4	3.3	107.2
Mean Sep. flow	cfs	4.6	27.1	3.4	43.0	21.5	2.4	79.2	13.8	9.6	20.1
Max. 1-day flow	cfs	793.8	368.8	359.5	561.6	575.9	634.0	486.0	630.1	616.5	414.8
Max. 3-day flow	cfs	723.4	304.3	319.3	497.1	533.1	554.1	381.4	558.8	497.5	342.3
Max. 7-day flow	cfs	635.8	209.3	268.2	355.3	444.3	433.6	253.5	409.1	418.5	232.0
Max. 30-day flow	cfs	213.2	66.7	142.7	162.3	173.7	200.4	109.2	159.3	132.9	117.5
Max. 90-day flow	cfs	87.7	26.2	84.2	85.2	94.2	93.0	83.5	87.7	84.0	44.0
Min. 1-day flow	cfs	1.30	0.97	1.44	1.48	1.05	1.08	1.77	1.27	1.19	0.98
Min. 3-day flow	cfs	1.31	0.97	1.45	1.53	1.07	1.08	1.78	1.37	1.29	1.05
Min. 7-day flow	cfs	1.35	1.05	1.47	1.70	1.12	1.10	1.80	1.44	1.59	1.13
Min. 30-day flow	cfs	2.63	2.15	2.09	3.69	1.90	1.55	2.84	3.32	2.84	1.91
Min. 90-day flow	cfs	3.02	2.72	3.84	4.24	3.45	2.32	4.32	3.88	3.56	3.25
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.053	0.068	0.052	0.041	0.031	0.032	0.048	0.046	0.053	0.054
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	4-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	7	4	9	8	4	13	6	6	7
Number of low pulses[2]	#/year	11	11	6	5	5	4	3	8	7	6
Mean high pulse duration	days	10.4	7.7	23.3	13.3	17.4	23.0	9.8	15.7	14.3	11.0
Mean low pulse duration	days	16.8	11.1	10.5	5.0	13.6	19.0	10.3	4.6	10.6	28.3
Mean rate of flow increase[3]	cfs/day	14.4	11.5	12.6	25.3	17.7	16.6	27.7	15.1	21.7	11.5
Mean rate of flow decrease[4]	cfs/day	4.7	2.7	4.5	7.3	5.8	5.4	8.8	5.8	6.4	4.2
Number of flow reversals	#/year	61	72	69	50	56	45	51	65	48	59

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 5f: Summary of modeled hydrologic parameters for **Mine Year 20** at surface water monitoring location **SW-004a**

Statistic	Units	Water Year									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	10.7	15.6	28.6	145.4	168.5	89.6	6.8	60.7	14.8	3.0
Mean Nov. flow	cfs	3.4	57.2	5.8	9.7	60.9	18.5	7.6	15.9	13.4	8.1
Mean Dec. flow	cfs	3.0	4.7	3.6	5.4	17.0	6.3	17.2	5.4	3.5	3.3
Mean Jan. flow	cfs	2.9	3.3	3.4	4.1	5.7	4.1	3.8	4.1	3.2	3.1
Mean Feb. flow	cfs	2.9	3.1	48.5	3.7	4.3	5.1	3.2	3.6	3.2	3.1
Mean Mar. flow	cfs	40.1	11.4	46.1	14.2	21.5	13.0	75.9	42.5	110.1	31.5
Mean Apr. flow	cfs	196.4	49.6	142.7	118.0	100.4	193.0	93.1	159.3	10.9	55.5
Mean May flow	cfs	23.8	2.3	19.6	69.6	8.1	22.9	34.2	57.5	120.2	12.4
Mean Jun. flow	cfs	10.7	3.3	18.8	3.7	1.9	55.3	104.8	3.8	11.5	2.1
Mean Jul. flow	cfs	3.2	2.5	17.0	55.1	4.7	2.3	16.3	3.7	48.9	2.0
Mean Aug. flow	cfs	2.9	5.0	2.4	16.4	18.5	2.8	11.3	3.4	3.2	107.2
Mean Sep. flow	cfs	4.5	27.1	3.3	42.9	21.4	2.3	79.2	13.7	9.5	20.0
Max. 1-day flow	cfs	794.0	369.0	359.7	561.7	576.2	634.1	486.3	630.2	616.8	414.8
Max. 3-day flow	cfs	723.5	304.4	319.3	497.2	533.1	554.1	381.4	558.9	497.8	342.3
Max. 7-day flow	cfs	635.8	209.3	268.1	355.3	444.3	433.6	253.5	409.1	418.6	232.0
Max. 30-day flow	cfs	213.2	66.6	142.7	162.3	173.7	200.3	109.2	159.3	132.9	117.5
Max. 90-day flow	cfs	87.6	26.1	84.1	85.2	94.1	93.0	83.5	87.6	84.0	43.9
Min. 1-day flow	cfs	1.26	0.94	1.40	1.43	1.02	1.05	1.72	1.23	1.16	0.95
Min. 3-day flow	cfs	1.28	0.94	1.41	1.49	1.04	1.05	1.73	1.33	1.25	1.02
Min. 7-day flow	cfs	1.31	1.02	1.43	1.65	1.08	1.07	1.75	1.40	1.54	1.10
Min. 30-day flow	cfs	2.55	2.09	2.03	3.58	1.85	1.50	2.76	3.23	2.76	1.85
Min. 90-day flow	cfs	2.93	2.64	3.73	4.12	3.35	2.26	4.19	3.77	3.46	3.16
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.052	0.067	0.051	0.040	0.030	0.031	0.046	0.045	0.052	0.052
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	4-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	7	4	9	8	4	13	6	6	7
Number of low pulses[2]	#/year	11	11	6	5	5	4	3	8	7	6
Mean high pulse duration	days	10.4	7.9	23.3	13.3	17.4	23.0	9.8	15.7	14.2	11.0
Mean low pulse duration	days	16.8	11.1	10.5	5.0	13.6	19.0	10.3	4.6	10.6	28.3
Mean rate of flow increase[3]	cfs/day	14.6	11.5	12.6	25.6	18.1	16.5	28.0	15.1	22.0	11.5
Mean rate of flow decrease[4]	cfs/day	4.7	2.7	4.5	7.3	5.8	5.4	8.7	5.8	6.4	4.2
Number of flow reversals	#/year	59	72	71	52	56	47	51	65	48	60

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 5g: Summary of modeled hydrologic parameters for **Mine Facilities off** at surface water monitoring location **SW-004a**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	10.5	15.4	28.5	145.2	168.3	89.4	6.6	60.6	14.6	2.8
Mean Nov. flow	cfs	3.3	57.1	5.7	9.6	60.7	18.4	7.5	15.8	13.2	7.9
Mean Dec. flow	cfs	2.9	4.6	3.5	5.3	16.8	6.2	17.1	5.3	3.4	3.2
Mean Jan. flow	cfs	2.8	3.2	3.3	4.0	5.6	4.0	3.7	4.0	3.1	3.0
Mean Feb. flow	cfs	2.8	3.0	48.3	3.6	4.2	4.9	3.0	3.5	3.1	2.9
Mean Mar. flow	cfs	39.9	11.2	46.0	14.0	21.4	12.9	75.7	42.3	109.9	31.3
Mean Apr. flow	cfs	196.1	49.4	142.5	117.8	100.2	192.8	92.9	159.0	10.8	55.3
Mean May flow	cfs	23.7	2.3	19.5	69.5	8.0	22.7	34.0	57.4	120.0	12.3
Mean Jun. flow	cfs	10.6	3.1	18.8	3.6	1.8	55.1	104.6	3.6	11.3	2.1
Mean Jul. flow	cfs	3.0	2.4	16.8	54.9	4.5	2.2	16.2	3.6	48.7	1.9
Mean Aug. flow	cfs	2.8	4.7	2.3	16.2	18.4	2.7	11.1	3.2	3.1	106.9
Mean Sep. flow	cfs	4.3	26.9	3.2	42.8	21.3	2.3	79.0	13.5	9.4	19.9
Max. 1-day flow	cfs	793.1	368.5	359.1	561.2	575.2	633.5	485.6	629.4	615.8	414.3
Max. 3-day flow	cfs	722.8	304.0	318.9	496.7	532.5	553.7	381.0	558.3	496.9	341.9
Max. 7-day flow	cfs	635.2	209.0	267.8	355.0	443.9	433.3	253.1	408.7	418.1	231.7
Max. 30-day flow	cfs	212.9	66.5	142.5	162.0	173.5	200.1	109.0	159.0	132.6	117.3
Max. 90-day flow	cfs	87.4	26.0	83.9	85.0	94.0	92.8	83.3	87.4	83.8	43.8
Min. 1-day flow	cfs	1.22	0.91	1.34	1.38	0.99	1.01	1.66	1.19	1.12	0.91
Min. 3-day flow	cfs	1.23	0.91	1.36	1.43	1.00	1.01	1.66	1.28	1.20	0.98
Min. 7-day flow	cfs	1.26	0.98	1.37	1.59	1.04	1.03	1.68	1.34	1.49	1.06
Min. 30-day flow	cfs	2.46	2.01	1.95	3.45	1.78	1.45	2.66	3.11	2.65	1.78
Min. 90-day flow	cfs	2.82	2.54	3.59	3.97	3.23	2.17	4.04	3.62	3.33	3.04
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.050	0.065	0.049	0.039	0.029	0.030	0.045	0.043	0.050	0.051
Julian date of max. flow	day	21-Apr	2-Nov	24-Apr	18-Apr	8-Oct	15-Apr	27-Jun	2-Apr	20-May	26-Aug
Julian date of min. flow	day	22-Aug	4-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	2-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	7	4	9	8	4	13	6	6	7
Number of low pulses[2]	#/year	10	11	6	5	5	4	3	8	7	6
Mean high pulse duration	days	10.4	7.7	23.3	13.3	17.4	23.0	9.8	15.7	14.3	11.0
Mean low pulse duration	days	18.6	11.0	10.5	5.0	14.0	19.0	10.3	4.6	10.6	28.3
Mean rate of flow increase[3]	cfs/day	14.4	11.5	12.4	25.2	17.7	16.6	27.6	15.0	21.7	11.4
Mean rate of flow decrease[4]	cfs/day	4.7	2.7	4.5	7.3	5.8	5.4	8.8	5.8	6.4	4.2
Number of flow reversals	#/year	63	72	69	50	56	45	51	67	48	62

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 6a: Summary of modeled hydrologic parameters for **Existing Conditions** at surface water monitoring location **SW-005**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	22.4	19.9	56.7	291.7	326.9	178.6	13.6	121.6	32.7	6.4
Mean Nov. flow	cfs	7.8	129.1	12.6	18.4	117.8	36.3	11.6	29.8	27.6	15.5
Mean Dec. flow	cfs	6.3	10.4	7.3	9.7	31.9	12.5	38.7	9.5	7.4	7.1
Mean Jan. flow	cfs	6.1	7.0	6.9	7.7	9.8	7.8	8.2	7.7	6.7	6.0
Mean Feb. flow	cfs	6.1	6.5	94.5	7.1	8.0	9.4	6.7	7.1	6.8	6.1
Mean Mar. flow	cfs	81.0	16.3	83.9	24.5	41.2	22.9	145.4	56.2	216.5	59.2
Mean Apr. flow	cfs	393.3	105.6	276.2	233.5	194.5	362.7	183.3	300.9	23.0	112.7
Mean May flow	cfs	54.1	4.9	50.4	134.9	18.2	58.2	55.8	143.9	232.6	24.1
Mean Jun. flow	cfs	18.8	6.7	22.7	7.7	3.9	107.9	202.5	7.5	23.3	4.2
Mean Jul. flow	cfs	8.1	5.4	45.5	110.6	9.5	5.5	49.0	7.6	95.6	4.1
Mean Aug. flow	cfs	6.2	8.5	5.2	32.5	32.9	5.7	22.1	7.0	7.8	208.0
Mean Sep. flow	cfs	9.6	54.7	6.1	85.2	46.0	4.6	153.5	26.5	19.9	41.8
Max. 1-day flow	cfs	1369.6	571.5	577.0	930.9	996.5	1051.1	716.0	1114.7	934.9	640.8
Max. 3-day flow	cfs	1351.6	526.0	548.9	860.1	960.8	985.4	641.2	1010.1	819.6	586.8
Max. 7-day flow	cfs	1233.2	400.8	495.3	669.5	854.4	819.6	479.0	776.0	764.5	448.6
Max. 30-day flow	cfs	430.1	135.5	277.7	319.3	337.2	386.7	202.5	300.9	257.4	230.1
Max. 90-day flow	cfs	177.8	53.5	165.2	165.9	182.7	180.8	163.9	169.3	164.1	86.4
Min. 1-day flow	cfs	2.74	2.14	3.02	3.06	2.30	2.36	3.70	2.92	2.66	2.26
Min. 3-day flow	cfs	2.76	2.14	3.02	3.17	2.32	2.37	3.71	3.05	2.82	2.28
Min. 7-day flow	cfs	2.82	2.45	3.07	3.48	2.49	2.40	3.78	3.38	3.22	2.53
Min. 30-day flow	cfs	5.53	4.37	4.31	7.03	3.95	3.29	5.49	6.35	5.61	3.88
Min. 90-day flow	cfs	6.11	5.38	7.38	7.59	6.72	4.74	8.35	7.35	6.98	6.20
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.055	0.079	0.056	0.043	0.035	0.036	0.051	0.056	0.054	0.061
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	16-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	3-Aug	10-Jul	14-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	7	11	9	3	5	3	4	11	8	5
Mean high pulse duration	days	11.2	10.4	23.5	15.4	22.0	23.0	9.9	23.0	14.5	12.7
Mean low pulse duration	days	23.6	11.9	7.8	6.7	14.2	24.0	7.8	3.5	9.3	36.0
Mean rate of flow increase[3]	cfs/day	22.6	12.1	15.9	35.9	25.7	24.0	35.8	22.6	27.9	14.8
Mean rate of flow decrease[4]	cfs/day	7.3	4.5	6.8	12.0	9.6	8.8	13.7	9.6	9.8	6.4
Number of flow reversals	#/year	67	67	61	48	50	49	58	66	52	54

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 6b: Summary of modeled hydrologic parameters for **Mine Year 1** at surface water monitoring location **SW-005**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	22.1	19.5	55.8	287.0	321.9	175.8	13.4	119.9	32.1	6.3
Mean Nov. flow	cfs	7.7	126.9	12.5	18.3	116.0	35.7	11.6	29.4	27.2	15.1
Mean Dec. flow	cfs	6.2	10.2	7.2	9.6	31.5	12.3	38.1	9.4	7.3	7.0
Mean Jan. flow	cfs	6.1	6.9	6.8	7.6	9.6	7.7	8.1	7.6	6.6	6.0
Mean Feb. flow	cfs	6.0	6.4	92.9	7.0	7.8	9.3	6.5	6.9	6.7	6.1
Mean Mar. flow	cfs	79.7	16.0	82.6	24.1	40.5	22.5	142.9	55.3	212.9	58.1
Mean Apr. flow	cfs	386.7	103.9	271.7	229.8	191.4	357.2	180.4	296.2	22.8	111.0
Mean May flow	cfs	53.6	4.9	49.9	132.9	18.0	57.4	55.3	141.9	228.6	23.7
Mean Jun. flow	cfs	18.5	6.5	22.3	7.6	3.9	106.1	198.7	7.4	23.1	4.2
Mean Jul. flow	cfs	8.0	5.4	44.8	108.6	9.4	5.5	48.5	7.5	94.0	4.0
Mean Aug. flow	cfs	6.1	8.4	5.2	31.8	32.2	5.7	21.7	6.9	7.7	204.8
Mean Sep. flow	cfs	9.4	53.8	6.0	83.7	45.3	4.6	150.9	25.8	19.6	41.5
Max. 1-day flow	cfs	1345.6	555.0	565.2	910.3	971.7	1030.2	698.1	1090.8	911.8	625.2
Max. 3-day flow	cfs	1325.8	514.3	537.4	841.5	941.8	967.9	627.0	989.0	800.8	574.0
Max. 7-day flow	cfs	1210.0	393.4	486.7	657.3	839.2	805.6	470.5	761.9	749.5	441.1
Max. 30-day flow	cfs	423.2	133.2	273.2	313.9	332.1	380.7	198.7	296.2	253.2	226.7
Max. 90-day flow	cfs	175.0	52.6	162.6	163.3	179.9	178.0	161.3	166.7	161.4	85.2
Min. 1-day flow	cfs	2.73	2.13	3.01	3.05	2.29	2.35	3.68	2.91	2.65	2.25
Min. 3-day flow	cfs	2.75	2.13	3.01	3.15	2.32	2.36	3.70	3.04	2.81	2.28
Min. 7-day flow	cfs	2.81	2.44	3.06	3.47	2.48	2.39	3.77	3.36	3.21	2.52
Min. 30-day flow	cfs	5.50	4.35	4.30	7.00	3.93	3.27	5.47	6.33	5.59	3.86
Min. 90-day flow	cfs	6.09	5.36	7.35	7.56	6.70	4.72	8.32	7.32	6.96	6.17
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.056	0.080	0.056	0.044	0.036	0.036	0.052	0.056	0.055	0.062
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	3-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	7	11	9	3	5	3	4	11	8	5
Mean high pulse duration	days	11.2	10.4	23.5	15.4	21.8	23.0	9.9	23.0	14.5	12.8
Mean low pulse duration	days	23.6	11.9	7.9	6.7	14.2	24.0	7.8	3.5	9.1	36.0
Mean rate of flow increase[3]	cfs/day	22.2	12.1	15.5	35.5	25.1	24.0	34.6	22.5	27.8	14.4
Mean rate of flow decrease[4]	cfs/day	7.2	4.4	6.7	11.6	9.4	8.5	13.5	9.3	9.5	6.3
Number of flow reversals	#/year	67	65	63	50	50	49	60	68	54	56

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 6c: Summary of modeled hydrologic parameters for **Mine Year 5** at surface water monitoring location **SW-005**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	21.9	19.4	55.4	284.9	319.8	174.5	13.2	119.1	31.8	6.2
Mean Nov. flow	cfs	7.6	126.0	12.4	18.2	115.2	35.5	11.4	29.2	26.9	15.0
Mean Dec. flow	cfs	6.1	10.1	7.1	9.5	31.3	12.2	37.7	9.3	7.2	6.9
Mean Jan. flow	cfs	6.0	6.8	6.7	7.5	9.6	7.6	8.0	7.5	6.5	5.9
Mean Feb. flow	cfs	6.0	6.3	92.1	6.9	7.8	9.2	6.5	6.8	6.6	6.0
Mean Mar. flow	cfs	79.0	15.9	82.0	23.9	40.2	22.3	141.8	54.7	211.3	57.6
Mean Apr. flow	cfs	383.8	103.1	269.8	228.1	190.1	354.6	179.1	294.1	22.7	110.2
Mean May flow	cfs	53.4	4.9	49.7	132.0	17.9	57.1	55.0	141.2	226.9	23.5
Mean Jun. flow	cfs	18.4	6.5	22.1	7.5	3.9	105.3	197.0	7.3	23.0	4.2
Mean Jul. flow	cfs	7.9	5.3	44.5	107.7	9.3	5.5	48.3	7.4	93.2	4.0
Mean Aug. flow	cfs	6.0	8.3	5.2	31.5	31.9	5.7	21.5	6.8	7.6	203.0
Mean Sep. flow	cfs	9.3	53.0	6.0	83.2	45.0	4.5	149.7	25.6	19.3	41.0
Max. 1-day flow	cfs	1335.0	546.9	560.5	901.1	959.3	1021.3	688.7	1079.8	900.5	618.1
Max. 3-day flow	cfs	1314.2	509.3	532.3	833.0	933.1	960.4	620.8	979.6	791.7	568.3
Max. 7-day flow	cfs	1199.7	390.1	482.9	651.8	832.5	799.5	466.6	756.0	742.9	437.9
Max. 30-day flow	cfs	420.1	132.3	271.3	311.7	329.9	378.0	197.0	294.1	251.4	224.8
Max. 90-day flow	cfs	173.7	52.2	161.5	162.2	178.7	176.7	160.1	165.6	160.3	84.4
Min. 1-day flow	cfs	2.71	2.11	2.98	3.02	2.28	2.33	3.66	2.89	2.63	2.23
Min. 3-day flow	cfs	2.73	2.12	2.99	3.13	2.30	2.34	3.67	3.01	2.79	2.26
Min. 7-day flow	cfs	2.79	2.42	3.04	3.44	2.46	2.37	3.74	3.34	3.18	2.50
Min. 30-day flow	cfs	5.46	4.32	4.26	6.95	3.90	3.25	5.43	6.28	5.55	3.83
Min. 90-day flow	cfs	6.04	5.32	7.30	7.50	6.65	4.68	8.25	7.26	6.90	6.13
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.056	0.080	0.056	0.044	0.036	0.036	0.052	0.056	0.055	0.062
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	3-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	7	11	9	3	5	3	4	11	8	6
Mean high pulse duration	days	11.2	10.4	23.5	15.4	22.0	23.0	9.9	23.0	14.5	12.7
Mean low pulse duration	days	23.6	11.9	7.9	6.7	14.2	24.0	8.0	3.5	9.1	29.8
Mean rate of flow increase[3]	cfs/day	22.0	11.9	15.3	35.1	24.6	24.1	33.8	22.2	26.9	14.4
Mean rate of flow decrease[4]	cfs/day	7.1	4.3	6.7	11.5	9.3	8.4	13.3	9.2	9.5	6.2
Number of flow reversals	#/year	67	65	61	50	50	49	60	68	56	55

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 6d: Summary of modeled hydrologic parameters for **Mine Year 10** at surface water monitoring location **SW-005**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	21.7	19.2	55.1	283.3	318.0	173.5	13.2	118.5	31.5	6.1
Mean Nov. flow	cfs	7.5	125.2	12.3	18.1	114.6	35.2	11.3	29.0	26.7	14.8
Mean Dec. flow	cfs	6.0	10.1	7.0	9.4	31.2	12.2	37.5	9.2	7.1	6.8
Mean Jan. flow	cfs	6.0	6.7	6.6	7.4	9.5	7.5	7.9	7.4	6.5	5.9
Mean Feb. flow	cfs	5.9	6.2	91.5	6.8	7.7	9.1	6.4	6.8	6.5	6.0
Mean Mar. flow	cfs	78.5	15.7	81.5	23.7	39.9	22.1	140.9	54.2	210.1	57.2
Mean Apr. flow	cfs	381.5	102.5	268.3	226.8	189.0	352.6	178.1	292.4	22.6	109.7
Mean May flow	cfs	53.2	4.8	49.5	131.2	17.8	56.9	54.7	140.6	225.5	23.3
Mean Jun. flow	cfs	18.2	6.4	21.8	7.4	3.9	104.6	195.6	7.2	22.8	4.1
Mean Jul. flow	cfs	7.8	5.3	44.2	107.0	9.2	5.4	48.2	7.3	92.6	4.0
Mean Aug. flow	cfs	5.9	8.2	5.1	31.3	31.7	5.6	21.3	6.7	7.5	201.7
Mean Sep. flow	cfs	9.2	52.7	5.9	82.6	44.7	4.5	148.8	25.4	19.0	40.8
Max. 1-day flow	cfs	1327.2	540.3	557.2	894.2	949.9	1014.3	680.9	1071.2	892.1	612.5
Max. 3-day flow	cfs	1305.3	505.4	529.2	826.2	926.7	954.8	615.8	972.2	785.8	563.5
Max. 7-day flow	cfs	1191.7	387.3	479.9	647.5	827.3	794.9	463.6	751.3	737.6	435.1
Max. 30-day flow	cfs	417.8	131.5	269.9	309.8	328.1	375.9	195.6	292.4	250.0	223.4
Max. 90-day flow	cfs	172.7	51.9	160.6	161.2	177.7	175.7	159.2	164.7	159.4	83.9
Min. 1-day flow	cfs	2.68	2.09	2.96	3.00	2.26	2.31	3.63	2.87	2.61	2.21
Min. 3-day flow	cfs	2.71	2.10	2.96	3.10	2.28	2.32	3.64	2.99	2.76	2.24
Min. 7-day flow	cfs	2.76	2.40	3.01	3.41	2.44	2.35	3.71	3.31	3.16	2.48
Min. 30-day flow	cfs	5.42	4.28	4.23	6.89	3.87	3.22	5.38	6.23	5.50	3.80
Min. 90-day flow	cfs	5.99	5.28	7.24	7.44	6.59	4.64	8.18	7.20	6.84	6.07
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.055	0.080	0.056	0.043	0.036	0.036	0.052	0.056	0.055	0.062
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	3-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	7	11	9	3	5	3	4	10	7	6
Mean high pulse duration	days	11.2	10.4	23.5	15.4	22.0	23.0	9.9	23.0	14.5	12.7
Mean low pulse duration	days	23.6	11.9	7.9	6.7	14.2	24.0	8.3	3.7	10.3	29.8
Mean rate of flow increase[3]	cfs/day	21.6	11.8	15.4	35.2	24.4	23.9	33.2	22.5	26.9	14.1
Mean rate of flow decrease[4]	cfs/day	7.0	4.3	6.6	11.4	9.2	8.4	13.2	9.1	9.3	6.1
Number of flow reversals	#/year	67	65	63	50	50	49	60	68	56	56

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 6e: Summary of modeled hydrologic parameters for **Mine Year 15** at surface water monitoring location **SW-005**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	21.6	19.2	55.0	282.7	317.4	173.1	13.2	118.3	31.4	6.1
Mean Nov. flow	cfs	7.5	124.9	12.3	18.1	114.4	35.2	11.3	29.0	26.7	14.9
Mean Dec. flow	cfs	6.0	10.1	7.0	9.4	31.1	12.2	37.4	9.2	7.1	6.8
Mean Jan. flow	cfs	6.0	6.7	6.6	7.4	9.5	7.5	7.9	7.4	6.4	5.9
Mean Feb. flow	cfs	5.9	6.2	91.3	6.8	7.7	9.1	6.4	6.8	6.5	6.0
Mean Mar. flow	cfs	78.3	15.7	81.3	23.7	39.8	22.1	140.5	54.1	209.7	57.1
Mean Apr. flow	cfs	380.7	102.3	267.7	226.4	188.6	351.8	177.8	291.8	22.6	109.4
Mean May flow	cfs	53.1	4.8	49.5	130.9	17.8	56.8	54.6	140.4	225.0	23.3
Mean Jun. flow	cfs	18.1	6.4	21.8	7.4	3.9	104.3	195.1	7.2	22.8	4.1
Mean Jul. flow	cfs	7.8	5.3	44.1	106.8	9.2	5.4	48.1	7.3	92.4	4.0
Mean Aug. flow	cfs	5.9	8.2	5.1	31.3	31.6	5.6	21.3	6.7	7.5	201.3
Mean Sep. flow	cfs	9.2	52.7	6.0	82.5	44.6	4.5	148.4	25.4	19.0	40.7
Max. 1-day flow	cfs	1324.1	537.3	555.8	891.3	945.5	1011.6	677.6	1067.7	888.0	610.1
Max. 3-day flow	cfs	1301.7	503.8	528.2	823.5	923.9	952.6	613.8	969.3	783.8	561.6
Max. 7-day flow	cfs	1188.6	386.3	478.8	645.8	825.2	793.1	462.5	749.6	735.4	434.2
Max. 30-day flow	cfs	416.9	131.2	269.3	309.2	327.5	375.2	195.1	291.8	249.4	223.0
Max. 90-day flow	cfs	172.3	51.8	160.2	160.9	177.4	175.4	158.8	164.3	159.0	83.7
Min. 1-day flow	cfs	2.69	2.09	2.96	3.00	2.26	2.31	3.63	2.87	2.61	2.21
Min. 3-day flow	cfs	2.71	2.10	2.97	3.10	2.28	2.32	3.64	2.99	2.77	2.24
Min. 7-day flow	cfs	2.77	2.40	3.01	3.41	2.44	2.35	3.71	3.31	3.16	2.48
Min. 30-day flow	cfs	5.42	4.28	4.23	6.89	3.87	3.22	5.38	6.23	5.50	3.80
Min. 90-day flow	cfs	6.00	5.28	7.24	7.44	6.59	4.65	8.19	7.20	6.85	6.08
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.056	0.080	0.056	0.044	0.036	0.036	0.052	0.056	0.055	0.062
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	3-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	7	11	9	3	5	3	4	10	8	5
Mean high pulse duration	days	11.2	10.4	23.5	15.4	22.0	23.0	9.9	23.0	14.5	12.7
Mean low pulse duration	days	23.6	12.0	7.9	6.7	14.2	24.0	8.3	3.7	9.1	35.6
Mean rate of flow increase[3]	cfs/day	21.6	11.9	15.2	35.1	24.5	23.8	33.0	22.4	26.8	14.1
Mean rate of flow decrease[4]	cfs/day	7.0	4.3	6.6	11.3	9.1	8.3	13.2	9.1	9.3	6.1
Number of flow reversals	#/year	67	63	65	50	48	49	60	68	56	56

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 6f: Summary of modeled hydrologic parameters for **Mine Year 20** at surface water monitoring location **SW-005**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	21.6	19.1	54.9	282.7	317.4	173.1	13.1	118.2	31.3	6.1
Mean Nov. flow	cfs	7.4	124.9	12.2	18.0	114.3	35.1	11.2	28.9	26.6	14.8
Mean Dec. flow	cfs	5.9	10.0	6.9	9.3	31.0	12.1	37.3	9.1	7.0	6.7
Mean Jan. flow	cfs	5.9	6.6	6.5	7.4	9.4	7.4	7.8	7.3	6.4	5.8
Mean Feb. flow	cfs	5.9	6.1	91.2	6.7	7.6	9.0	6.3	6.7	6.4	5.9
Mean Mar. flow	cfs	78.3	15.6	81.3	23.6	39.8	22.0	140.5	54.0	209.6	57.0
Mean Apr. flow	cfs	380.7	102.2	267.6	226.3	188.5	351.8	177.7	291.8	22.5	109.3
Mean May flow	cfs	53.0	4.8	49.4	130.9	17.7	56.8	54.6	140.3	225.0	23.2
Mean Jun. flow	cfs	18.1	6.3	21.6	7.3	3.8	104.3	195.0	7.1	22.7	4.1
Mean Jul. flow	cfs	7.7	5.2	44.0	106.7	9.1	5.3	48.1	7.2	92.4	3.9
Mean Aug. flow	cfs	5.8	8.2	5.1	31.1	31.5	5.5	21.2	6.6	7.4	201.3
Mean Sep. flow	cfs	9.2	52.7	5.9	82.4	44.5	4.4	148.4	25.3	19.0	40.7
Max. 1-day flow	cfs	1324.1	537.5	555.8	891.4	945.8	1011.6	677.8	1067.7	888.5	610.2
Max. 3-day flow	cfs	1301.8	503.9	528.2	823.6	924.0	952.7	613.9	969.3	784.0	561.7
Max. 7-day flow	cfs	1188.7	386.3	478.7	645.9	825.3	793.1	462.4	749.6	735.6	434.1
Max. 30-day flow	cfs	416.9	131.1	269.2	309.1	327.4	375.1	195.0	291.8	249.4	222.9
Max. 90-day flow	cfs	172.3	51.7	160.2	160.8	177.3	175.3	158.8	164.3	159.0	83.7
Min. 1-day flow	cfs	2.65	2.06	2.92	2.96	2.23	2.28	3.57	2.83	2.57	2.18
Min. 3-day flow	cfs	2.67	2.07	2.92	3.06	2.25	2.29	3.59	2.95	2.73	2.21
Min. 7-day flow	cfs	2.73	2.37	2.97	3.37	2.40	2.32	3.66	3.26	3.11	2.45
Min. 30-day flow	cfs	5.34	4.22	4.17	6.79	3.81	3.18	5.31	6.14	5.43	3.75
Min. 90-day flow	cfs	5.91	5.20	7.14	7.34	6.50	4.58	8.07	7.10	6.75	5.99
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.055	0.079	0.056	0.043	0.035	0.036	0.051	0.056	0.054	0.061
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	3-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	7	11	9	3	5	3	4	9	8	5
Mean high pulse duration	days	11.2	10.4	23.5	15.4	22.0	23.0	9.9	23.0	14.5	12.7
Mean low pulse duration	days	23.6	12.0	8.0	6.7	14.2	24.0	8.3	4.0	9.1	35.6
Mean rate of flow increase[3]	cfs/day	21.6	11.8	15.2	35.1	24.3	23.8	33.1	22.4	26.8	14.1
Mean rate of flow decrease[4]	cfs/day	7.0	4.3	6.6	11.3	9.2	8.3	13.2	9.1	9.3	6.2
Number of flow reversals	#/year	67	65	65	50	50	49	60	68	56	55

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 6g: Summary of modeled hydrologic parameters for **Mine Facilities Off** at surface water monitoring location **SW-005**

Statistic	Units	Water Year									
		1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Mean Oct. flow	cfs	21.4	19.0	54.7	282.4	317.1	172.9	12.9	118.0	31.2	5.9
Mean Nov. flow	cfs	7.3	124.7	12.1	17.9	114.2	35.0	11.1	28.8	26.5	14.6
Mean Dec. flow	cfs	5.8	9.9	6.8	9.2	30.9	12.0	37.2	9.0	6.9	6.6
Mean Jan. flow	cfs	5.7	6.5	6.4	7.2	9.3	7.3	7.7	7.2	6.2	5.7
Mean Feb. flow	cfs	5.7	6.0	91.1	6.6	7.5	8.9	6.2	6.6	6.3	5.8
Mean Mar. flow	cfs	78.1	15.5	81.1	23.5	39.6	21.9	140.3	53.8	209.4	56.8
Mean Apr. flow	cfs	380.4	102.1	267.4	226.1	188.3	351.5	177.5	291.5	22.4	109.2
Mean May flow	cfs	52.9	4.7	49.3	130.7	17.6	56.6	54.4	140.2	224.8	23.1
Mean Jun. flow	cfs	17.9	6.1	21.5	7.2	3.7	104.1	194.8	7.0	22.6	4.0
Mean Jul. flow	cfs	7.6	5.1	43.9	106.6	9.0	5.2	47.9	7.1	92.2	3.8
Mean Aug. flow	cfs	5.7	8.0	4.9	31.0	31.4	5.4	21.1	6.5	7.3	201.0
Mean Sep. flow	cfs	9.0	52.5	5.7	82.2	44.4	4.3	148.2	25.1	18.8	40.5
Max. 1-day flow	cfs	1323.5	536.9	555.4	890.6	945.0	1011.1	677.0	1067.1	887.4	609.7
Max. 3-day flow	cfs	1301.2	503.5	527.9	822.9	923.4	952.2	613.4	968.7	783.4	561.2
Max. 7-day flow	cfs	1188.1	386.0	478.4	645.4	824.8	792.7	462.1	749.2	735.0	433.8
Max. 30-day flow	cfs	416.6	131.0	269.0	308.9	327.1	374.9	194.8	291.5	249.1	222.7
Max. 90-day flow	cfs	172.1	51.6	160.0	160.6	177.1	175.1	158.6	164.1	158.8	83.5
Min. 1-day flow	cfs	2.59	2.02	2.85	2.89	2.18	2.22	3.49	2.76	2.51	2.13
Min. 3-day flow	cfs	2.61	2.02	2.86	2.99	2.20	2.24	3.51	2.88	2.66	2.16
Min. 7-day flow	cfs	2.66	2.31	2.90	3.29	2.35	2.26	3.57	3.19	3.04	2.39
Min. 30-day flow	cfs	5.22	4.12	4.07	6.64	3.73	3.11	5.19	6.00	5.30	3.66
Min. 90-day flow	cfs	5.78	5.08	6.98	7.17	6.35	4.48	7.89	6.94	6.60	5.85
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.054	0.078	0.055	0.042	0.035	0.035	0.050	0.055	0.053	0.060
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	3-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	7	11	9	3	5	3	4	11	7	6
Mean high pulse duration	days	11.2	10.4	23.5	15.4	22.0	23.0	9.9	23.0	14.5	12.7
Mean low pulse duration	days	23.6	11.9	7.9	6.7	14.2	24.0	8.3	3.5	10.3	29.8
Mean rate of flow increase[3]	cfs/day	21.6	11.9	15.2	35.1	24.2	23.8	33.0	22.4	26.8	14.3
Mean rate of flow decrease[4]	cfs/day	7.0	4.3	6.6	11.3	9.2	8.3	13.2	9.1	9.3	6.1
Number of flow reversals	#/year	67	65	65	50	50	49	60	68	56	56

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 7a: Summary of modeled hydrologic parameters for **Existing Conditions** at **USGS gaging station #04015475**

Statistic	Units	Water Year									
		1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Mean Oct. flow	cfs	23.6	24.0	59.7	306.4	343.0	187.7	14.7	126.8	34.2	6.8
Mean Nov. flow	cfs	8.2	132.9	13.0	18.7	123.3	37.9	12.4	31.0	29.1	16.5
Mean Dec. flow	cfs	6.6	10.7	7.6	10.0	32.9	12.8	40.3	9.8	7.7	7.4
Mean Jan. flow	cfs	6.4	7.3	7.3	8.0	10.1	8.1	8.5	8.0	7.1	6.3
Mean Feb. flow	cfs	6.4	6.8	99.7	7.4	8.3	9.9	7.0	7.4	7.2	6.5
Mean Mar. flow	cfs	85.5	18.3	89.2	26.4	43.4	24.3	153.7	63.4	227.7	62.6
Mean Apr. flow	cfs	414.3	109.7	290.1	244.7	204.6	382.2	192.1	315.7	23.4	117.8
Mean May flow	cfs	55.8	5.3	51.1	141.7	18.7	59.1	61.4	146.7	245.0	25.5
Mean Jun. flow	cfs	20.2	7.5	26.2	8.3	4.5	114.2	211.3	8.3	24.1	4.8
Mean Jul. flow	cfs	8.9	6.0	46.6	117.1	10.5	5.9	50.0	8.5	101.5	4.6
Mean Aug. flow	cfs	6.7	9.9	5.7	34.6	35.0	6.4	23.5	7.8	8.2	219.9
Mean Sep. flow	cfs	10.3	58.0	7.0	89.8	48.1	5.0	162.2	28.8	21.3	43.7
Max. 1-day flow	cfs	1444.0	587.1	592.1	958.1	1033.3	1104.7	735.3	1150.7	953.0	649.7
Max. 3-day flow	cfs	1418.4	537.3	571.0	890.3	1001.6	1028.6	652.1	1053.0	850.9	600.7
Max. 7-day flow	cfs	1292.4	420.0	515.5	698.8	892.9	857.0	501.2	810.5	794.2	470.6
Max. 30-day flow	cfs	451.8	142.5	291.5	335.4	353.7	405.0	211.3	315.7	270.7	242.4
Max. 90-day flow	cfs	187.0	56.3	173.4	174.2	191.7	189.9	172.1	177.8	172.4	91.3
Min. 1-day flow	cfs	3.00	2.36	3.30	3.34	2.54	2.61	3.93	3.26	2.92	2.49
Min. 3-day flow	cfs	3.03	2.36	3.32	3.49	2.56	2.63	3.94	3.33	3.15	2.53
Min. 7-day flow	cfs	3.09	2.67	3.36	3.86	2.73	2.66	4.02	3.77	3.63	2.96
Min. 30-day flow	cfs	5.93	4.90	4.70	7.35	4.39	3.60	6.21	7.08	6.18	4.36
Min. 90-day flow	cfs	6.42	6.00	7.73	7.93	7.45	5.21	9.02	7.90	7.33	6.53
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.057	0.082	0.058	0.045	0.037	0.038	0.052	0.059	0.058	0.068
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	6-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	8	11	10	2	5	3	4	9	8	6
Mean high pulse duration	days	11.0	10.6	23.5	15.1	22.2	23.0	10.0	22.5	14.7	12.7
Mean low pulse duration	days	20.8	11.6	7.6	10.0	13.4	23.7	8.3	4.1	9.5	29.7
Mean rate of flow increase[3]	cfs/day	27.3	15.4	17.6	36.3	26.0	25.4	36.5	23.7	28.8	15.0
Mean rate of flow decrease[4]	cfs/day	7.3	4.4	6.8	12.3	9.8	9.2	14.0	9.7	9.8	6.5
Number of flow reversals	#/year	68	79	63	48	50	49	54	68	52	58

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 7b: Summary of modeled hydrologic parameters for **Mine Year 1** at **USGS gaging station #04015475**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	23.2	23.7	58.9	301.7	338.0	184.9	14.5	125.1	33.7	6.7
Mean Nov. flow	cfs	8.0	130.7	12.9	18.6	121.5	37.3	12.3	30.6	28.6	16.2
Mean Dec. flow	cfs	6.5	10.6	7.5	9.9	32.5	12.7	39.7	9.7	7.6	7.3
Mean Jan. flow	cfs	6.4	7.2	7.1	7.9	10.0	8.0	8.4	7.9	7.0	6.3
Mean Feb. flow	cfs	6.3	6.7	98.1	7.3	8.2	9.8	6.9	7.2	7.0	6.3
Mean Mar. flow	cfs	84.1	18.0	87.9	26.0	42.7	23.9	151.2	62.4	224.1	61.6
Mean Apr. flow	cfs	407.7	108.0	285.6	240.9	201.5	376.7	189.2	310.9	23.2	116.1
Mean May flow	cfs	55.3	5.3	50.6	139.6	18.5	58.4	60.9	144.7	241.0	25.1
Mean Jun. flow	cfs	19.9	7.3	25.8	8.2	4.4	112.3	207.5	8.2	24.0	4.8
Mean Jul. flow	cfs	8.8	6.0	45.9	115.0	10.4	5.9	49.5	8.4	99.8	4.6
Mean Aug. flow	cfs	6.6	9.7	5.7	33.9	34.4	6.4	23.2	7.7	8.1	216.7
Mean Sep. flow	cfs	10.2	57.1	6.9	88.3	47.4	5.0	159.6	28.1	21.0	43.4
Max. 1-day flow	cfs	1419.9	570.6	580.2	937.4	1008.6	1085.7	717.4	1126.7	929.7	634.1
Max. 3-day flow	cfs	1392.6	525.5	559.4	871.8	982.6	1009.9	638.0	1031.8	836.1	587.9
Max. 7-day flow	cfs	1269.1	412.6	506.9	686.5	877.7	843.0	492.6	796.4	779.2	463.1
Max. 30-day flow	cfs	445.0	140.2	287.1	330.0	348.6	399.0	207.5	310.9	266.4	239.0
Max. 90-day flow	cfs	184.1	55.4	170.8	171.6	188.8	187.1	169.5	175.1	169.7	90.1
Min. 1-day flow	cfs	2.99	2.35	3.29	3.33	2.53	2.60	3.91	3.25	2.91	2.48
Min. 3-day flow	cfs	3.02	2.35	3.31	3.48	2.56	2.62	3.93	3.32	3.14	2.53
Min. 7-day flow	cfs	3.08	2.66	3.35	3.85	2.72	2.65	4.00	3.75	3.62	2.95
Min. 30-day flow	cfs	5.91	4.88	4.68	7.32	4.37	3.58	6.18	7.06	6.16	4.35
Min. 90-day flow	cfs	6.40	5.97	7.70	7.90	7.42	5.19	8.99	7.87	7.30	6.50
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.058	0.083	0.059	0.046	0.037	0.038	0.052	0.060	0.059	0.068
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	6-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	9	12	10	3	5	3	4	8	9	6
Mean high pulse duration	days	11.0	10.4	23.5	15.1	22.2	23.0	10.0	22.5	14.8	12.7
Mean low pulse duration	days	18.6	10.8	7.6	6.7	13.4	23.7	8.3	4.4	8.4	29.5
Mean rate of flow increase[3]	cfs/day	27.2	15.5	17.5	35.5	25.7	24.9	34.9	23.6	27.5	14.6
Mean rate of flow decrease[4]	cfs/day	7.2	4.3	6.6	12.0	9.6	9.0	13.7	9.4	9.7	6.3
Number of flow reversals	#/year	71	77	63	48	48	49	56	66	54	58

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 7c: Summary of modeled hydrologic parameters for **Mine Year 5** at **USGS gaging station #04015475**

Statistic	Units	Water Year									
		1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Mean Oct. flow	cfs	23.0	23.5	58.5	299.6	335.8	183.6	14.4	124.3	33.4	6.6
Mean Nov. flow	cfs	7.9	129.8	12.8	18.5	120.7	37.1	12.2	30.4	28.4	16.1
Mean Dec. flow	cfs	6.4	10.5	7.4	9.8	32.4	12.6	39.3	9.6	7.5	7.2
Mean Jan. flow	cfs	6.3	7.1	7.1	7.8	9.9	7.9	8.3	7.8	6.9	6.2
Mean Feb. flow	cfs	6.3	6.6	97.4	7.2	8.1	9.7	6.8	7.2	7.0	6.2
Mean Mar. flow	cfs	83.5	17.9	87.3	25.8	42.4	23.7	150.0	61.9	222.6	61.1
Mean Apr. flow	cfs	404.8	107.2	283.7	239.3	200.1	374.1	187.9	308.8	23.1	115.3
Mean May flow	cfs	55.1	5.2	50.4	138.7	18.4	58.1	60.5	144.0	239.3	24.9
Mean Jun. flow	cfs	19.7	7.3	25.6	8.1	4.4	111.5	205.8	8.1	23.9	4.7
Mean Jul. flow	cfs	8.7	5.9	45.5	114.2	10.3	5.8	49.3	8.3	99.1	4.5
Mean Aug. flow	cfs	6.5	9.6	5.7	33.6	34.1	6.3	23.0	7.6	8.1	215.0
Mean Sep. flow	cfs	10.1	56.3	6.8	87.8	47.1	5.0	158.4	27.9	20.7	42.9
Max. 1-day flow	cfs	1409.3	562.4	575.3	928.1	996.2	1076.8	708.0	1115.7	918.3	626.9
Max. 3-day flow	cfs	1380.9	520.5	554.4	863.1	973.8	1001.3	631.3	1022.4	829.6	582.2
Max. 7-day flow	cfs	1258.8	409.3	503.1	681.1	871.0	836.9	488.8	790.4	772.5	459.9
Max. 30-day flow	cfs	441.9	139.3	285.2	327.8	346.4	396.3	205.8	308.8	264.6	237.1
Max. 90-day flow	cfs	182.9	55.0	169.7	170.5	187.6	185.9	168.3	174.0	168.6	89.3
Min. 1-day flow	cfs	2.97	2.33	3.27	3.31	2.51	2.59	3.89	3.23	2.89	2.46
Min. 3-day flow	cfs	2.99	2.33	3.29	3.45	2.54	2.60	3.90	3.29	3.11	2.51
Min. 7-day flow	cfs	3.05	2.64	3.33	3.82	2.70	2.63	3.97	3.73	3.59	2.93
Min. 30-day flow	cfs	5.87	4.85	4.65	7.27	4.34	3.56	6.14	7.01	6.12	4.32
Min. 90-day flow	cfs	6.36	5.93	7.65	7.85	7.37	5.16	8.93	7.81	7.25	6.46
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.058	0.083	0.059	0.046	0.037	0.038	0.052	0.060	0.059	0.069
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	6-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	9	12	10	3	5	3	4	8	9	6
Mean high pulse duration	days	11.0	10.4	23.5	15.1	22.2	23.0	10.0	22.5	14.8	12.7
Mean low pulse duration	days	18.6	10.9	7.5	6.7	13.4	23.7	8.3	4.5	8.4	29.5
Mean rate of flow increase[3]	cfs/day	27.0	14.9	17.3	35.2	24.9	24.7	34.5	23.2	26.9	14.4
Mean rate of flow decrease[4]	cfs/day	7.1	4.3	6.5	11.9	9.6	8.9	13.6	9.4	9.6	6.3
Number of flow reversals	#/year	70	75	63	48	50	49	58	66	54	58

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 7d: Summary of modeled hydrologic parameters for **Mine Year 10** at **USGS gaging station #04015475**

Statistic	Units	Water Year									
		1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Mean Oct. flow	cfs	22.8	23.3	58.2	298.0	334.1	182.6	14.3	123.7	33.1	6.5
Mean Nov. flow	cfs	7.9	129.1	12.7	18.5	120.1	36.9	12.1	30.2	28.2	15.9
Mean Dec. flow	cfs	6.3	10.4	7.4	9.7	32.2	12.5	39.1	9.5	7.4	7.1
Mean Jan. flow	cfs	6.3	7.0	7.0	7.8	9.8	7.8	8.2	7.7	6.8	6.2
Mean Feb. flow	cfs	6.3	6.5	96.7	7.1	8.0	9.6	6.7	7.1	6.9	6.2
Mean Mar. flow	cfs	83.0	17.7	86.8	25.7	42.2	23.5	149.1	61.4	221.3	60.7
Mean Apr. flow	cfs	402.6	106.6	282.1	238.0	199.0	372.1	186.9	307.2	23.0	114.8
Mean May flow	cfs	54.8	5.2	50.2	137.9	18.3	57.8	60.2	143.4	238.0	24.7
Mean Jun. flow	cfs	19.5	7.2	25.3	8.0	4.4	110.8	204.4	8.0	23.7	4.7
Mean Jul. flow	cfs	8.6	5.9	45.2	113.5	10.2	5.8	49.2	8.2	98.5	4.5
Mean Aug. flow	cfs	6.4	9.5	5.6	33.4	33.8	6.3	22.8	7.5	8.0	213.7
Mean Sep. flow	cfs	10.0	56.0	6.7	87.2	46.8	5.0	157.5	27.7	20.5	42.7
Max. 1-day flow	cfs	1401.4	555.8	572.0	921.0	986.7	1069.9	700.2	1107.1	909.7	621.3
Max. 3-day flow	cfs	1372.0	516.5	551.1	856.4	967.3	994.8	626.2	1015.0	824.6	577.4
Max. 7-day flow	cfs	1250.8	406.6	500.1	676.8	865.8	832.2	485.7	785.8	767.2	457.1
Max. 30-day flow	cfs	439.5	138.5	283.7	325.9	344.6	394.2	204.4	307.2	263.2	235.7
Max. 90-day flow	cfs	181.9	54.7	168.8	169.5	186.7	184.9	167.4	173.1	167.6	88.8
Min. 1-day flow	cfs	2.95	2.31	3.24	3.28	2.50	2.57	3.86	3.21	2.87	2.45
Min. 3-day flow	cfs	2.97	2.32	3.26	3.43	2.52	2.58	3.87	3.27	3.09	2.49
Min. 7-day flow	cfs	3.03	2.62	3.30	3.79	2.68	2.61	3.94	3.70	3.56	2.90
Min. 30-day flow	cfs	5.82	4.81	4.61	7.22	4.31	3.53	6.09	6.96	6.07	4.28
Min. 90-day flow	cfs	6.31	5.89	7.59	7.79	7.32	5.12	8.86	7.75	7.20	6.41
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.058	0.083	0.059	0.046	0.037	0.038	0.052	0.060	0.059	0.068
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	9-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	6-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	9	12	10	3	5	3	4	8	9	6
Mean high pulse duration	days	11.0	10.4	23.5	15.1	22.2	23.0	10.0	22.5	14.8	12.7
Mean low pulse duration	days	18.6	10.9	7.6	6.7	13.4	23.7	8.3	4.4	8.4	29.5
Mean rate of flow increase[3]	cfs/day	26.8	14.8	17.2	34.9	24.7	24.5	34.2	23.2	26.9	14.3
Mean rate of flow decrease[4]	cfs/day	7.1	4.2	6.5	11.8	9.5	8.9	13.4	9.3	9.5	6.2
Number of flow reversals	#/year	71	75	63	48	50	49	58	66	54	60

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 7e: Summary of modeled hydrologic parameters for **Mine Year 15** at **USGS gaging station #04015475**

Statistic	**Units**	**Water Year**									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	22.8	23.3	58.1	297.4	333.5	182.2	14.3	123.5	33.0	6.5
Mean Nov. flow	cfs	7.8	128.8	12.7	18.5	119.9	36.8	12.1	30.2	28.1	16.0
Mean Dec. flow	cfs	6.3	10.4	7.3	9.7	32.2	12.5	39.0	9.5	7.4	7.1
Mean Jan. flow	cfs	6.3	7.0	7.0	7.7	9.8	7.8	8.2	7.7	6.8	6.2
Mean Feb. flow	cfs	6.3	6.5	96.5	7.1	8.0	9.5	6.7	7.1	6.9	6.2
Mean Mar. flow	cfs	82.8	17.7	86.6	25.6	42.1	23.5	148.8	61.2	220.9	60.5
Mean Apr. flow	cfs	401.8	106.4	281.6	237.5	198.6	371.3	186.6	306.5	23.0	114.5
Mean May flow	cfs	54.8	5.2	50.2	137.7	18.3	57.8	60.2	143.2	237.5	24.7
Mean Jun. flow	cfs	19.5	7.2	25.3	8.0	4.4	110.6	203.9	8.0	23.7	4.7
Mean Jul. flow	cfs	8.6	5.9	45.1	113.2	10.2	5.8	49.2	8.2	98.3	4.5
Mean Aug. flow	cfs	6.4	9.5	5.6	33.4	33.8	6.3	22.7	7.5	8.0	213.3
Mean Sep. flow	cfs	10.0	56.0	6.7	87.0	46.7	5.0	157.1	27.7	20.5	42.6
Max. 1-day flow	cfs	1398.3	552.8	570.6	918.1	982.6	1067.1	696.8	1103.6	905.6	618.9
Max. 3-day flow	cfs	1368.4	514.9	550.2	853.6	964.6	992.1	624.3	1012.1	822.6	575.5
Max. 7-day flow	cfs	1247.7	405.6	499.0	675.1	863.7	830.4	484.6	784.0	765.1	456.2
Max. 30-day flow	cfs	438.7	138.2	283.1	325.3	344.0	393.4	203.9	306.5	262.6	235.2
Max. 90-day flow	cfs	181.5	54.6	168.4	169.2	186.3	184.5	167.0	172.7	167.3	88.6
Min. 1-day flow	cfs	2.95	2.31	3.24	3.28	2.50	2.57	3.86	3.21	2.87	2.45
Min. 3-day flow	cfs	2.97	2.32	3.26	3.43	2.52	2.58	3.87	3.27	3.09	2.49
Min. 7-day flow	cfs	3.03	2.62	3.30	3.80	2.68	2.61	3.94	3.70	3.57	2.91
Min. 30-day flow	cfs	5.83	4.81	4.62	7.22	4.31	3.53	6.10	6.96	6.07	4.29
Min. 90-day flow	cfs	6.31	5.89	7.59	7.79	7.32	5.12	8.86	7.76	7.20	6.41
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.058	0.083	0.059	0.046	0.037	0.038	0.052	0.060	0.059	0.069
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	10-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	6-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	9	11	10	3	5	3	4	8	9	6
Mean high pulse duration	days	11.0	10.4	23.5	15.1	22.2	23.0	10.0	22.5	14.8	12.7
Mean low pulse duration	days	18.6	11.7	7.7	6.7	13.4	23.7	8.5	4.3	8.7	29.5
Mean rate of flow increase[3]	cfs/day	26.8	14.7	17.1	34.8	24.4	24.5	34.0	22.9	26.8	14.3
Mean rate of flow decrease[4]	cfs/day	7.1	4.2	6.5	11.8	9.5	8.8	13.4	9.3	9.4	6.2
Number of flow reversals	#/year	71	73	63	48	50	49	58	68	54	58

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 7f: Summary of modeled hydrologic parameters for **Mine Year 20** at **USGS gaging station #04015475**

Statistic	Units	Water Year									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	22.7	23.3	58.0	297.4	333.4	182.2	14.2	123.4	32.9	6.4
Mean Nov. flow	cfs	7.8	128.7	12.6	18.4	119.8	36.7	12.0	30.1	28.1	15.9
Mean Dec. flow	cfs	6.2	10.3	7.3	9.6	32.1	12.4	38.9	9.4	7.4	7.0
Mean Jan. flow	cfs	6.2	6.9	6.9	7.7	9.7	7.8	8.1	7.6	6.7	6.1
Mean Feb. flow	cfs	6.2	6.4	96.5	7.0	7.9	9.5	6.6	7.0	6.8	6.1
Mean Mar. flow	cfs	82.7	17.6	86.5	25.5	42.0	23.4	148.7	61.1	220.8	60.5
Mean Apr. flow	cfs	401.7	106.3	281.5	237.5	198.6	371.3	186.5	306.5	22.9	114.5
Mean May flow	cfs	54.7	5.1	50.1	137.6	18.2	57.7	60.1	143.1	237.4	24.6
Mean Jun. flow	cfs	19.4	7.1	25.1	7.9	4.3	110.5	203.8	7.9	23.6	4.6
Mean Jul. flow	cfs	8.5	5.8	45.0	113.2	10.1	5.7	49.1	8.1	98.2	4.5
Mean Aug. flow	cfs	6.4	9.5	5.6	33.2	33.6	6.2	22.7	7.4	7.9	213.3
Mean Sep. flow	cfs	9.9	55.9	6.7	87.0	46.6	4.9	157.0	27.7	20.4	42.6
Max. 1-day flow	cfs	1398.3	553.0	570.6	918.2	982.7	1067.2	697.0	1103.7	906.1	619.0
Max. 3-day flow	cfs	1368.5	515.0	550.1	853.7	964.6	992.2	624.4	1012.1	822.8	575.6
Max. 7-day flow	cfs	1247.8	405.5	498.9	675.1	863.8	830.5	484.6	784.0	765.3	456.1
Max. 30-day flow	cfs	438.6	138.1	283.1	325.2	343.9	393.4	203.8	306.5	262.6	235.2
Max. 90-day flow	cfs	181.5	54.5	168.4	169.2	186.3	184.4	167.0	172.7	167.3	88.6
Min. 1-day flow	cfs	2.91	2.28	3.20	3.24	2.46	2.53	3.81	3.16	2.83	2.41
Min. 3-day flow	cfs	2.93	2.29	3.22	3.38	2.49	2.55	3.82	3.23	3.05	2.46
Min. 7-day flow	cfs	2.99	2.59	3.26	3.75	2.65	2.58	3.89	3.65	3.52	2.87
Min. 30-day flow	cfs	5.75	4.75	4.56	7.13	4.25	3.49	6.02	6.87	5.99	4.23
Min. 90-day flow	cfs	6.23	5.81	7.49	7.69	7.22	5.05	8.75	7.66	7.11	6.33
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.057	0.082	0.058	0.045	0.037	0.038	0.052	0.059	0.058	0.068
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	10-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	6-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	5	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	9	11	10	3	5	3	4	8	9	6
Mean high pulse duration	days	11.0	10.4	23.5	15.1	22.2	23.0	10.0	22.5	14.8	12.7
Mean low pulse duration	days	18.6	11.7	7.7	6.7	13.4	23.7	8.5	4.4	8.6	29.5
Mean rate of flow increase[3]	cfs/day	26.8	14.6	17.0	34.8	24.4	24.5	34.0	22.7	26.8	14.1
Mean rate of flow decrease[4]	cfs/day	7.1	4.3	6.5	11.8	9.5	8.8	13.4	9.3	9.4	6.2
Number of flow reversals	#/year	70	73	65	48	50	49	58	70	54	60

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Table 7g: Summary of modeled hydrologic parameters for **Mine Facilities Off** at **USGS gaging station #04015475**

Statistic	Units	Water Year									
		1978	**1979**	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**
Mean Oct. flow	cfs	22.6	23.1	57.8	297.2	333.2	182.0	14.0	123.2	32.8	6.3
Mean Nov. flow	cfs	7.6	128.6	12.5	18.3	119.7	36.6	11.9	30.0	27.9	15.7
Mean Dec. flow	cfs	6.1	10.2	7.1	9.5	32.0	12.3	38.8	9.3	7.2	6.9
Mean Jan. flow	cfs	6.1	6.8	6.8	7.5	9.6	7.6	8.0	7.5	6.6	6.0
Mean Feb. flow	cfs	6.1	6.3	96.3	6.9	7.8	9.3	6.5	6.9	6.7	6.0
Mean Mar. flow	cfs	82.6	17.5	86.4	25.4	41.9	23.3	148.6	61.0	220.6	60.3
Mean Apr. flow	cfs	401.5	106.2	281.3	237.2	198.4	371.0	186.3	306.3	22.8	114.3
Mean May flow	cfs	54.6	5.0	50.0	137.4	18.1	57.6	60.0	143.0	237.2	24.5
Mean Jun. flow	cfs	19.3	6.9	25.0	7.8	4.2	110.3	203.6	7.8	23.5	4.5
Mean Jul. flow	cfs	8.4	5.7	44.9	113.0	10.0	5.6	49.0	8.0	98.0	4.4
Mean Aug. flow	cfs	6.2	9.3	5.4	33.1	33.5	6.1	22.5	7.3	7.8	213.0
Mean Sep. flow	cfs	9.7	55.7	6.5	86.8	46.5	4.8	156.8	27.5	20.2	42.4
Max. 1-day flow	cfs	1397.7	552.4	570.2	917.5	982.1	1066.7	696.3	1103.0	904.9	618.4
Max. 3-day flow	cfs	1367.9	514.6	549.8	853.0	964.1	991.7	623.8	1011.5	822.2	575.1
Max. 7-day flow	cfs	1247.2	405.3	498.6	674.6	863.3	830.1	484.2	783.6	764.7	455.8
Max. 30-day flow	cfs	438.4	138.0	282.9	325.0	343.6	393.1	203.6	306.3	262.3	235.0
Max. 90-day flow	cfs	181.3	54.4	168.2	169.0	186.1	184.2	166.8	172.5	167.1	88.4
Min. 1-day flow	cfs	2.86	2.24	3.14	3.18	2.42	2.49	3.74	3.11	2.78	2.37
Min. 3-day flow	cfs	2.88	2.25	3.16	3.32	2.44	2.50	3.75	3.17	3.00	2.41
Min. 7-day flow	cfs	2.94	2.54	3.20	3.68	2.60	2.53	3.82	3.58	3.45	2.82
Min. 30-day flow	cfs	5.64	4.66	4.47	6.99	4.17	3.42	5.91	6.74	5.88	4.15
Min. 90-day flow	cfs	6.11	5.71	7.36	7.55	7.09	4.96	8.59	7.52	6.98	6.21
Number of zero flow days	#/year	0	0	0	0	0	0	0	0	0	0
Min 7-day flow divided by mean annual flow	cfs	0.056	0.081	0.057	0.045	0.036	0.037	0.051	0.058	0.057	0.067
Julian date of max. flow	day	23-Apr	3-Nov	26-Apr	19-Apr	10-Oct	15-Apr	28-Jun	3-Apr	21-May	27-Aug
Julian date of min. flow	day	22-Aug	3-Aug	30-Aug	28-Jun	30-Jun	2-Aug	15-Oct	6-Aug	10-Jul	30-Jul
Number of high pulses[1]	#/year	5	4	4	8	6	4	13	4	6	6
Number of low pulses[2]	#/year	8	12	10	3	5	3	4	8	9	6
Mean high pulse duration	days	11.0	12.8	23.5	15.4	22.2	23.0	9.9	22.5	14.8	12.7
Mean low pulse duration	days	20.8	10.9	7.6	6.7	13.4	23.7	8.3	4.5	8.6	29.5
Mean rate of flow increase[3]	cfs/day	26.4	14.7	17.1	34.8	24.6	24.5	34.0	23.1	26.8	14.3
Mean rate of flow decrease[4]	cfs/day	7.1	4.2	6.5	11.8	9.4	8.8	13.4	9.2	9.4	6.2
Number of flow reversals	#/year	70	75	63	48	48	49	58	66	54	58

[1] Number of times per year the mean daily flow increases above the 75th percentile of all mean daily flows.

[2] Number of times per year the mean daily flow decreases below the 25th percentile of all mean daily flows.

[3] Mean of all positive differences between consecutive daily values

[4] Mean of all negative differences between consecutive daily values

Figure 1: Study Area of Hydrologic/Hydraulic Model

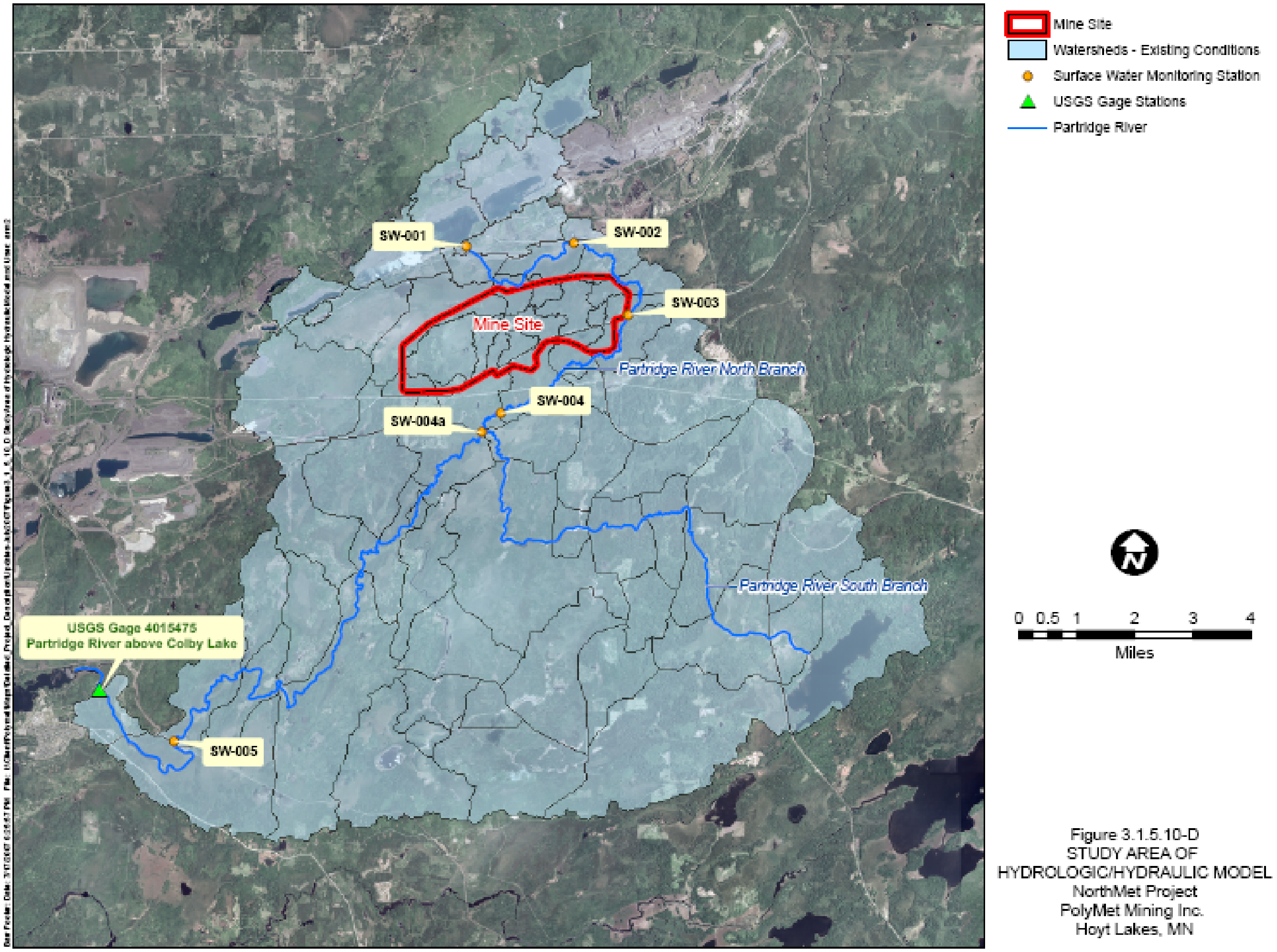


Figure 3.1.5.10-D
STUDY AREA OF
HYDROLOGIC/HYDRAULIC MODEL
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

Appendix B

Results of water balance calculations for Colby Lake-Whitewater Reservoir hydrologic system

Results of Water Balance Calculations for Average Flow Conditions

Table 1: 4-year model results comparing water level impacts for various make-up water demands assuming average flow in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

	Colby Lake			**Whitewater Reservoir**		
	Zero-demand	**Projected Future Conditions**		**Zero-demand**	**Projected Future Conditions**	
Make-up water demand (gpm)	0	3500	5000	0	3500	5000
Average Elevation[1] (feet)	1439.45	1439.42	1439.44	1439.33	1438.94	1438.33
Maximum Elevation[1] (feet)	1442.75	1442.51	1442.45	1440.26	1440.25	1440.23
Minimum Elevation[1] (feet)	1438.85	1438.88	1438.84	1437.41	1435.98	1433.34
Maximum Fluctuation[1] (feet)	3.90	3.63	3.60	2.86	4.22	6.84
Days Pumping Into Colby Lake[2]	NA	NA	NA	0	787	815
Days Flowing Into Whitewater Reservoir[2]	156	161	174	NA	NA	NA
Time[2] Below 1,439 feet	10.5	9.5	4.0	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

Figure 1a: 4-year model results for Colby Lake water levels assuming average flow in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

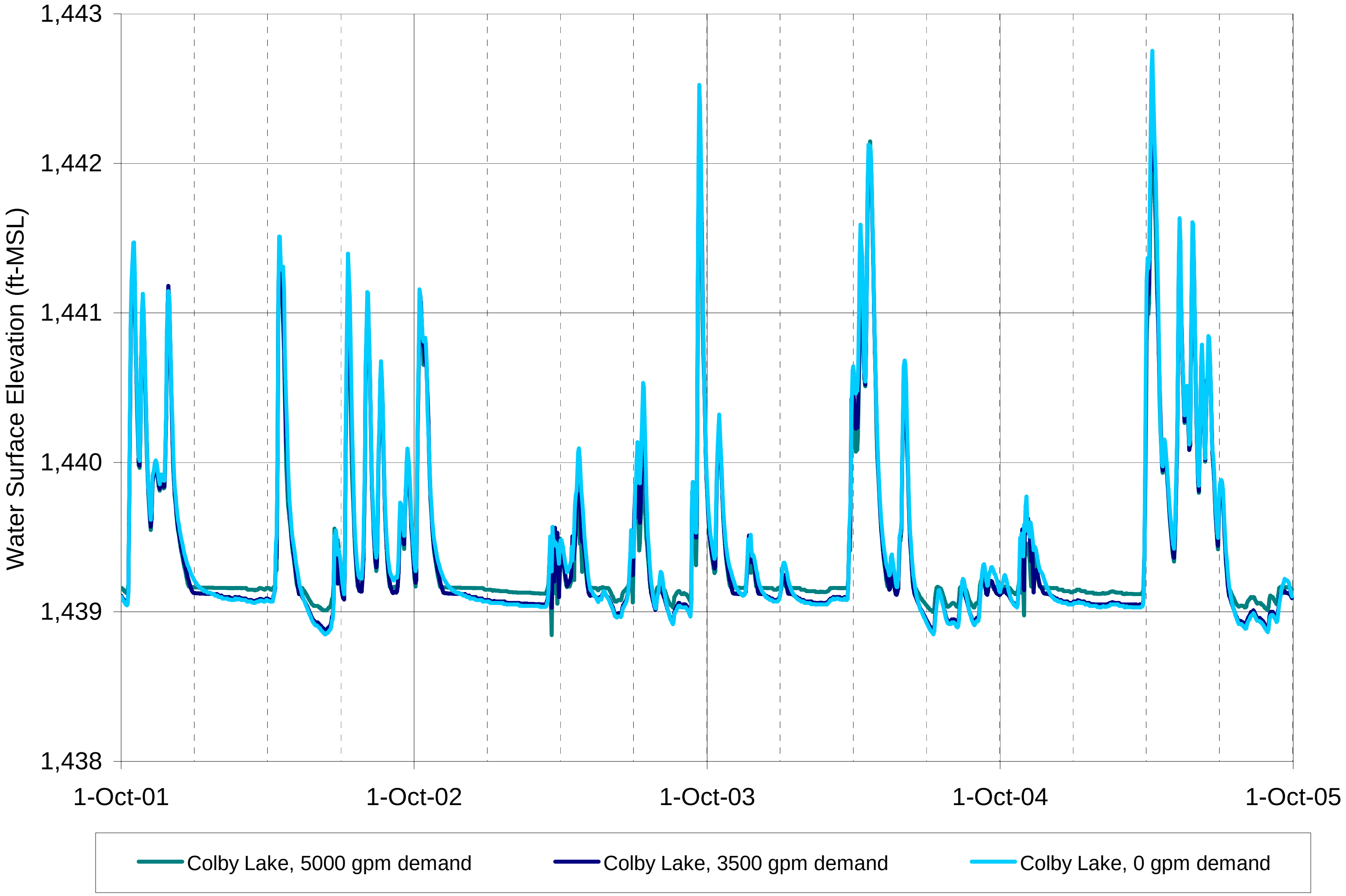


Figure 1b: 4-year model results for Whitewater Reservoir water levels assuming average flow in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

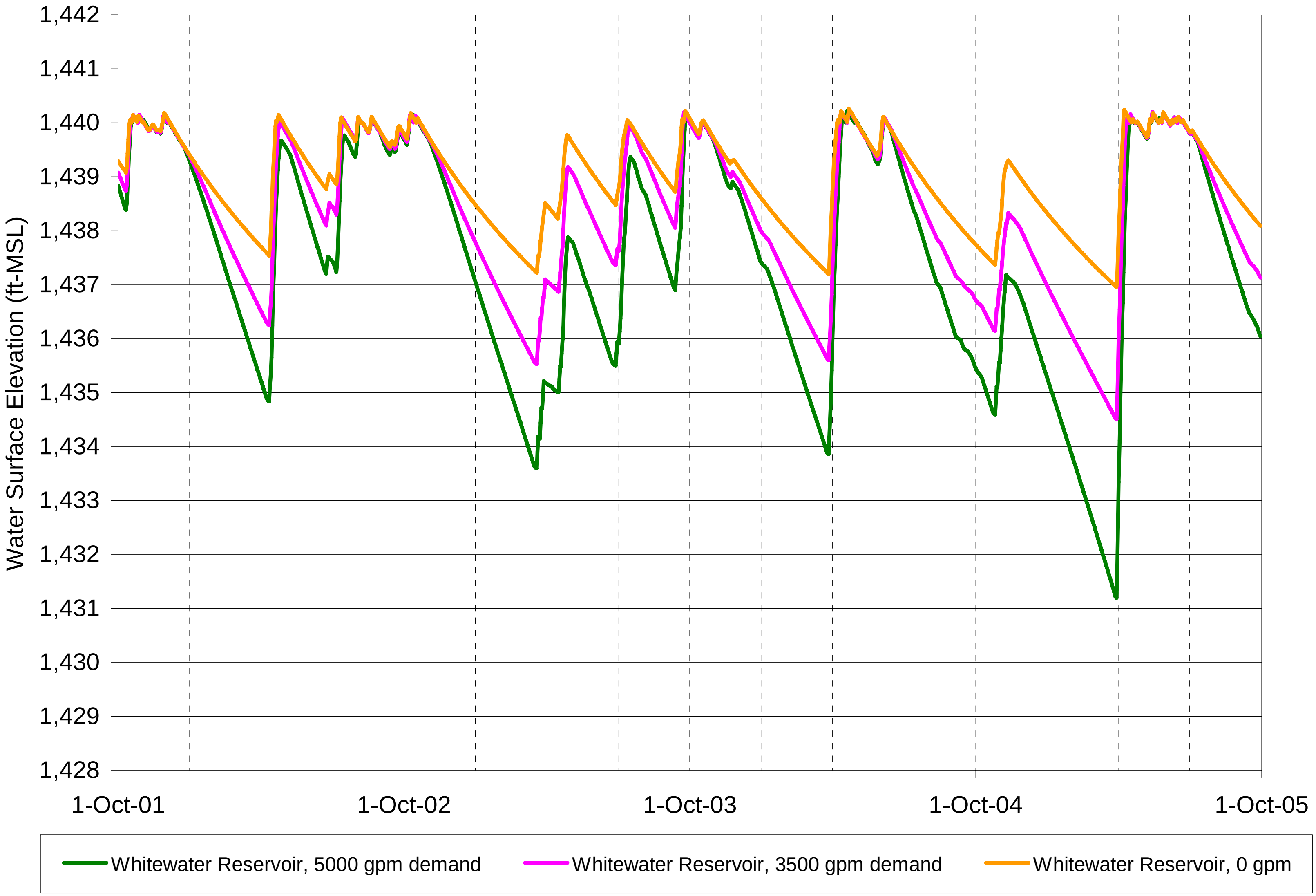


Figure 1c: Elevation – duration curves for Colby Lake water levels assuming average flow in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

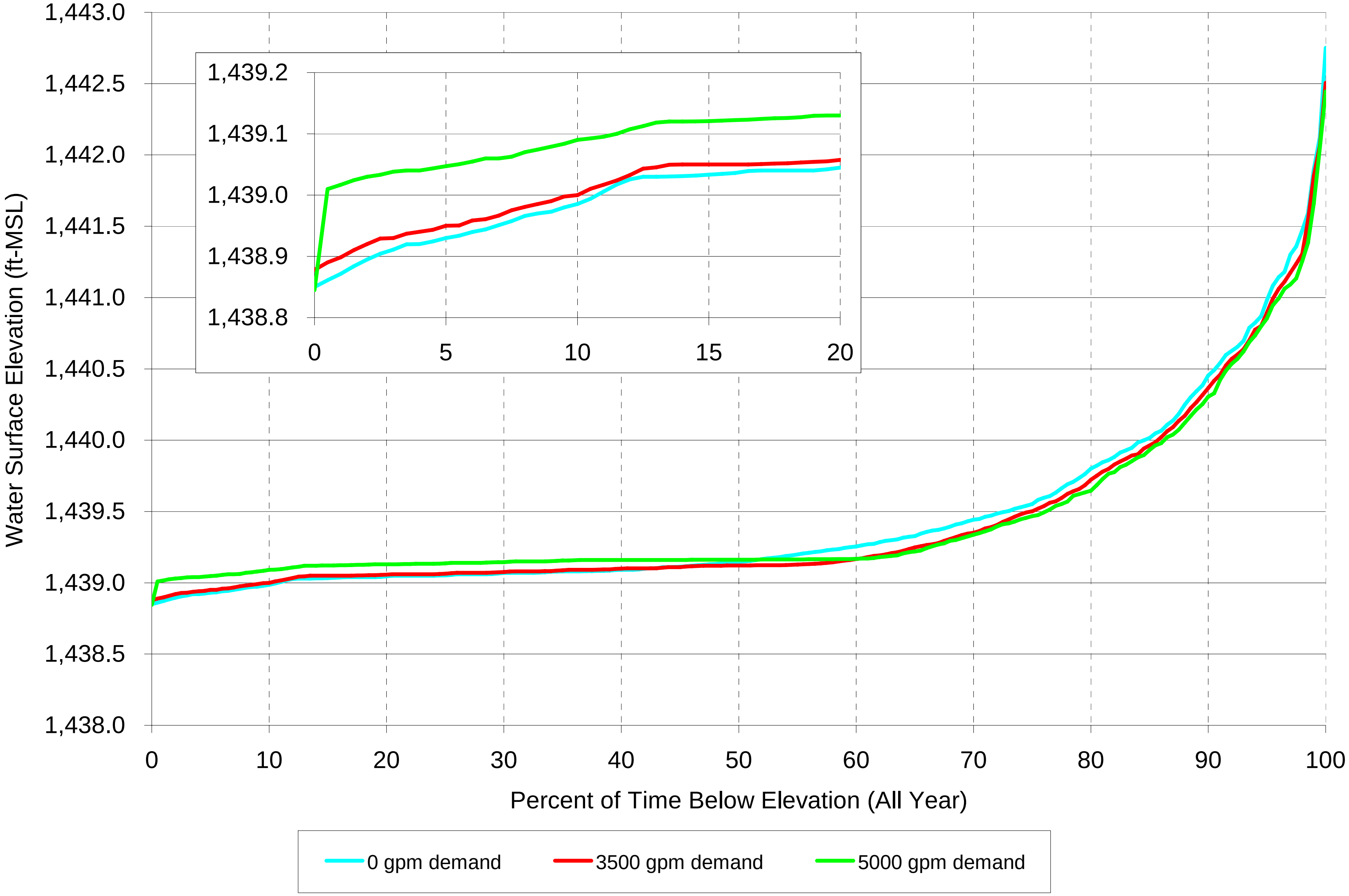
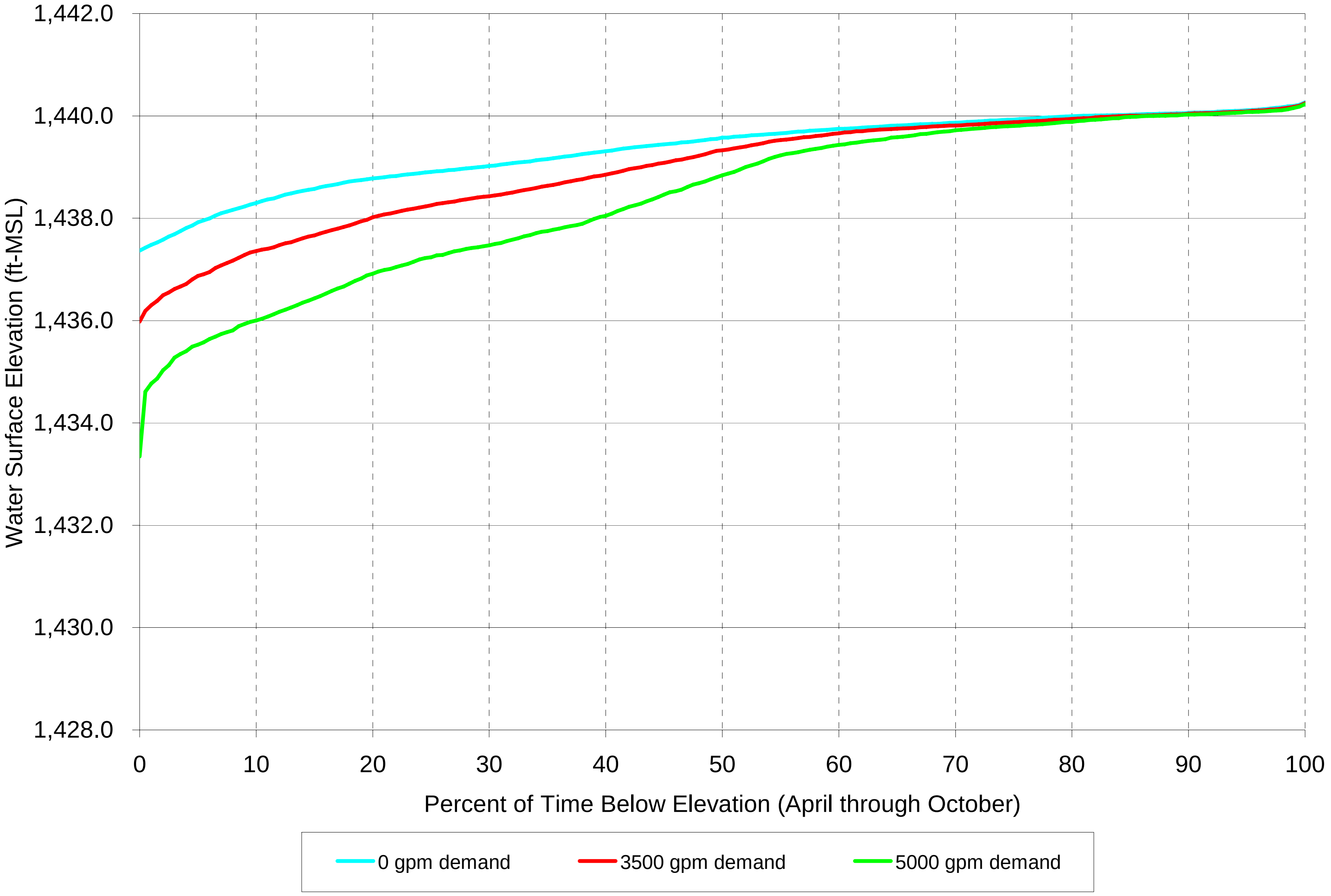


Figure 1d: Elevation – duration curves for Whitewater Reservoir water levels assuming average flow in the Partridge River, with Colby Lake water level above 1,439.50 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates



Results of Water Balance Calculations for 2001-2005 Flow Conditions

Table 2: 4-year model results comparing water level impacts for various make-up water demands assuming 2001-2005 flow conditions (approximately 87 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

	Colby Lake			**Whitewater Reservoir**		
	Zero-demand	**Projected Future Conditions**		**Zero-demand**	**Projected Future Conditions**	
Make-up water demand (gpm)	0	3500	5000	0	3500	5000
Average Elevation[1] (feet)	1439.39	1439.36	1439.39	1439.29	1438.78	1438.05
Maximum Elevation[1] (feet)	1442.48	1442.21	1442.13	1440.23	1440.24	1440.21
Minimum Elevation[1] (feet)	1438.83	1438.85	1438.78	1437.41	1435.63	1432.64
Maximum Fluctuation[1] (feet)	3.65	3.36	3.36	2.82	4.52	7.48
Days Pumping Into Colby Lake[2]	NA	NA	NA	0	854	883
Days Flowing Into Whitewater Reservoir[2]	158	168	179	NA	NA	NA
Time[2] Below 1,439 feet	14.0	11.0	1.0	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

Figure 2a: 4-year model results for Colby Lake water levels assuming 2001-2005 flow conditions (approximately 87 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

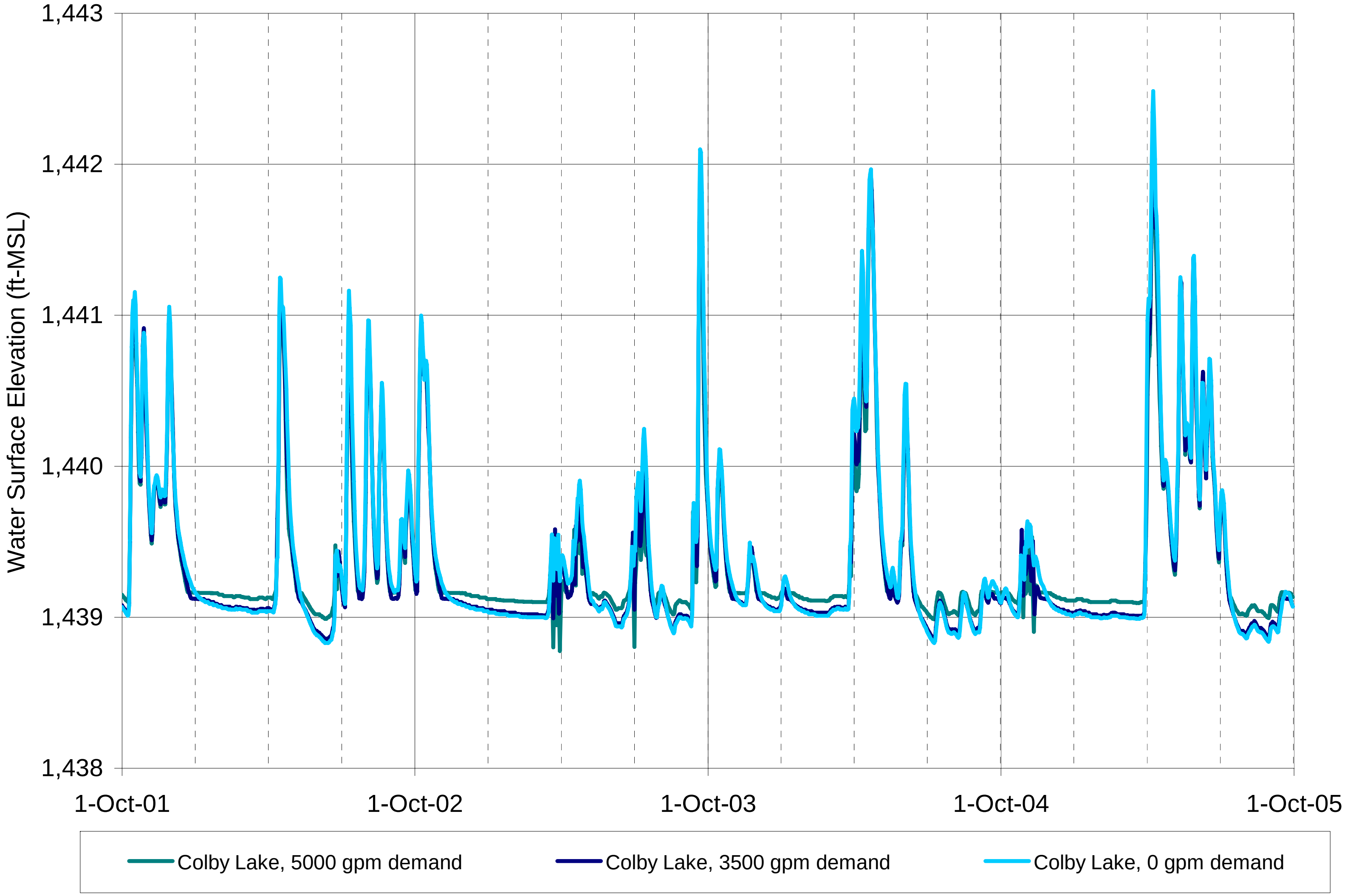


Figure 2b: 4-year model results for Whitewater Reservoir water levels assuming 2001-2005 flow conditions (approximately 87 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

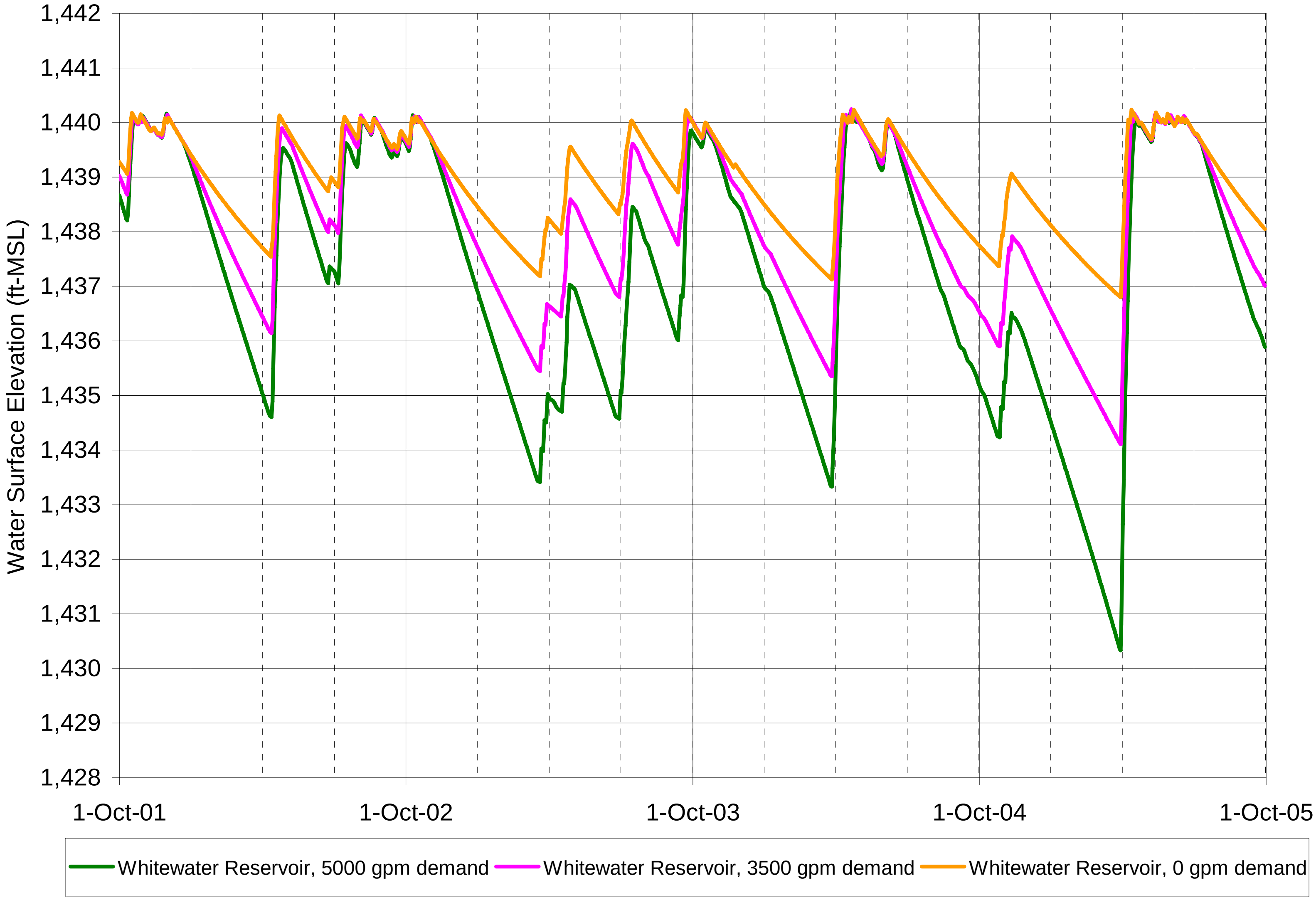


Figure 2c: Elevation – duration curves for Colby Lake water levels assuming 2001-2005 flow conditions (approximately 87 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

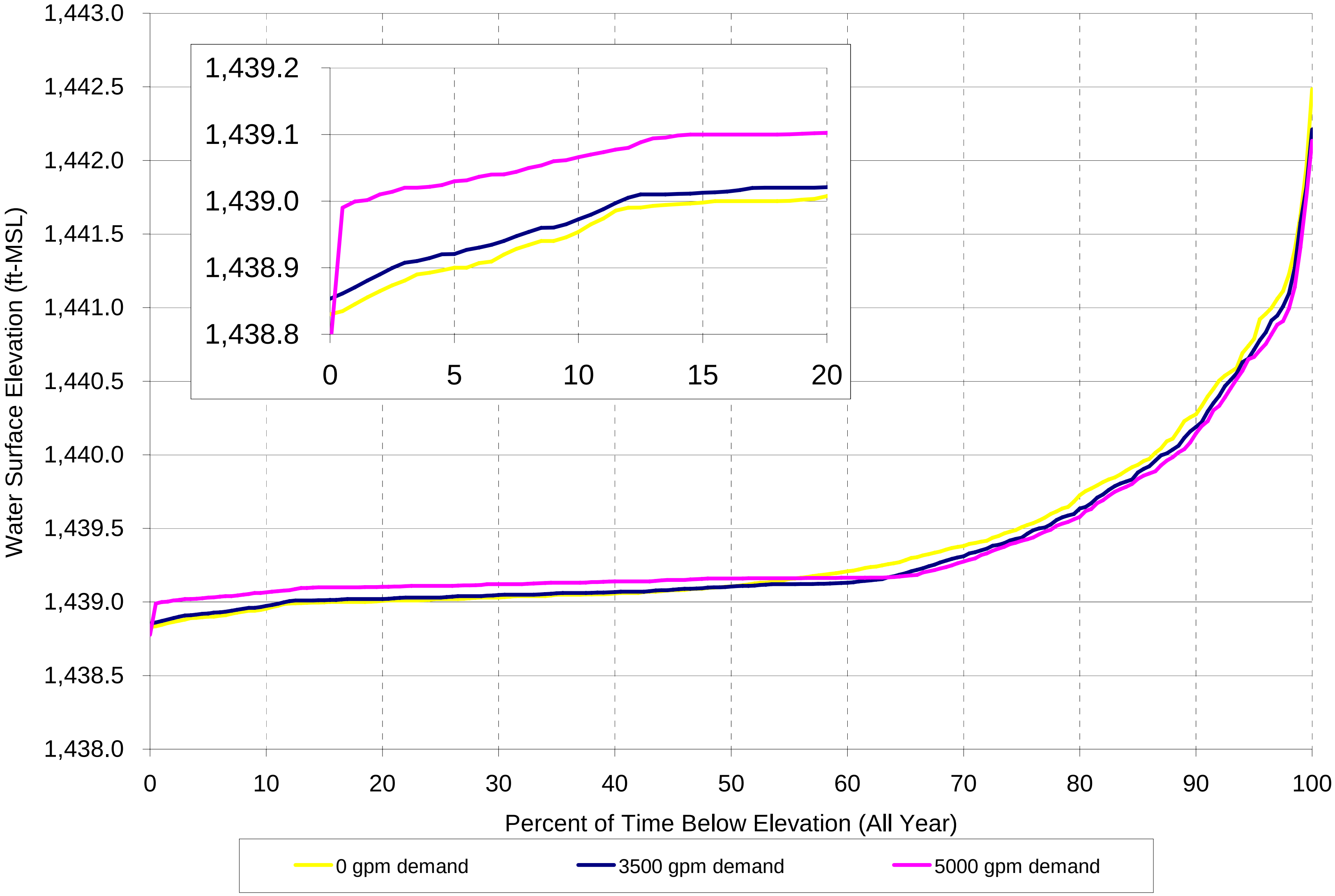
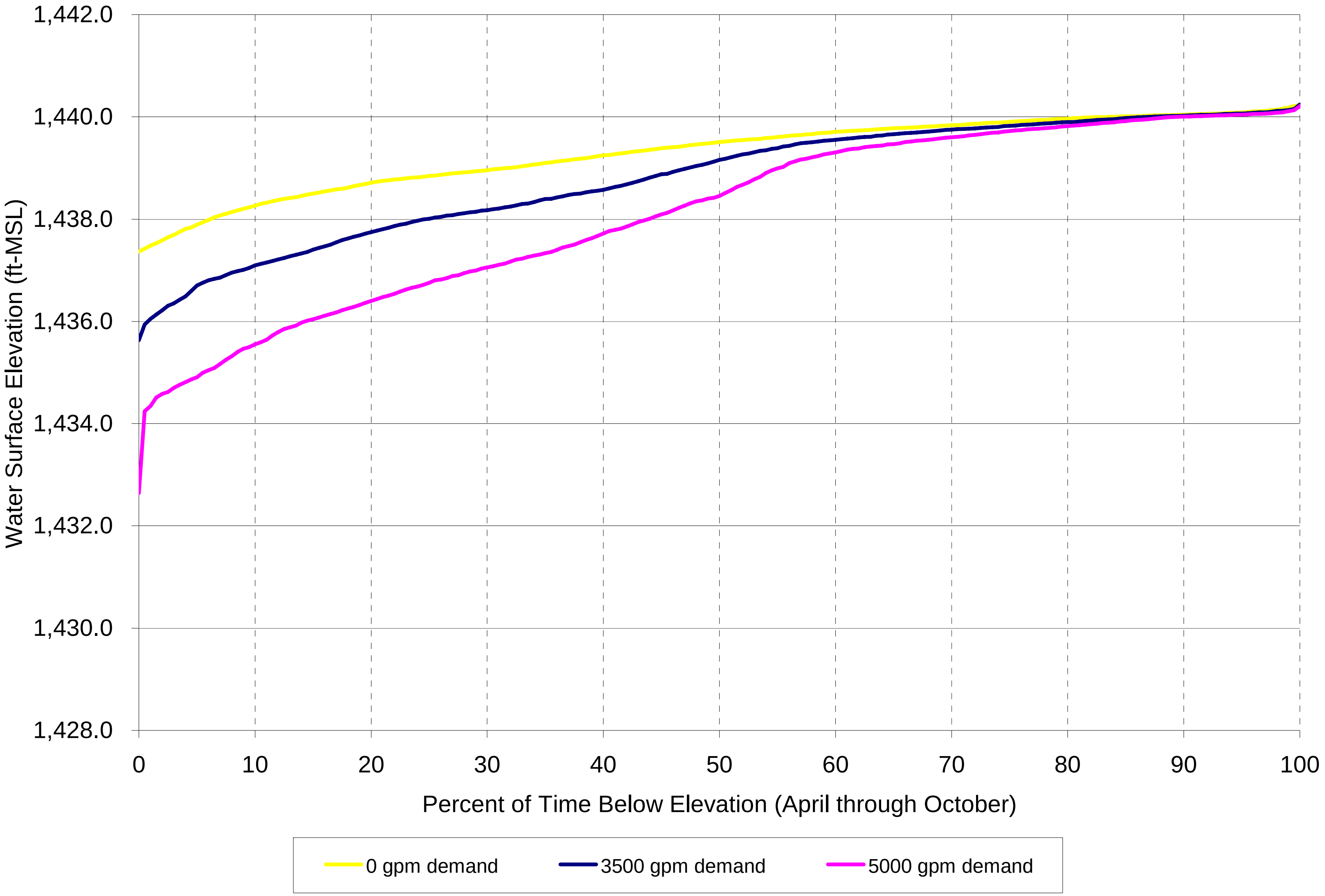


Figure 2d: Elevation – duration curves for Whitewater Reservoir water levels assuming 2001-2005 flow conditions (approximately 87 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates



Results of Water Balance Calculations for 10-Year Low-Flow Conditions

Table 3: 4-year model results comparing water level impacts for various make-up water demands assuming 10-year low flow conditions (approximately 79 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

	Colby Lake				**Whitewater Reservoir**			
	Zero-demand	**Projected Future Conditions**			**Zero-demand**	**Projected Future Conditions**		
Make-up water demand (gpm)	0	3500	5000	CHD[3]	0	3500	5000	CHD[3]
Average Elevation[1] (feet)	1439.35	1439.33	1439.36	1439.34	1439.25	1438.66	1437.82	1437.85
Maximum Elevation[1] (feet)	1442.18	1442.04	1441.94	1441.95	1440.23	1440.22	1440.23	1440.16
Minimum Elevation[1] (feet)	1438.81	1438.84	1438.69	1438.66	1437.38	1435.35	1431.19	1434.25
Maximum Fluctuation[1] (feet)	3.37	3.20	3.26	3.28	2.85	4.80	8.96	8.86
Days Pumping Into Colby Lake[2]	NA	NA	NA	NA	0	889	917	919
Days Flowing Into Whitewater Reservoir[2]	158	163	181	183	NA	NA	NA	NA
Time[2] Below 1,439 feet	24.5	16.0	1.5	11.5	NA	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

[3] CHD = Combined high demand (8,000 gallons per minute during three months of the year and 4,400 gallons per minute during the other nine months of year).

Figure 3a: 4-year model results for Colby Lake water levels assuming 10-year low flow conditions (approximately 79 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

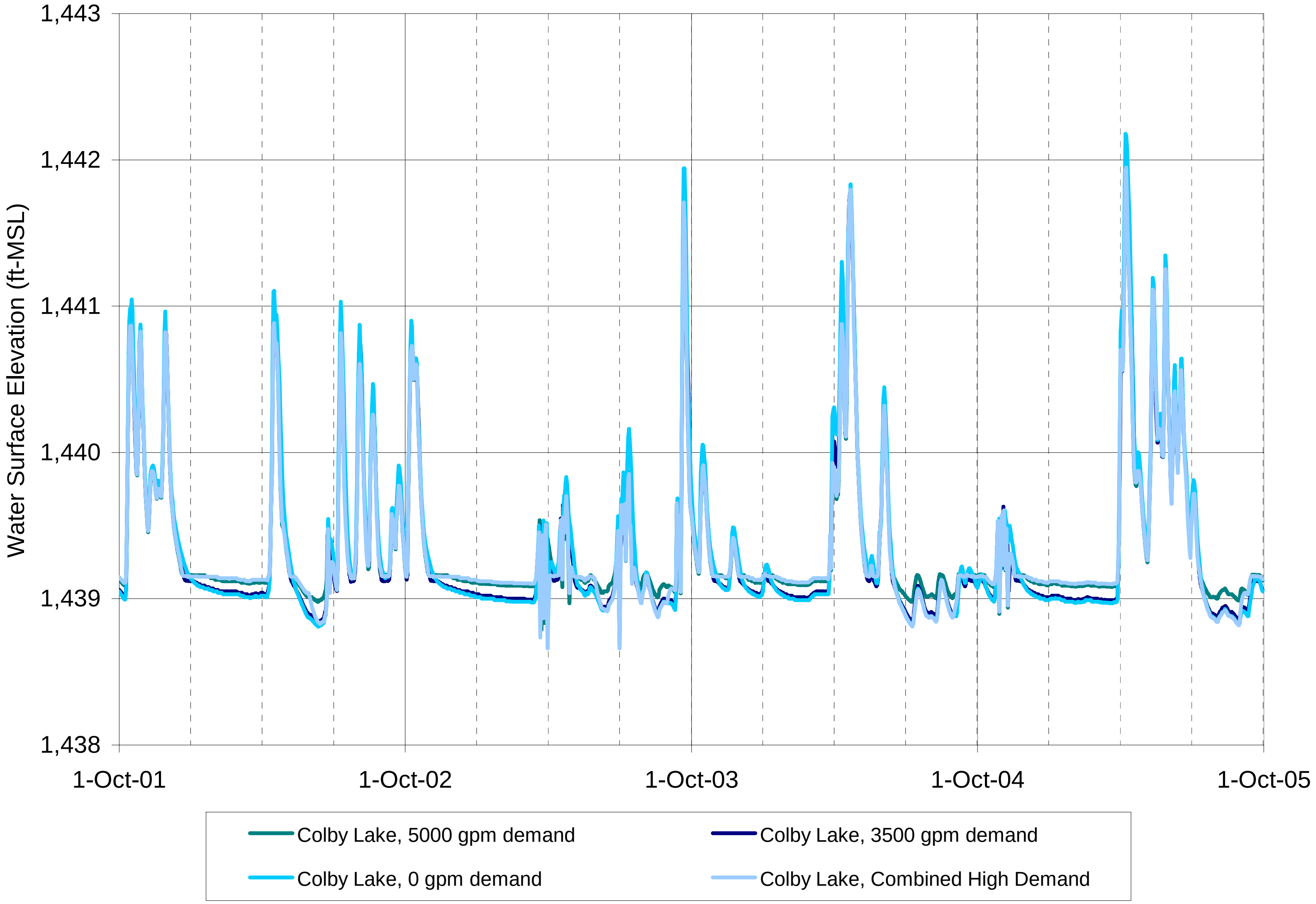


Figure 3b: 4-year model results for Whitewater Reservoir water assuming 10-year low flow conditions (approximately 79 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

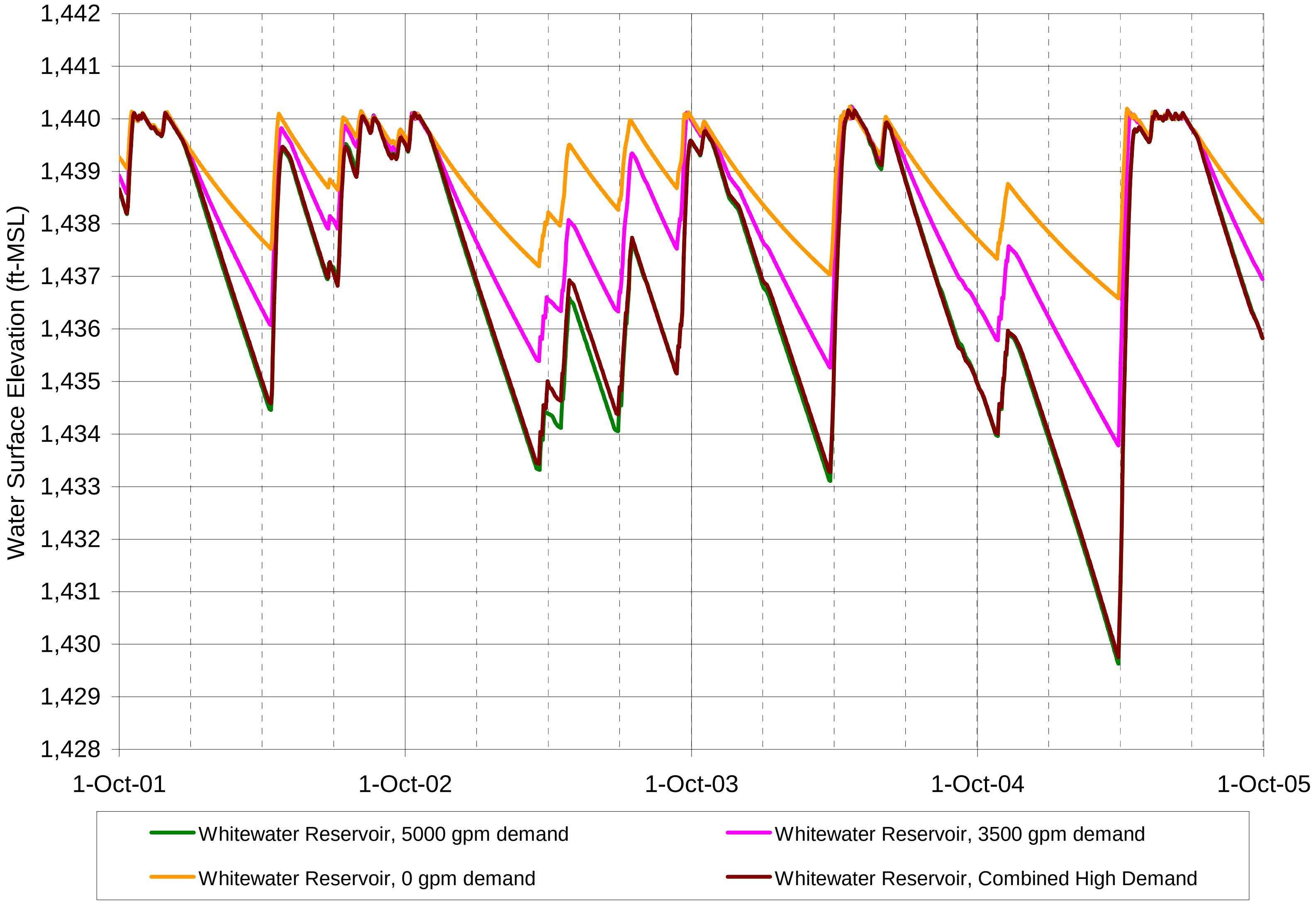


Figure 3c: Elevation – duration curves for Colby Lake water levels assuming 10-year low flow conditions (approximately 79 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

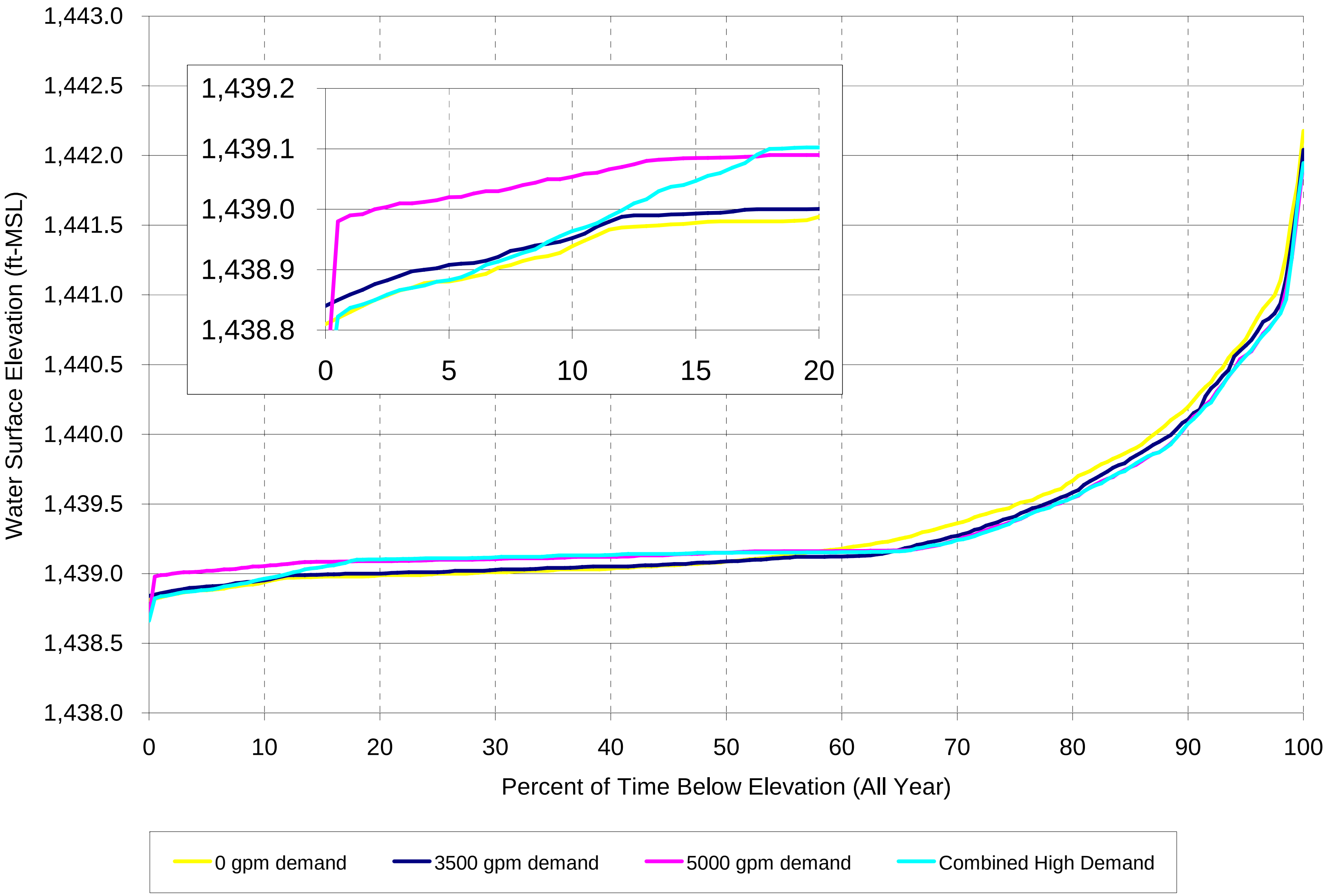
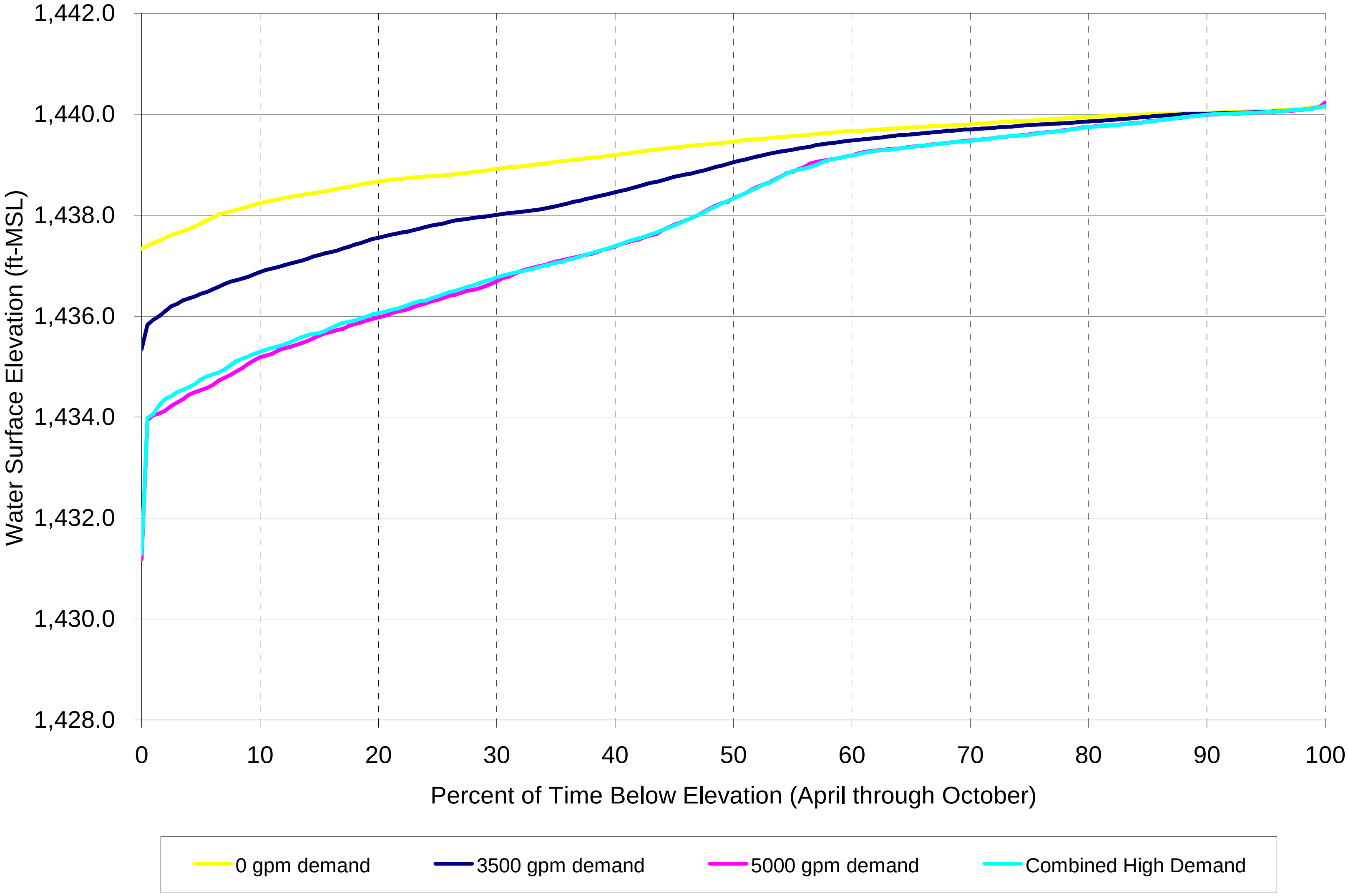


Figure 3d: Elevation – duration curves for Whitewater Reservoir water levels assuming 10-year low flow conditions (approximately 79 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates



Results of Water Balance Calculations for 25-Year Low-Flow Conditions

Table 4: 4-year model results comparing water level impacts for various make-up water demands assuming 25-year low flow conditions (approximately 73 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

	Colby Lake				**Whitewater Reservoir**			
	Zero-demand	**Projected Future Conditions**			**Zero-demand**	**Projected Future Conditions**		
Make-up water demand (gpm)	0	3500	5000	CHD[3]	0	3500	5000	CHD[3]
Average Elevation[1] (feet)	1439.32	1439.29	1439.33	1439.31	1439.21	1438.54	1437.63	1437.61
Maximum Elevation[1] (feet)	1442.01	1441.86	1441.77	1441.77	1440.17	1440.18	1440.21	1440.18
Minimum Elevation[1] (feet)	1438.80	1438.83	1438.70	1438.72	1437.38	1434.41	1430.86	1430.98
Maximum Fluctuation[1] (feet)	3.21	3.03	3.07	3.05	2.80	5.76	9.30	9.18
Days Pumping Into Colby Lake[2]	NA	NA	NA	NA	0	920	949	943
Days Flowing Into Whitewater Reservoir[2]	155	162	182	184	NA	NA	NA	NA
Time[2] Below 1,439 feet	32.5	26.0	2.5	12.5	NA	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

[3] CHD = Combined high demand (8,000 gallons per minute during three months of the year and 4,400 gallons per minute during the other nine months of year).

Figure 4a: 4-year model results for Colby Lake water levels assuming 25-year low flow conditions (approximately 73 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

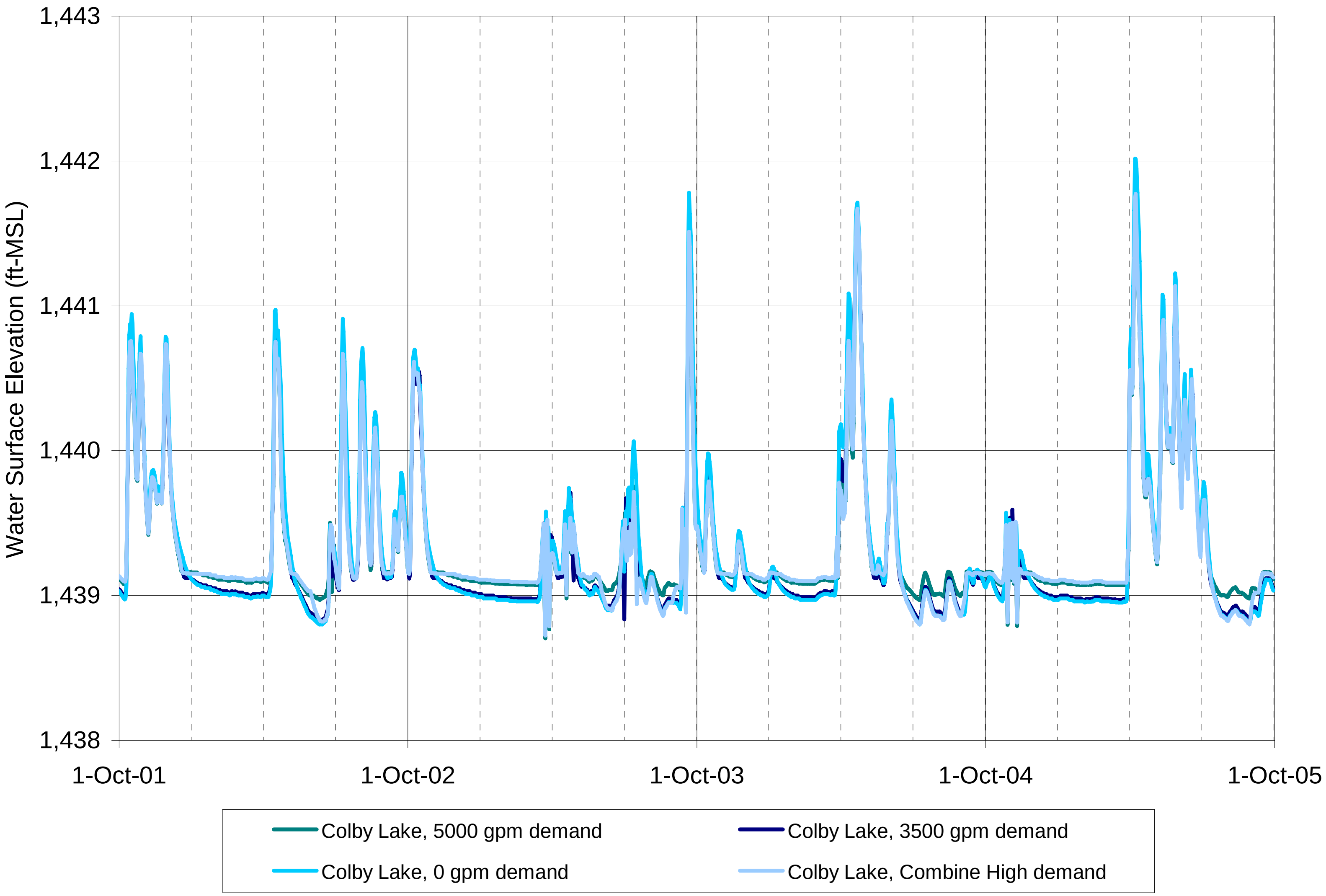


Figure 4b: 4-year model results for Whitewater Reservoir water assuming 25-year low flow conditions (approximately 73 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

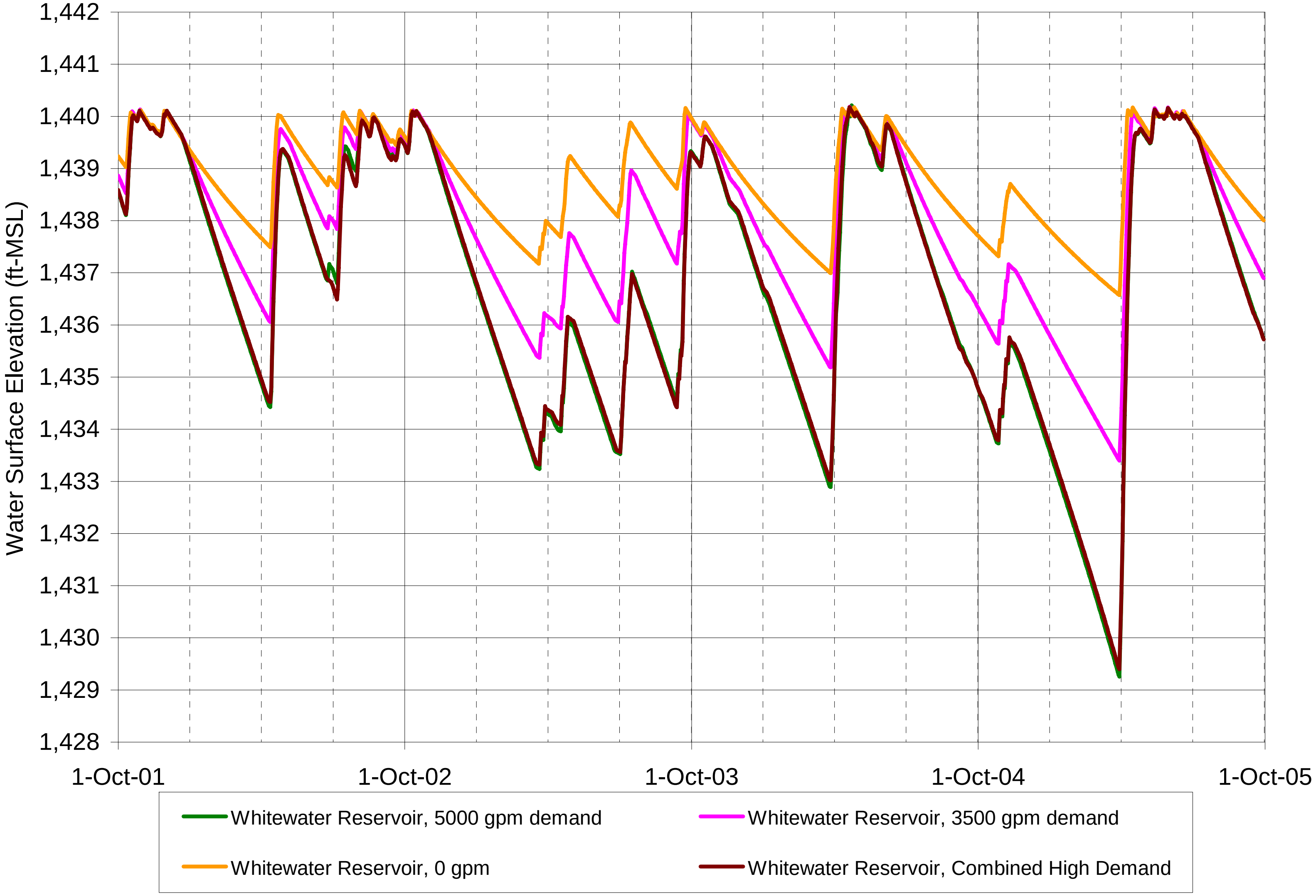


Figure 4c: Elevation – duration curves for Colby Lake water levels assuming 25-year low flow conditions (approximately 73 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

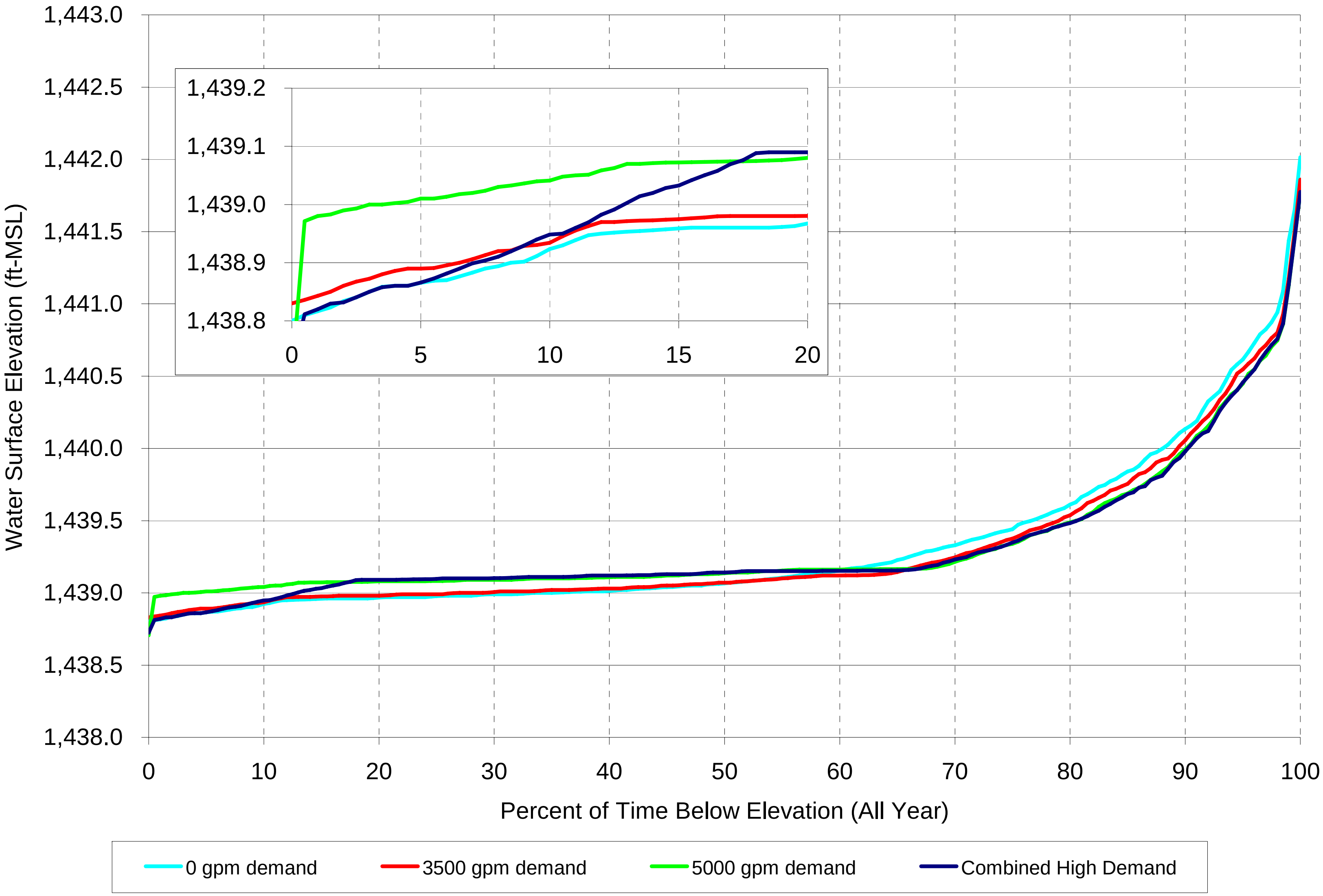
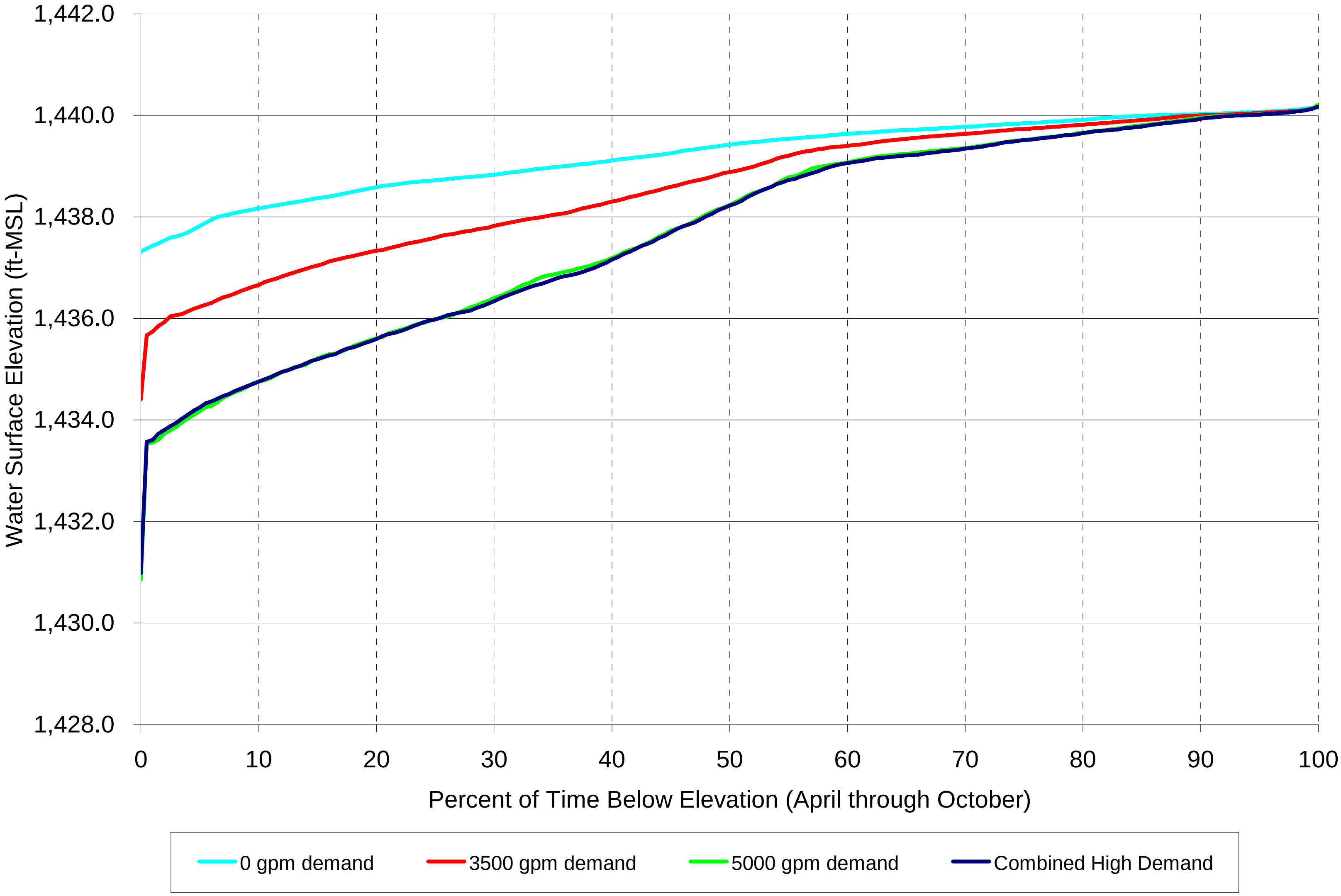


Figure 4d: Elevation – duration curves for Whitewater Reservoir water levels assuming 25-year low flow conditions (approximately 73 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates



Results of Water Balance Calculations for 50-Year Low-Flow Conditions

Table 5: 4-year model results comparing water level impacts for various make-up water demands assuming 50-year low flow conditions (approximately 69 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

	Colby Lake				**Whitewater Reservoir**			
	Zero-demand	**Projected Future Conditions**			**Zero-demand**	**Projected Future Conditions**		
Make-up water demand (gpm)	0	3500	5000	CHD[3]	0	3500	5000	CHD[3]
Average Elevation[1] (feet)	1439.30	1439.27	1439.31	1439.29	1439.30	1438.46	1437.50	1437.49
Maximum Elevation[1] (feet)	1441.91	1441.75	1441.65	1441.65	1440.40	1440.21	1440.18	1440.17
Minimum Elevation[1] (feet)	1438.79	1438.82	1438.65	1438.67	1437.38	1434.31	1430.29	1430.41
Maximum Fluctuation[1] (feet)	3.12	2.93	3.00	2.98	2.94	5.86	9.87	9.74
Days Pumping Into Colby Lake[2]	NA	NA	NA	NA	0	943	964	962
Days Flowing Into Whitewater Reservoir[2]	154	161	179	181	NA	NA	NA	NA
Time[2] Below 1,439 feet	39.0	31.5	4.0	13.5	NA	NA	NA	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

[3] CHD = Combined high demand (8,000 gallons per minute during three months of the year and 4,400 gallons per minute during the other nine months of year).

Figure 5a: 4-year model results for Colby Lake water levels assuming 50-year low flow conditions (approximately 69 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

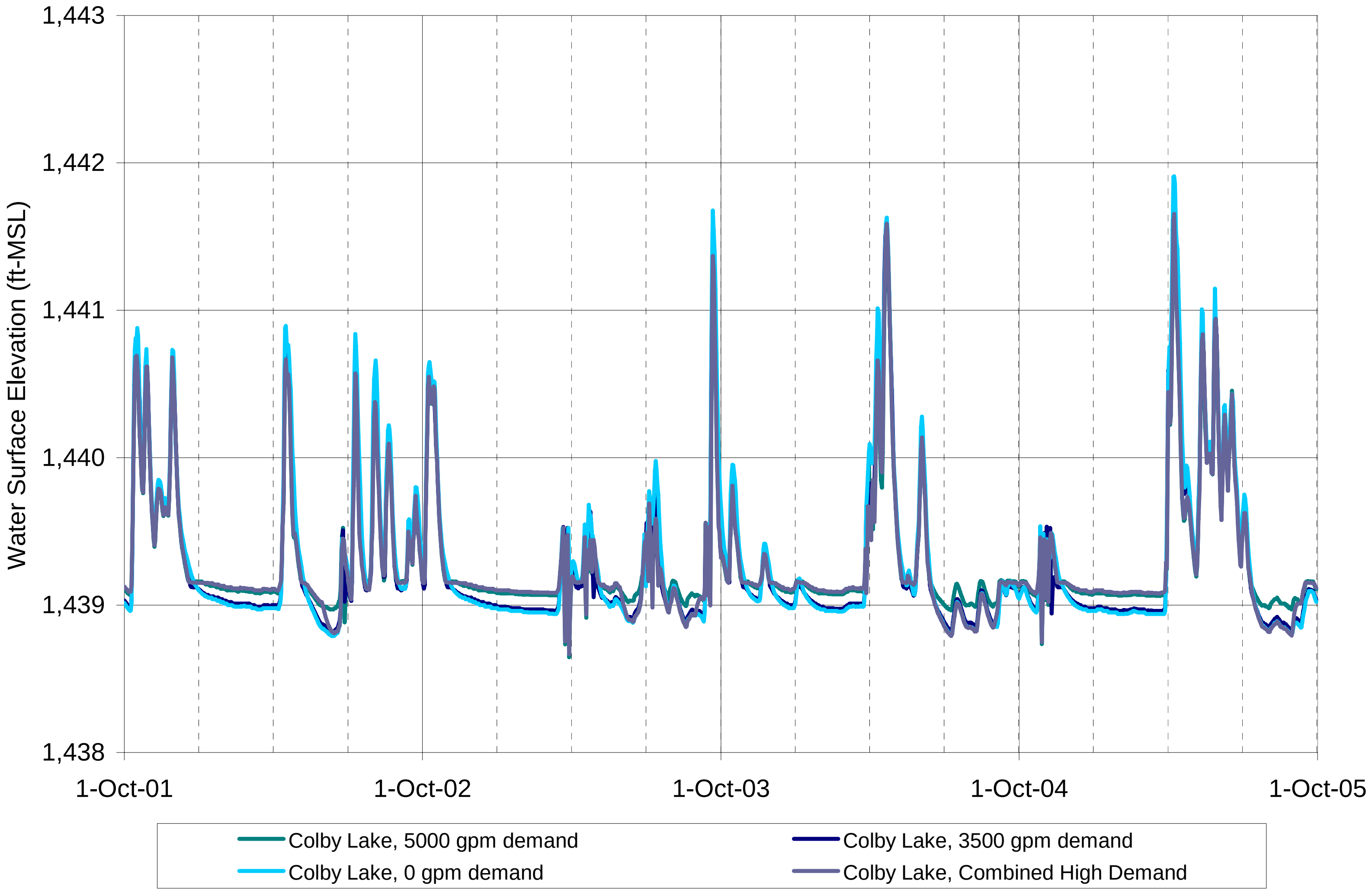


Figure 5b: 4-year model results for Whitewater Reservoir water assuming 50-year low flow conditions (approximately 69 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

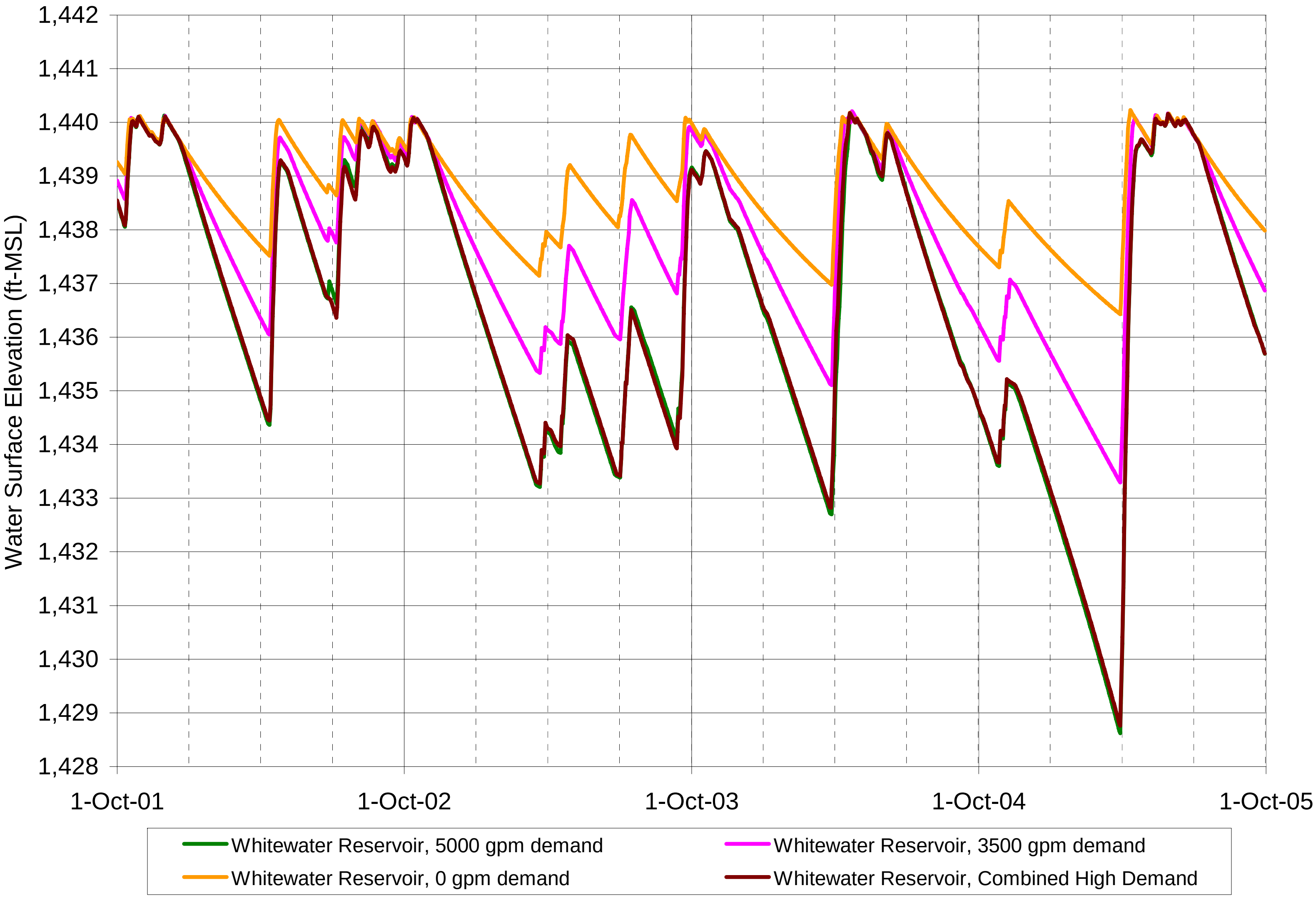


Figure 5c: Elevation – duration curves for Colby Lake water levels assuming 50-year low flow conditions (approximately 69 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates

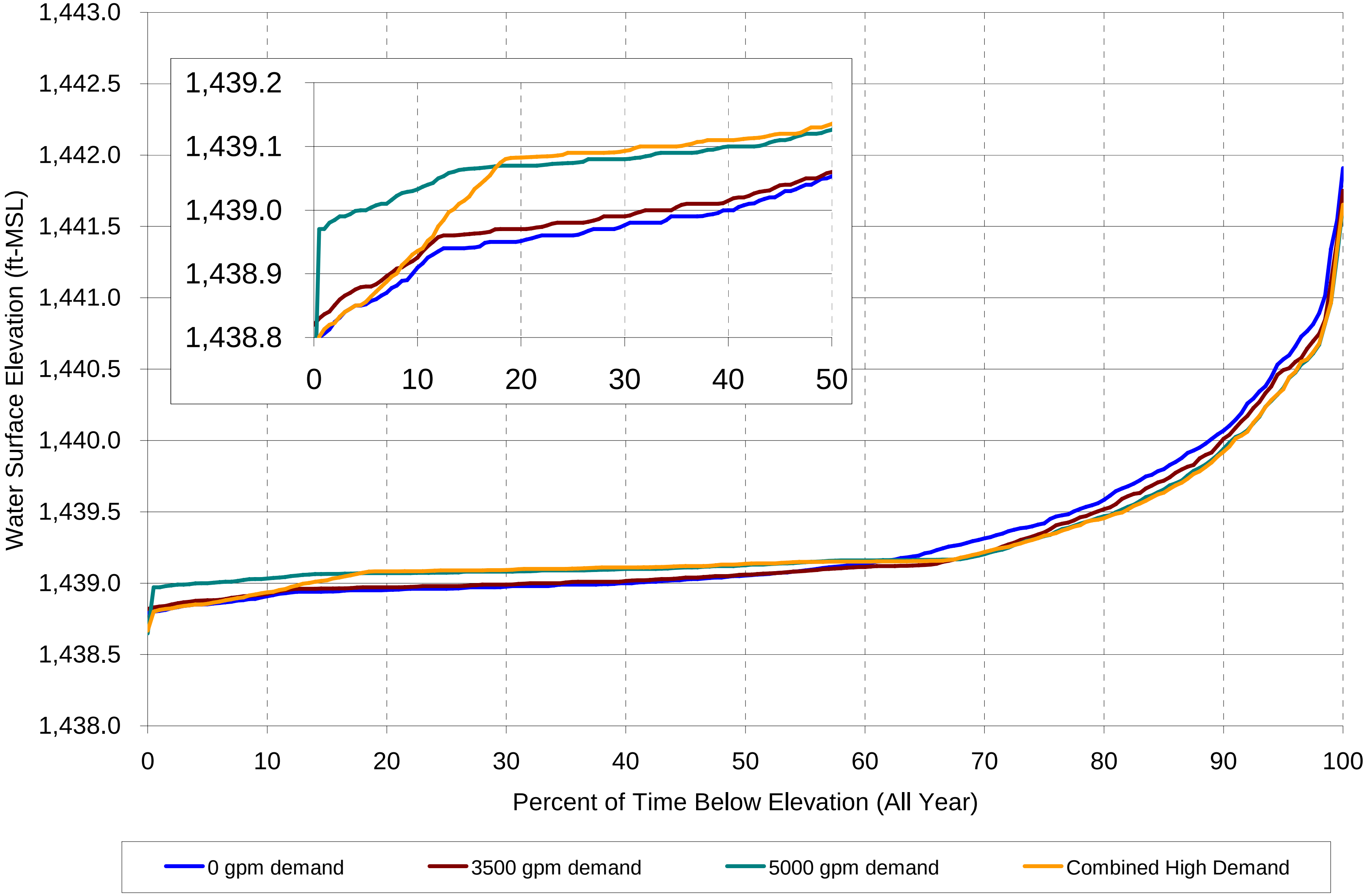
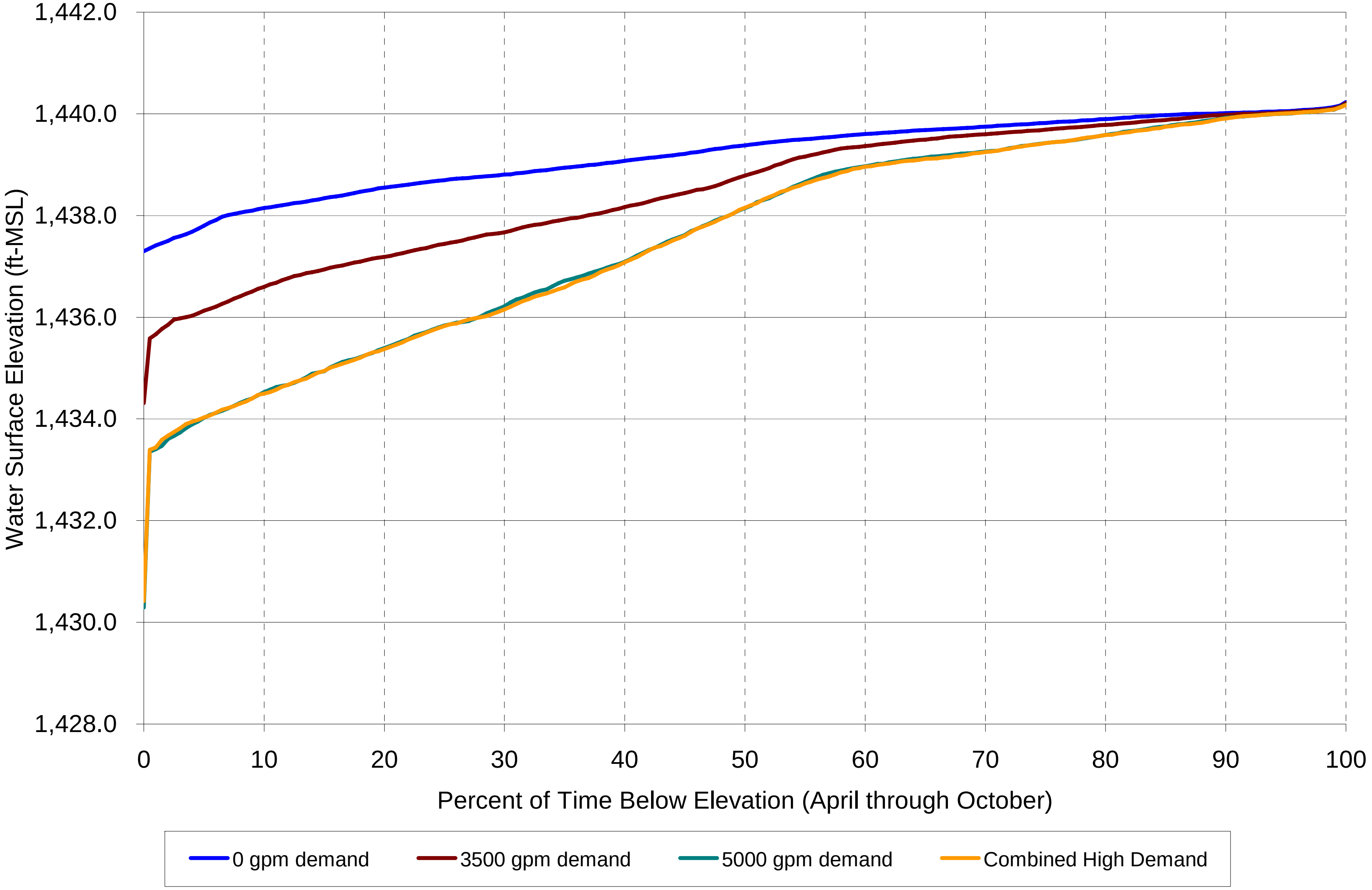


Figure 5d: Elevation – duration curves for Whitewater Reservoir water levels assuming 50-year low flow conditions (approximately 69 percent of average flow conditions) in the Partridge River, with Colby Lake water level above 1,439.5 feet above mean sea level for water to be diverted to Whitewater Reservoir via two open sluicegates



Recorded Water Level Data in Colby Lake and Whitewater Reservoir

Table 6: 4-year observed data for Colby Lake and Whitewater Reservoir for the time period October 1, 2001 to September 30, 2005

	Colby Lake	**Whitewater Reservoir**
	Observed Data	**Observed Data**
Average Elevation[1] (feet)	1439.48	1437.93
Maximum Elevation[1] (feet)	1441.58	1440.41
Minimum Elevation[1] (feet)	1438.78	1436.06
Maximum Fluctuation[1] (feet)	2.80	4.35
Time[2] Below 1,439 feet	10.0	NA

[1] Values for Colby Lake are those occurring over the entire four-year period of analysis. Values for Whitewater Reservoir are those occurring between April and October.

[2] Values computed for entire four-year period of analysis for Colby Lake and Whitewater Reservoir.

Figure 6a: Observed water surface elevation data for Colby Lake

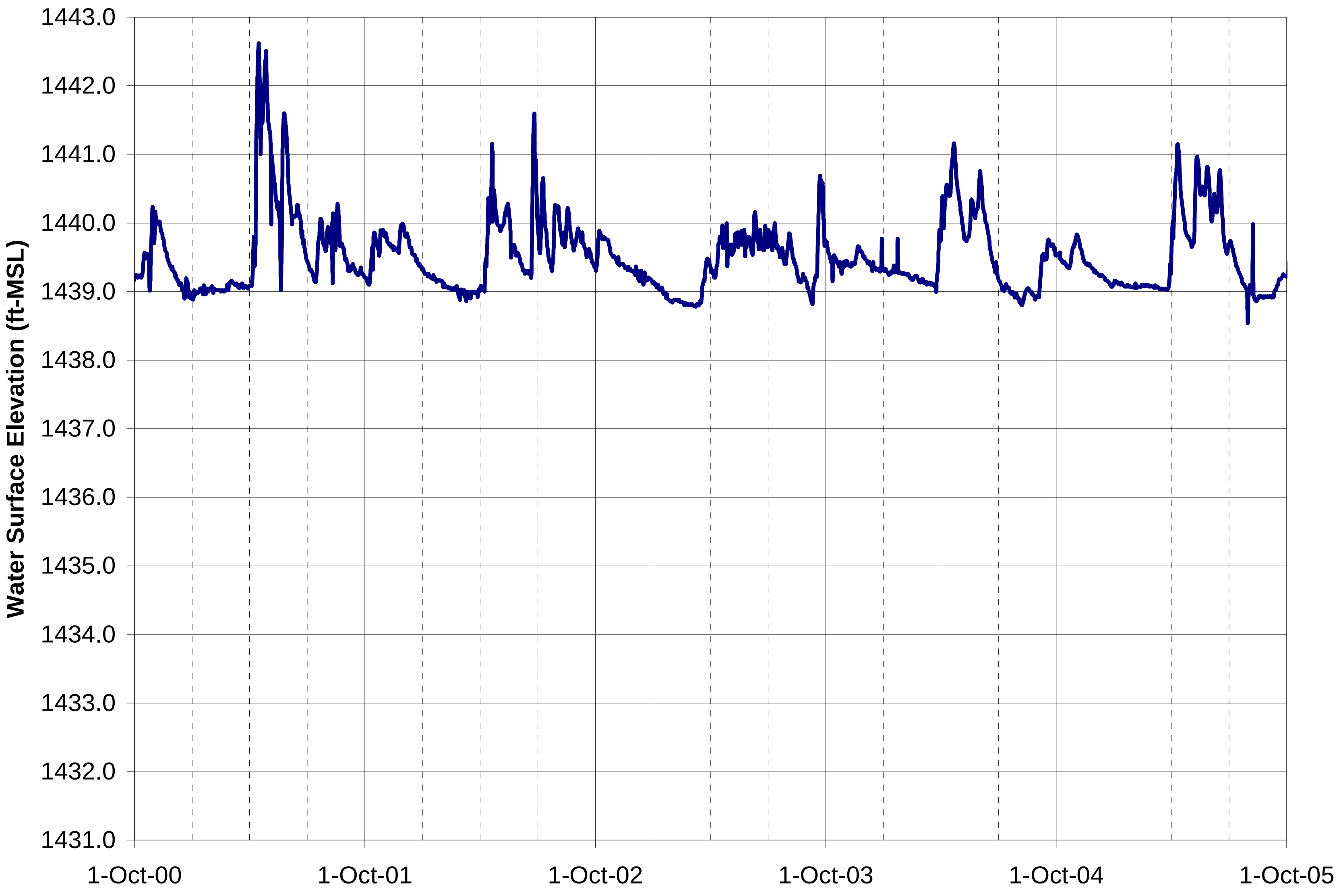


Figure 6b: Observed water surface elevation data for Whitewater Reservoir

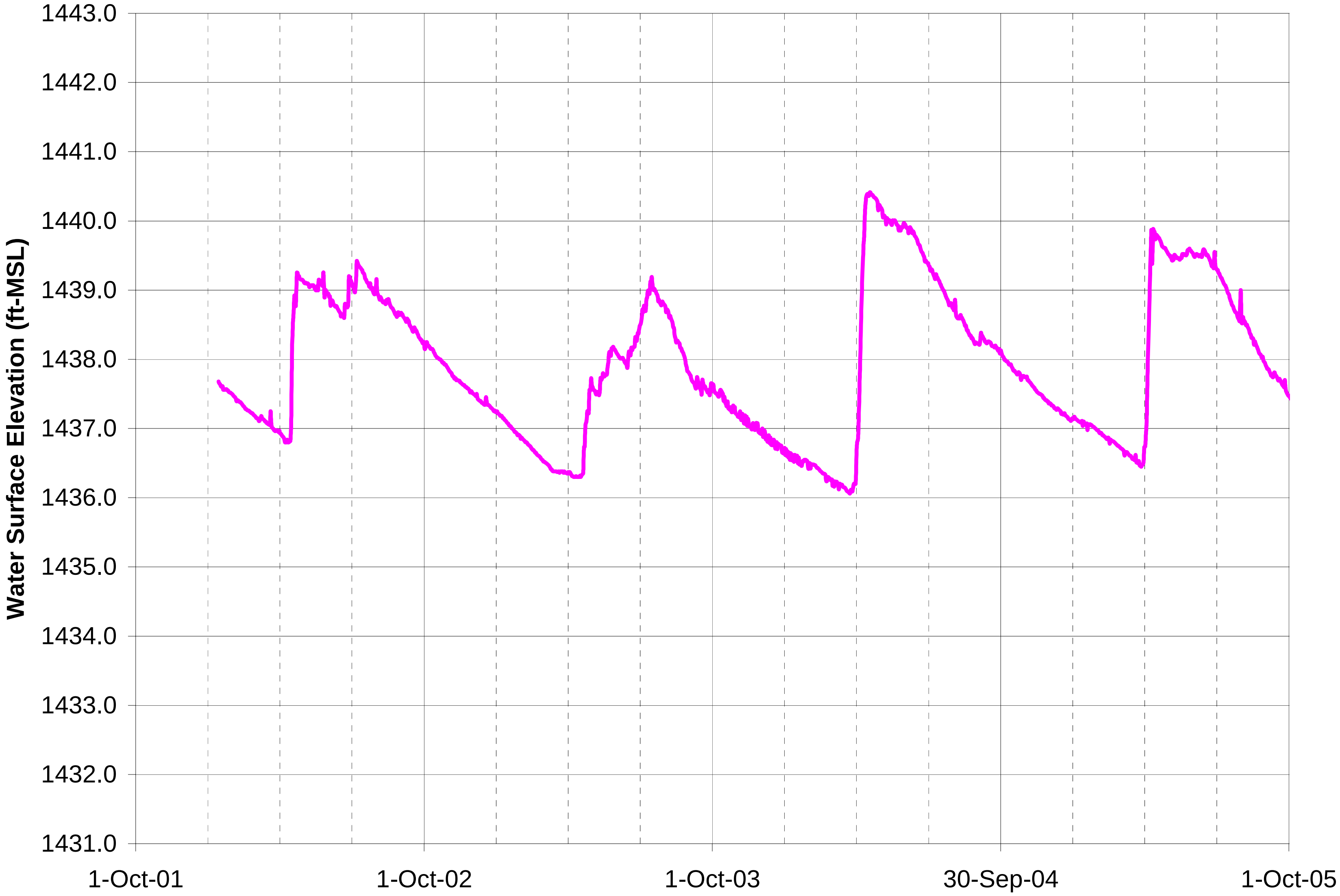


Figure 6c: Elevation – duration curves for observed Colby Lake water levels

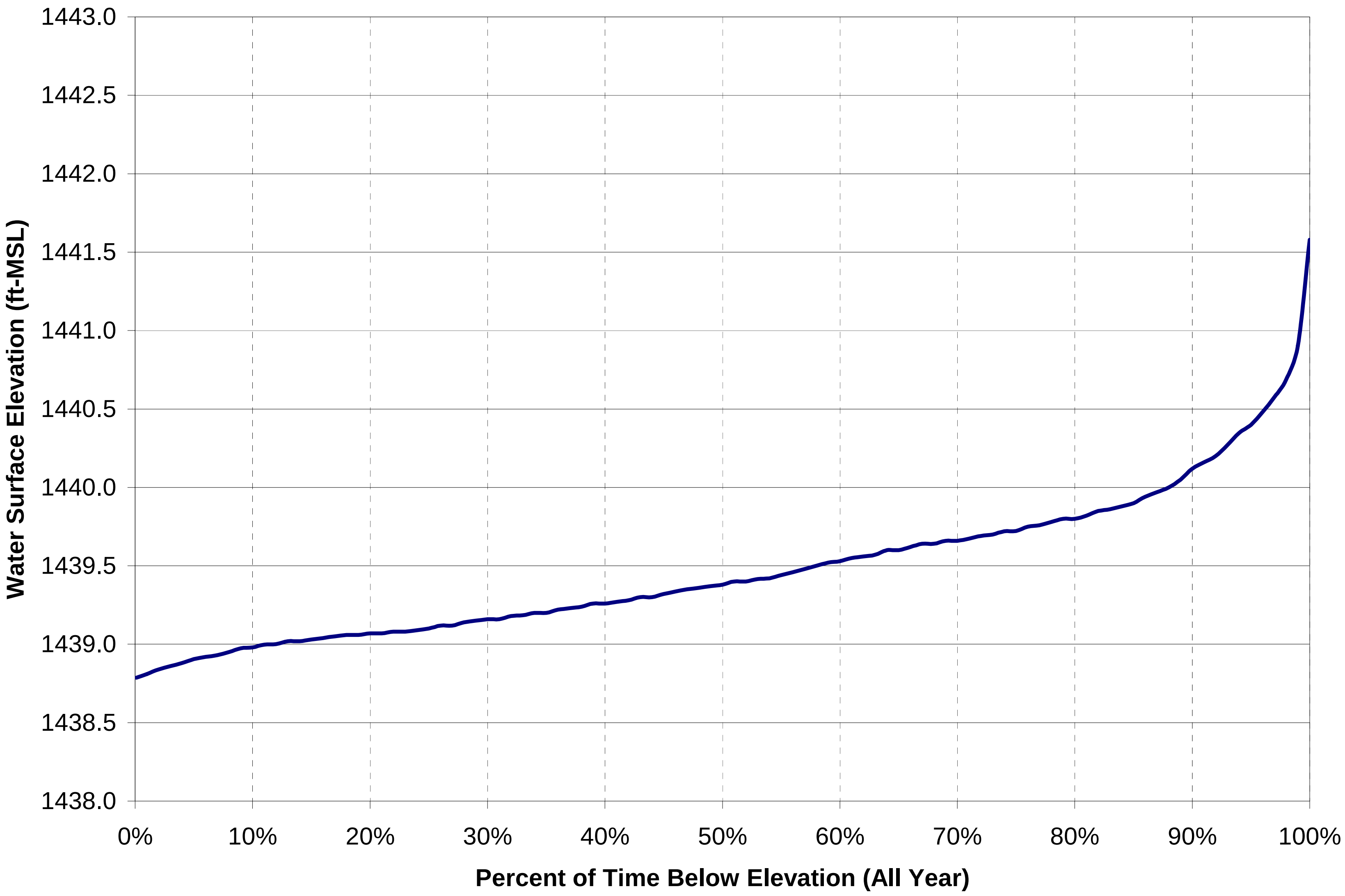
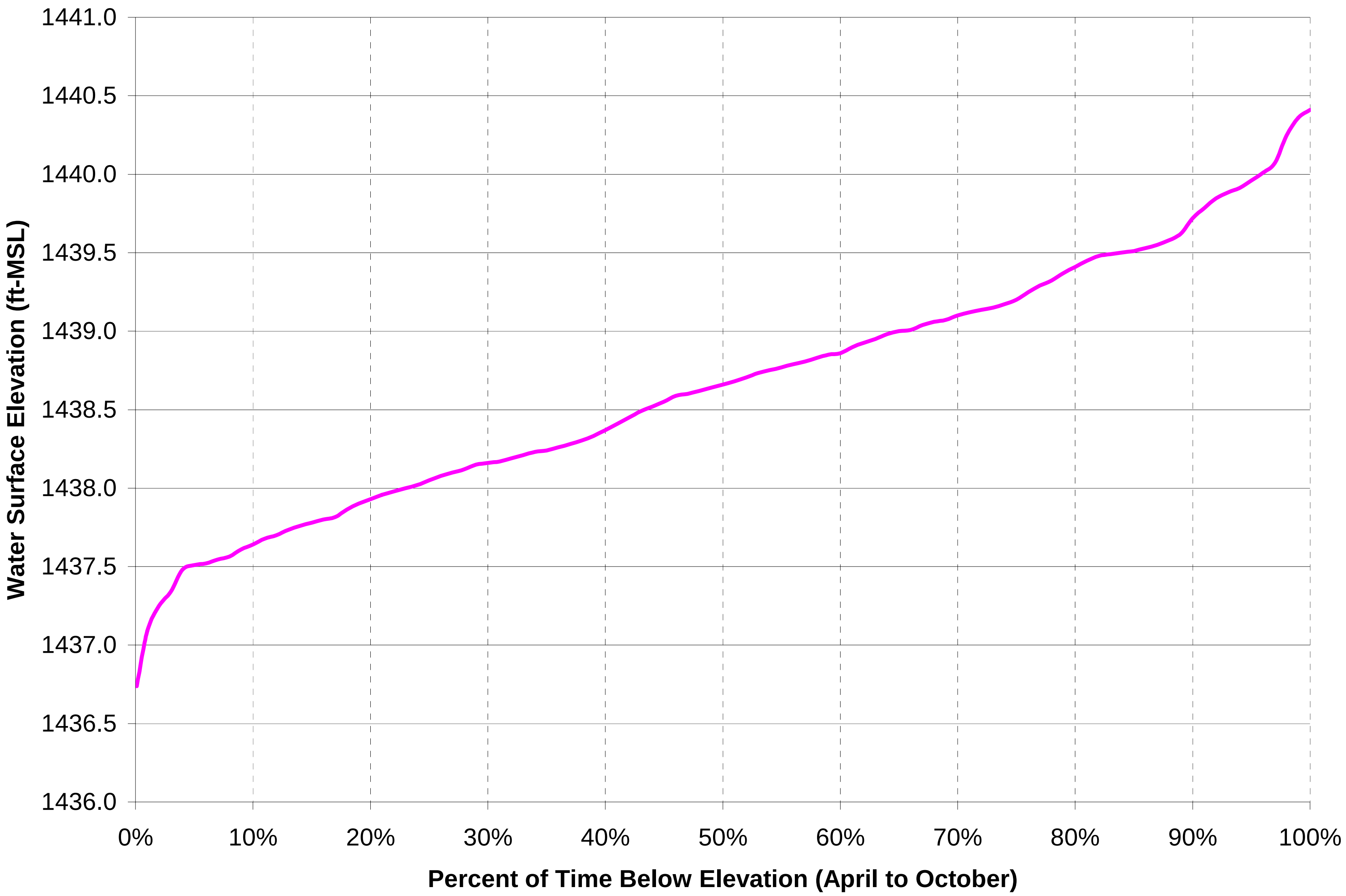


Figure 6d: Elevation – duration curves for observed Whitewater Reservoir water levels



Maps of Shoreline Retreat in Whitewater Reservoir (April-October) for Various NorthMet Make-Up Water Demands

Figure 7a: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for average flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 40 (view 1 of 4)

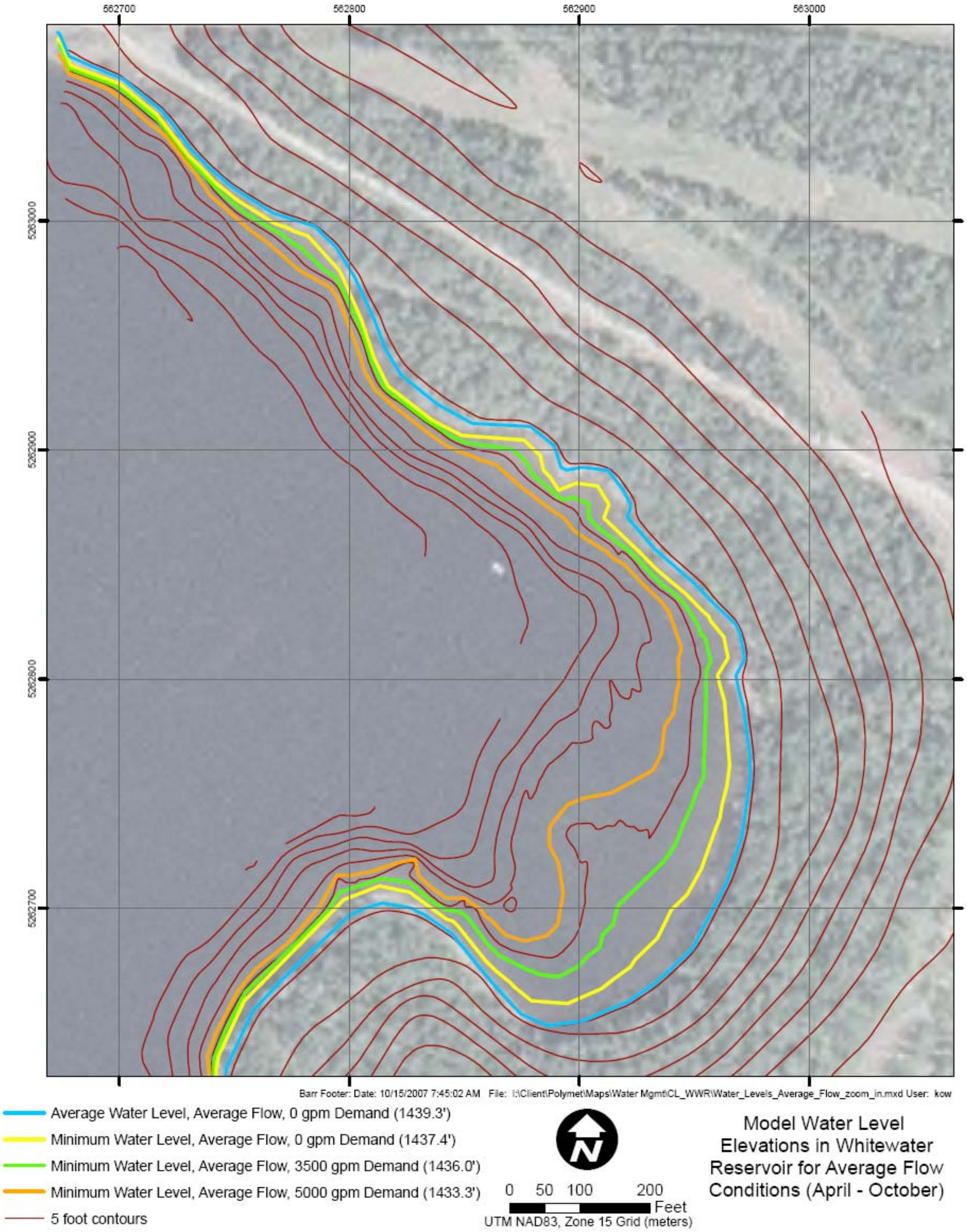


Figure 7b: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for average flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 40 (view 2 of 4)

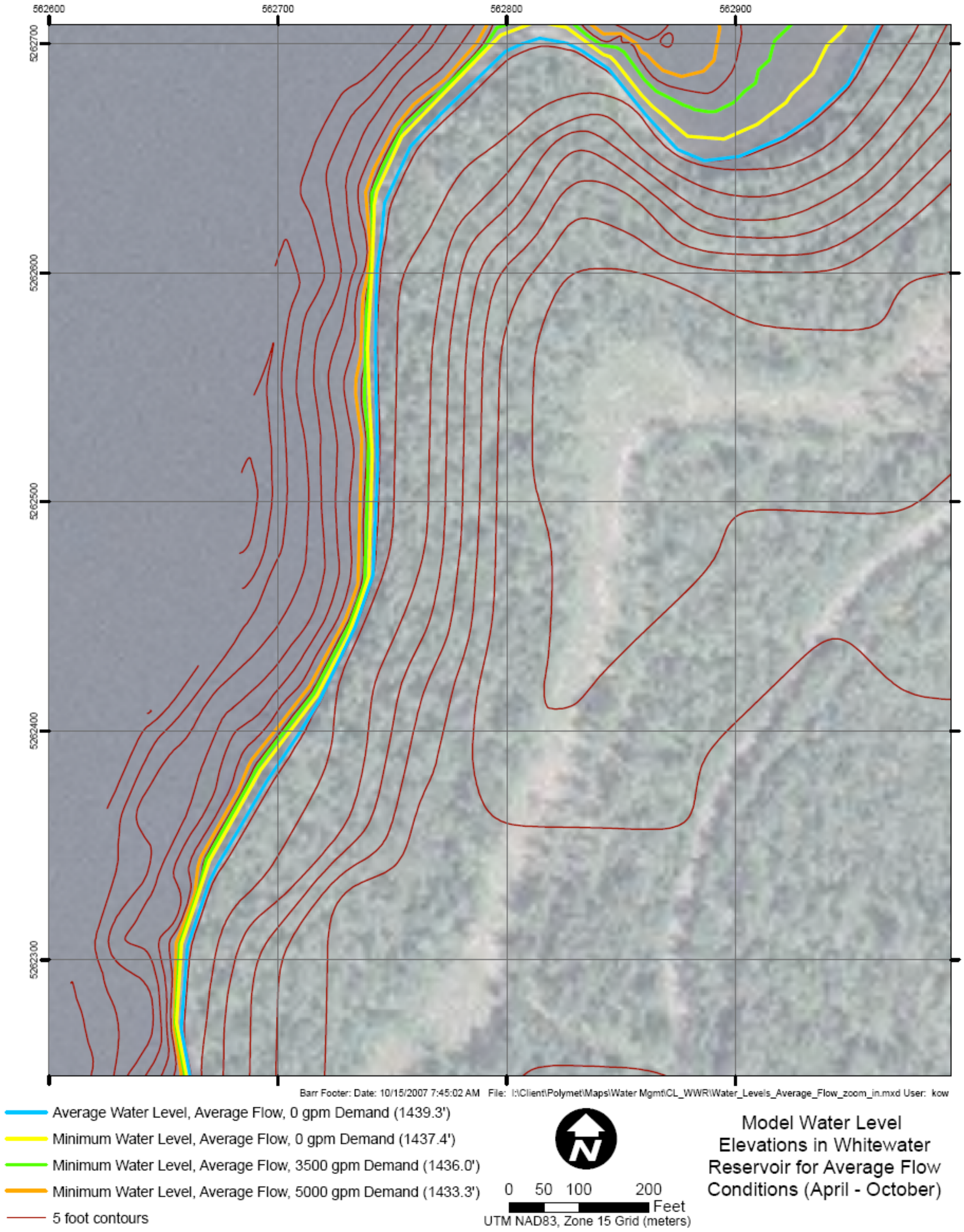


Figure 7c: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for average flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 40 (view 3 of 4)

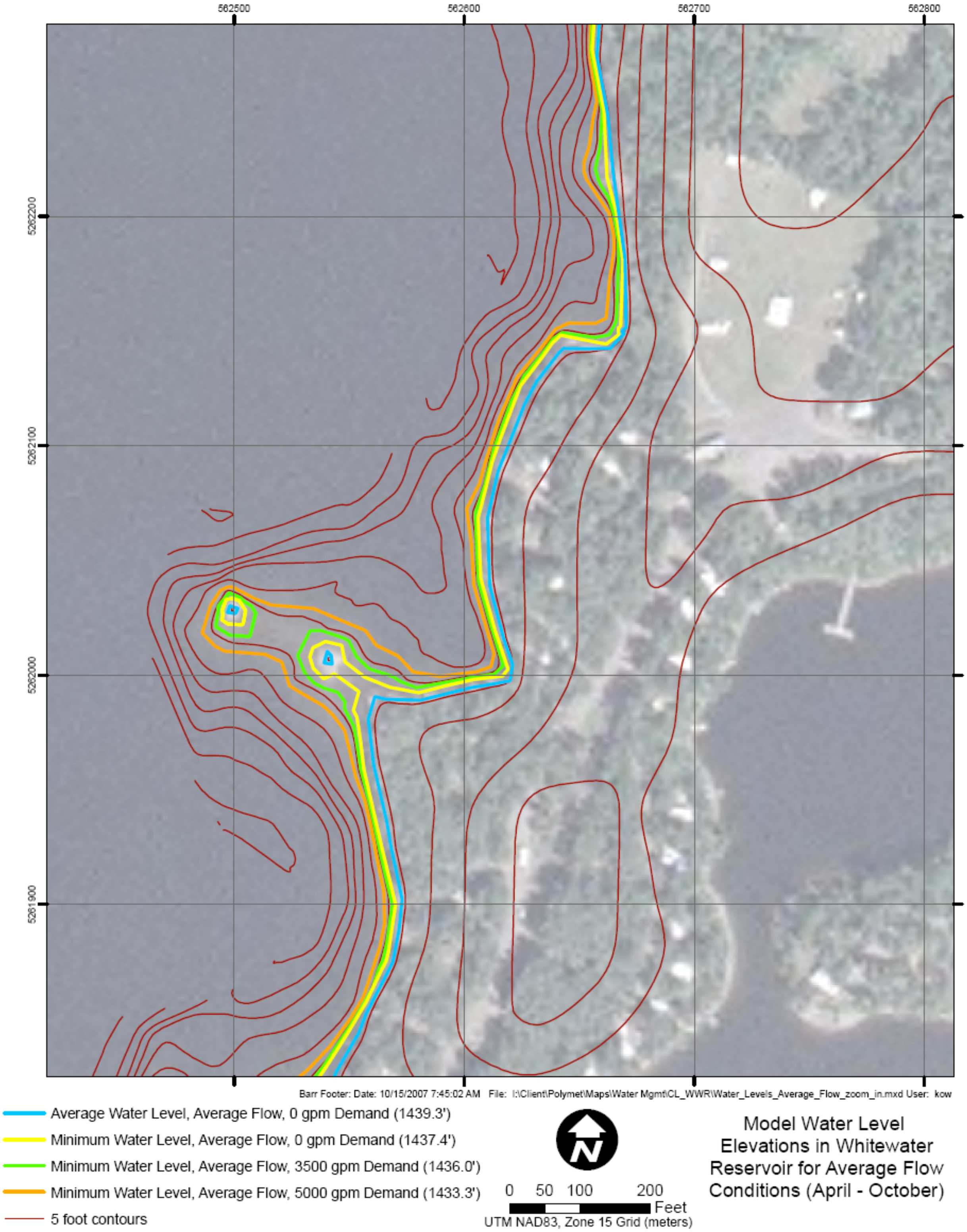


Figure 7d: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for average flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 40 (view 4 of 4)

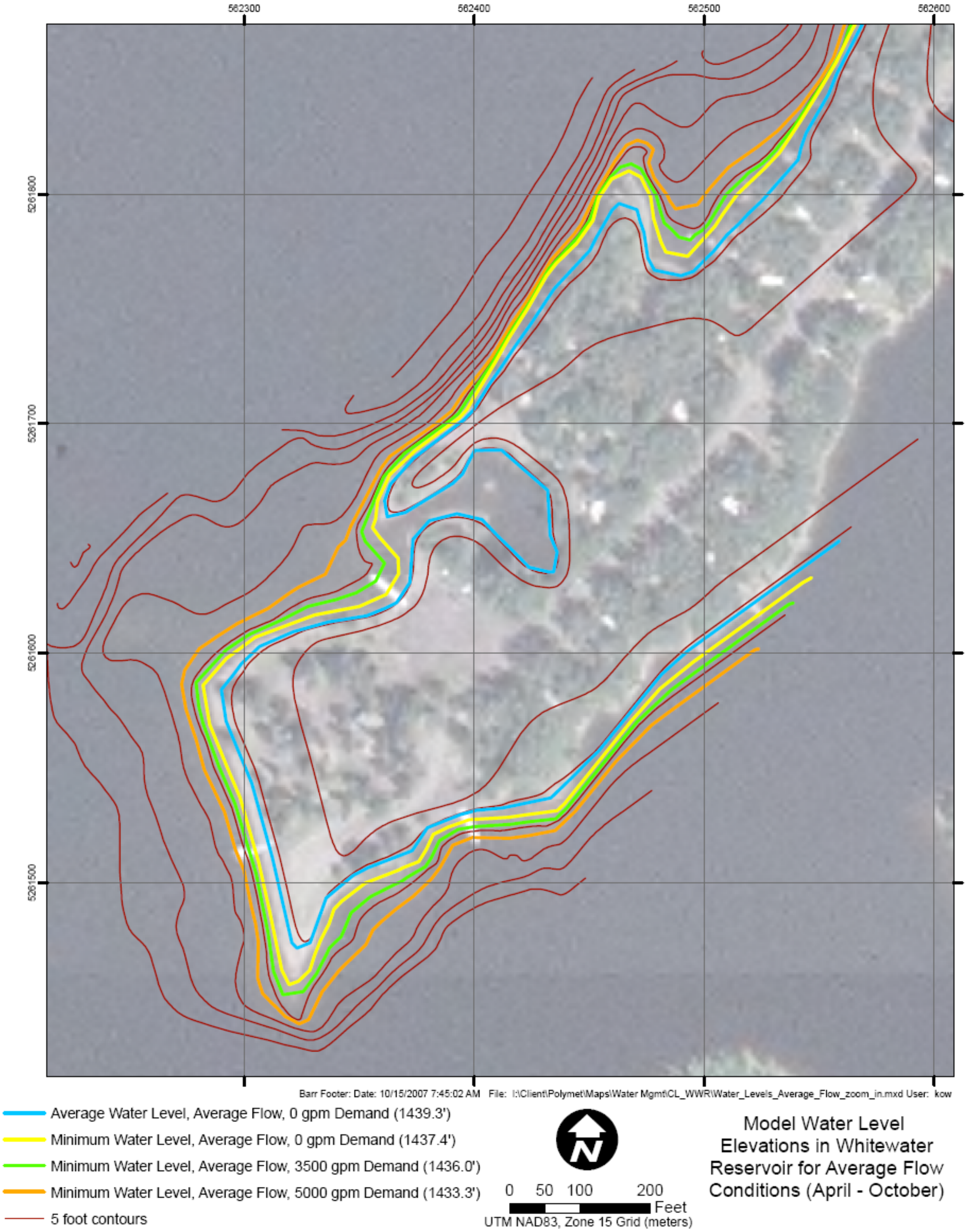


Figure 8a: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 2001-2005 flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 41 (view 1 of 4)

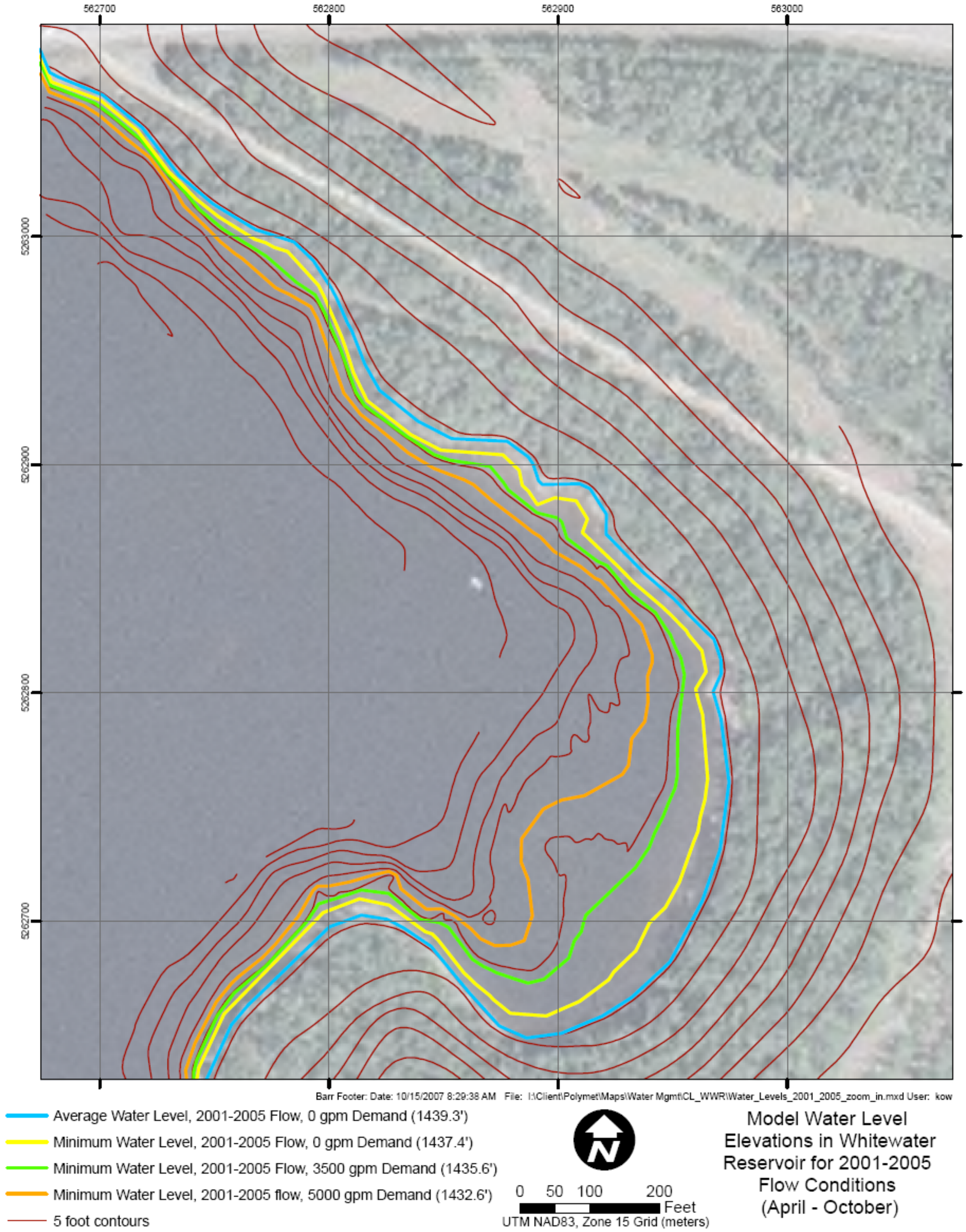


Figure 8b: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 2001-2005 flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 41 (view 2 of 4)

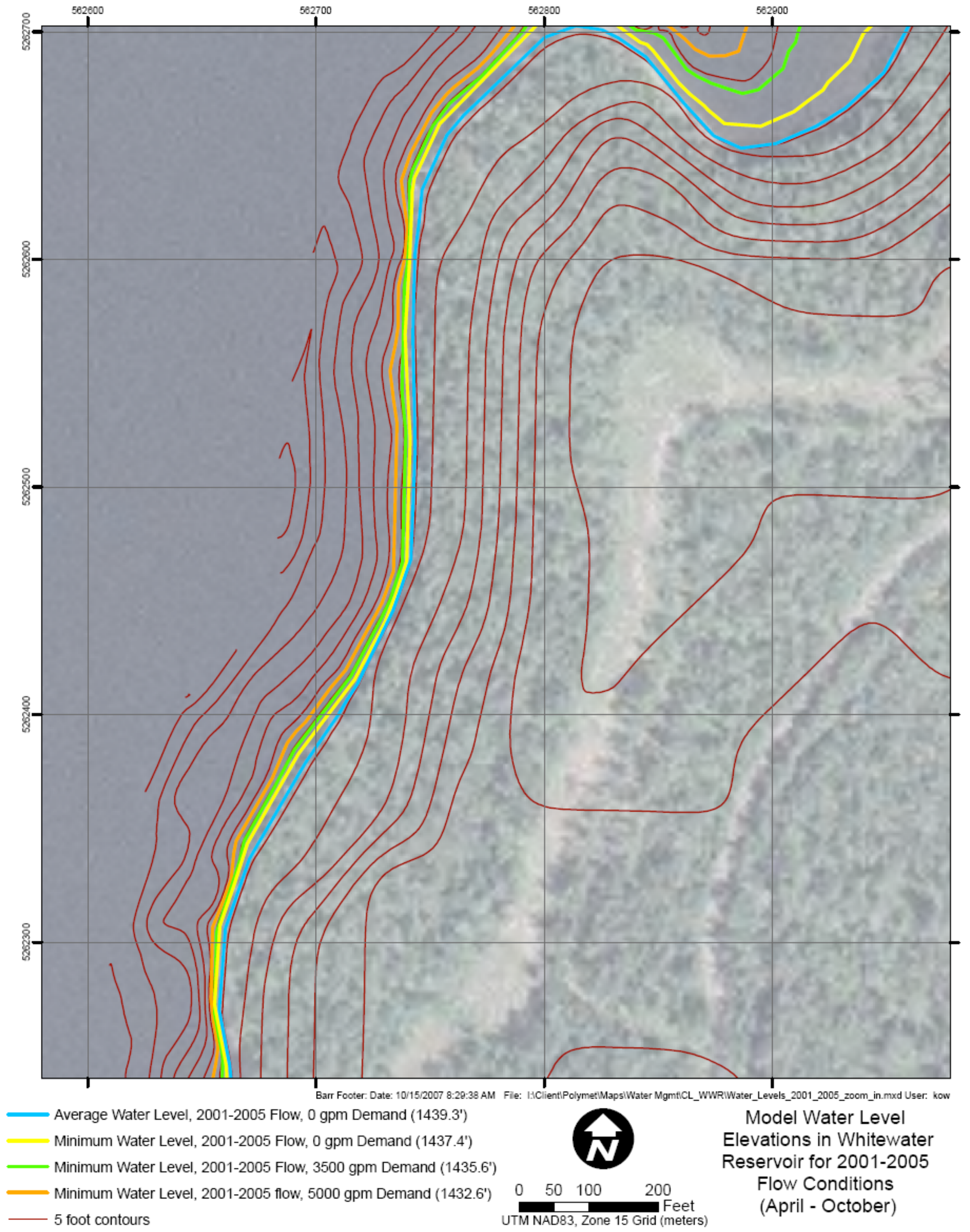


Figure 8c: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 2001-2005 flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 41 (view 3 of 4)

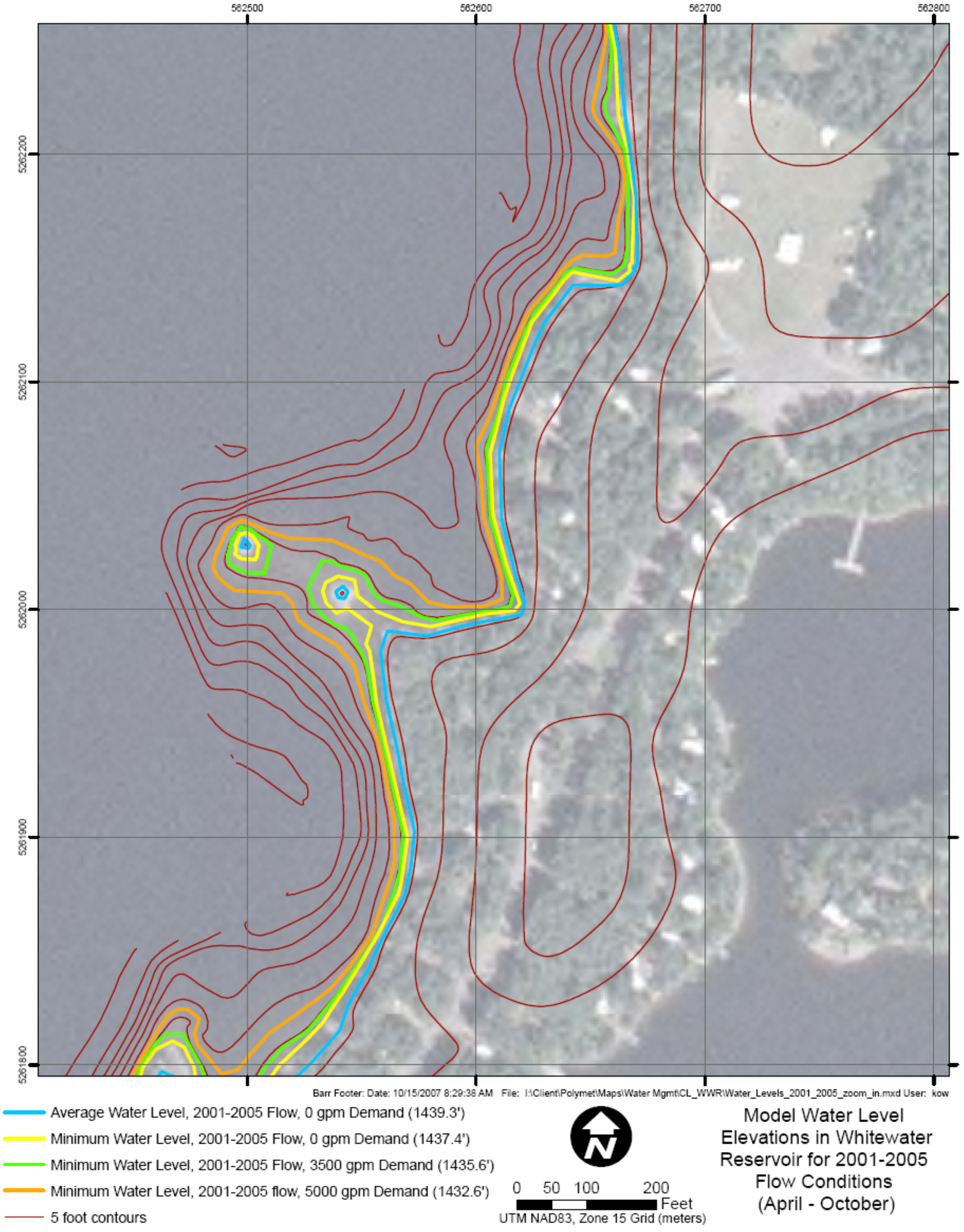


Figure 8d: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 2001-2005 flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 41 (view 4 of 4)

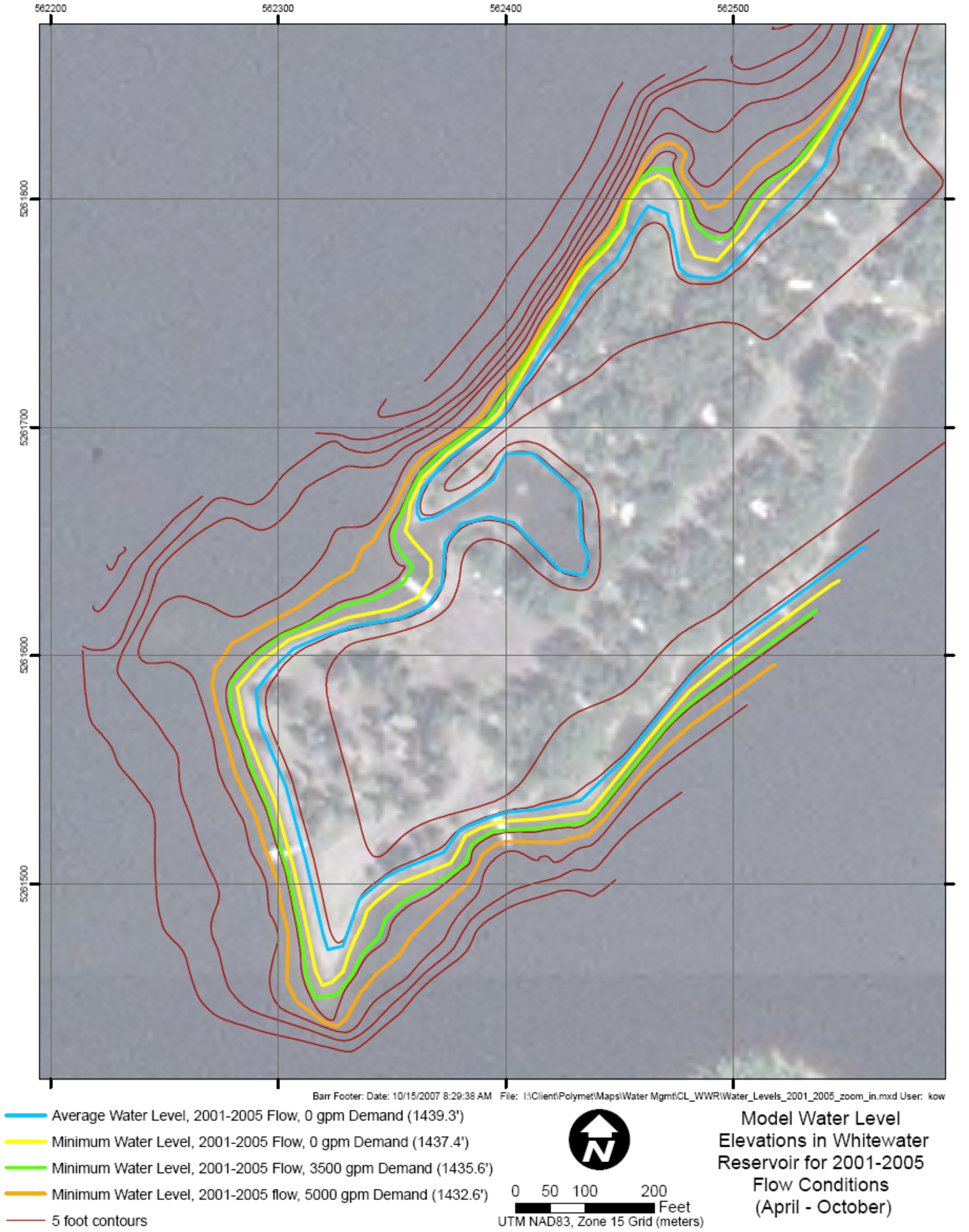


Figure 9a: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 50 year low flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 42 (view 1 of 4)

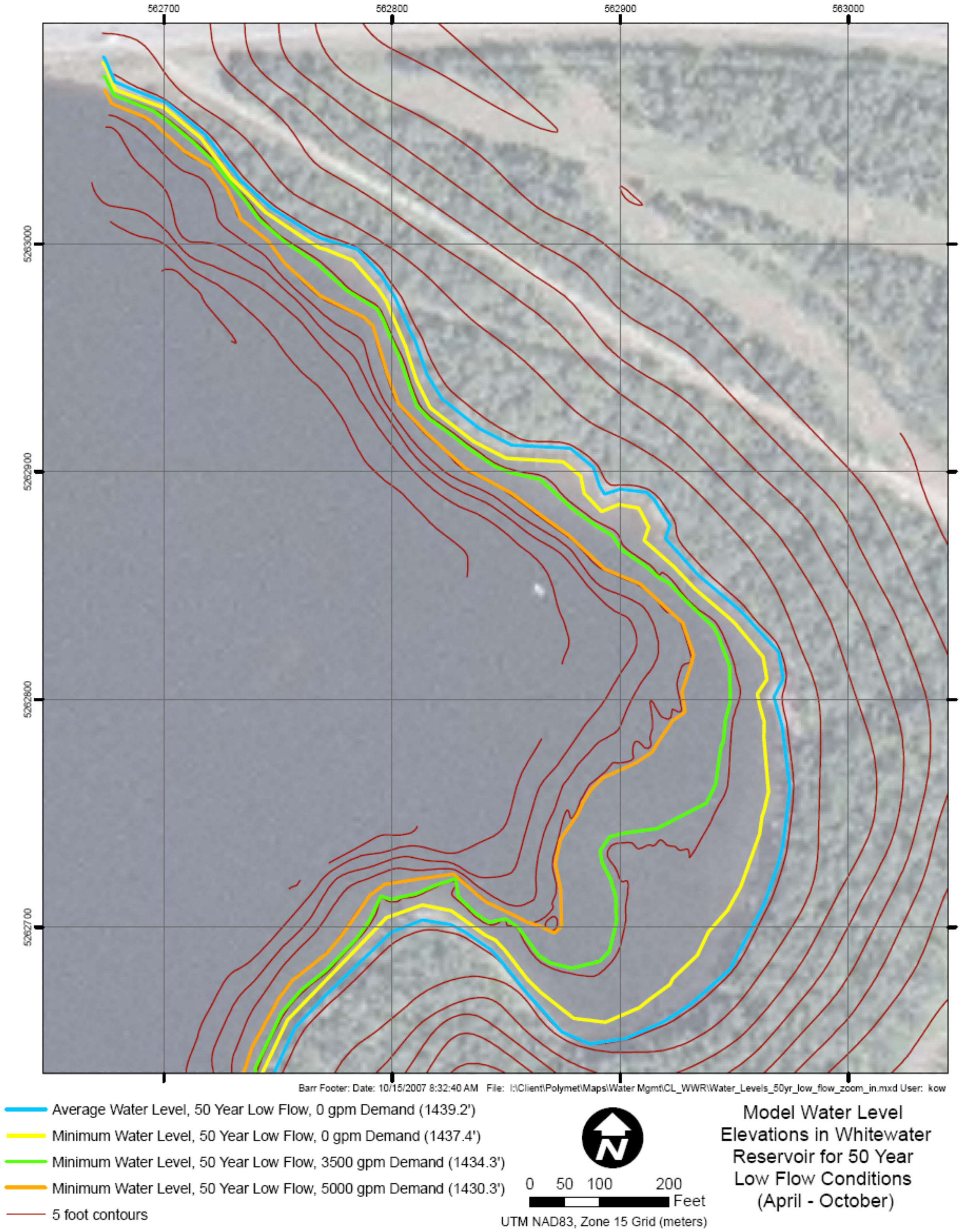


Figure 9b: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 50 year low flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 42 (view 2 of 4)

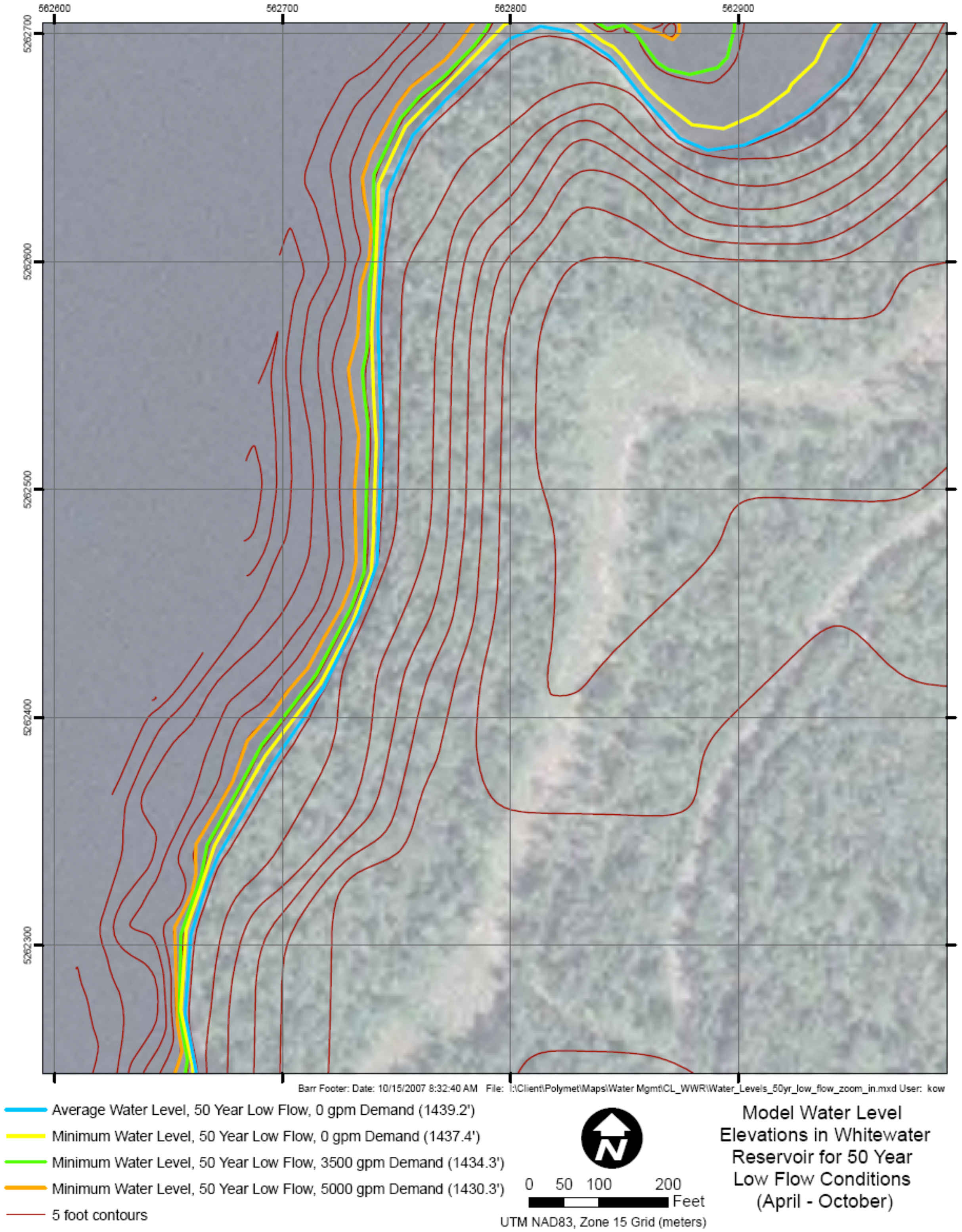


Figure 9c: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 50 year low flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 42 (view 3 of 4)

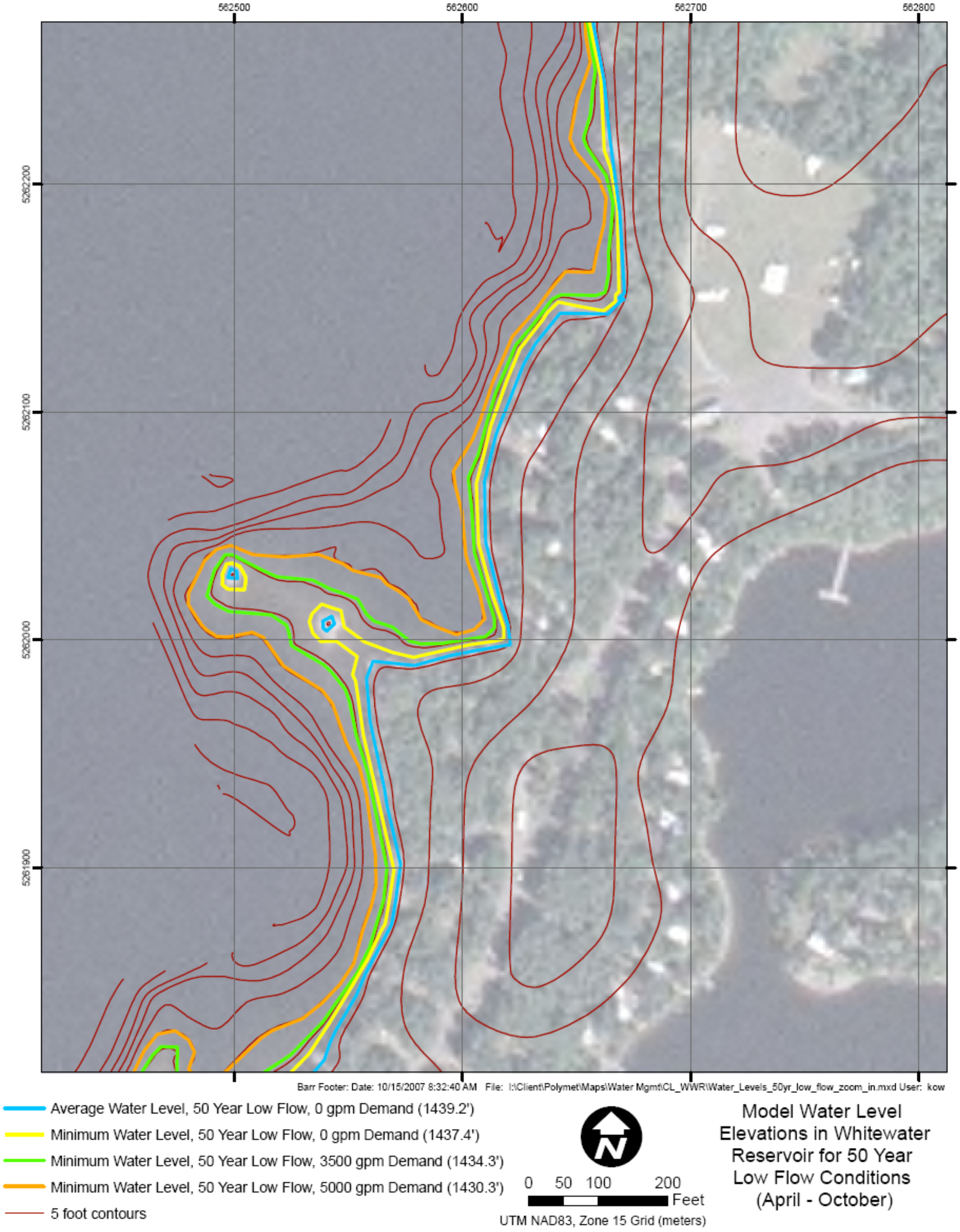
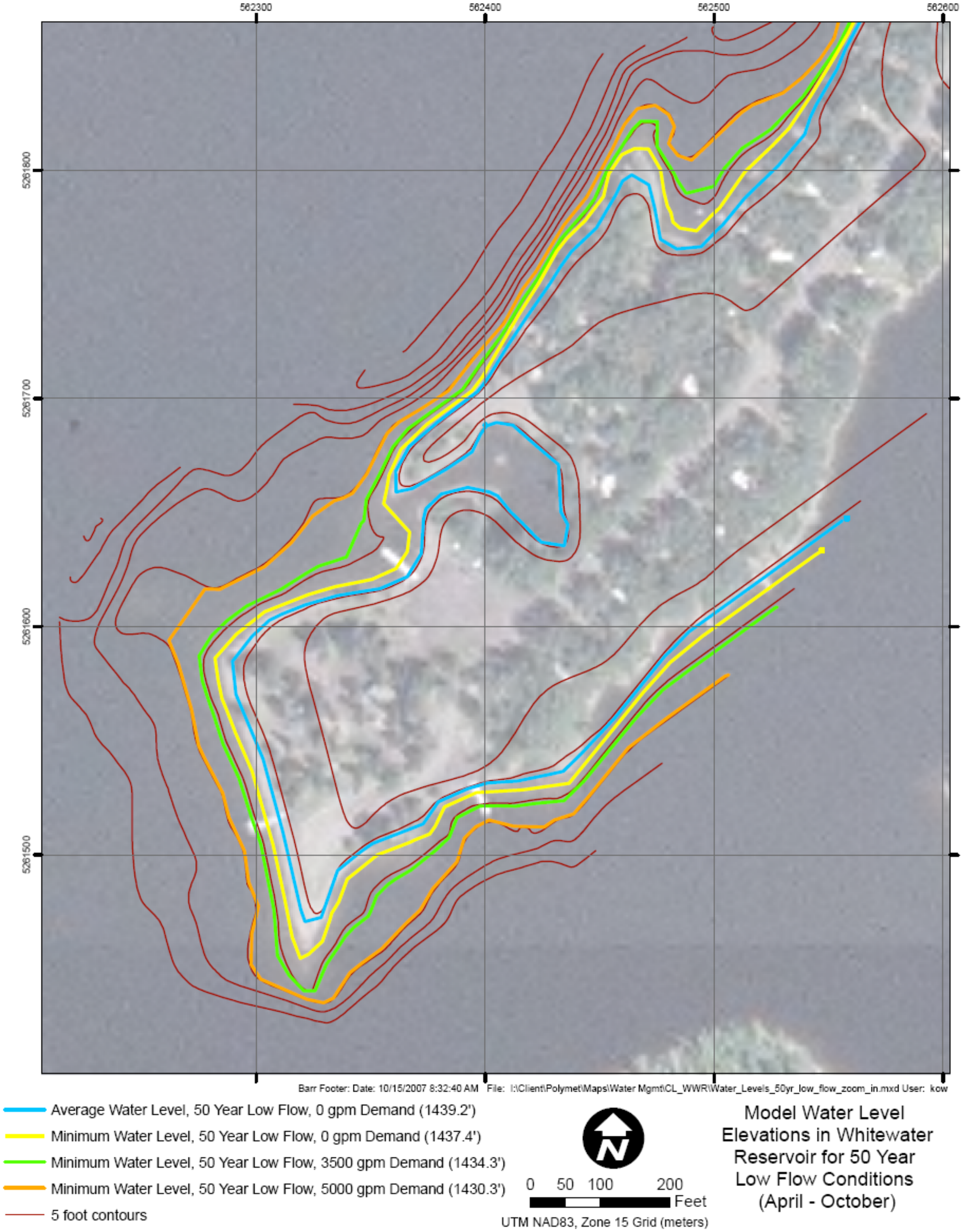


Figure 9d: Shoreline on east side of Whitewater Reservoir, based on estimated (April-October) minimum water levels for 50 year low flow conditions, Scenario 2b, and demands of 0, 3,500 and 5,000 gallons per minute. Enlarged view of Figure 42 (view 4 of 4)



RS73 Appendix A

Cumulative Impacts Work Plan

Streamflow and Lake Level Changes

Cumulative Impacts Work Plan for the PolyMet NorthMet Mine Site

PolyMet Mining, Inc.

March 19, 2007

Streamflow and Lake Level Changes: Cumulative Impacts Work Plan for the PolyMet NorthMet Mine Site

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Streamflow and Lake Level Changes: Cumulative Impacts Work Plan
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1.0 Objective

The goal of this study is to define the flow related impacts of the proposed PolyMet facilities on the adjacent rivers and lakes. It does not include the evaluation of water-quality impacts.

The management of water at the mine site will be evaluated in several separate studies. Figure 1 shows the general tasks being completed by each study, the sequence of these studies, and the predecessor tasks that provide data for these water management studies. Earlier studies will evaluate the collection and routing of reactive runoff water from the mine site (RS22 Mine Waste Water Management Systems), the collection and routing of non-contact and non-reactive runoff water from the mine site (RS24 Mine Surface Water Runoff Systems), and the perimeter diking system around the exterior of the mine site (RS25 Mine Diking/Ditching Effectiveness Study). Data from these three studies (RS22, RS24, and RS25) will be incorporated into the overall Mine Site Water Balance (RS21). The results of these studies will be incorporated into this evaluation of Cumulative Streamflow Impacts (RS73). The studies will be based on the Mine Site Plan (RS17).

Runoff yield from stockpiles will be developed in a separate study, as part of the Stockpile Design Report (RS49). The amount of runoff water that infiltrates into the system beneath the stockpiles will be determined in the Stockpile Design Report (RS49) and the Reactive Waste Segregation Report (RS23T) through analysis of capping systems (to minimize the amount of precipitation passing through the stockpile) and liner systems (to capture the water flowing through the stockpile and keep groundwater from entering the stockpile). The RS49 and RS23T reports will address operational phases (pre-capping) as well as closure/reclamation phase.

Cumulative impacts to the physical character of streams and lakes can occur from increases or decreases in flow or changes in the pattern of flow. The causes can include both point discharges (e.g., mine dewatering discharges) and changes in watershed runoff caused by land uses such as mining, timber harvest, residential development, road construction, etc. The impacts of flow changes can include erosion, sedimentation, and stream ecology. Changes in frequency of bankfull flow can cause stream degradation. Changes to streams may accumulate over time, even for non-contemporaneous impacts if, for example, a stream is eroded and degraded by one event and then further eroded by a second event.

Flow impacts to streams and lakes are regulated under the MDNR’s program for appropriations of water and for work in public waters. Physical impacts to wetlands are also regulated by the Corps of Engineers, the MDNR and the MPCA.

PolyMet will have point discharges of industrial wastewater to the Partridge River (from the Mine Site) and to the Embarrass River (from the Processing Facility and Tailings Basin). The discharges to the Embarrass River are expected to be relatively small in volume. (Other changes to the Embarrass River that might be cumulative are limited to the small and intermittent discharge from the Babbitt Wastewater Treatment Plant, forest harvesting and the impacts of rural residential development in Embarrass Township. Again, these are relatively small impacts.) Most mining-related discharges for Northshore Mining Company and Cliffs Erie are not to the Embarrass but to the Partridge. Therefore, the possibility of significant impacts to the Embarrass River via either direct discharge or cumulative impacts of discharge (including PolyMet) is believed to be small, and will not be addressed.

PolyMet’s net effect on the hydrology of the upper Partridge River is also expected to be larger than its effect on the Embarrass River. Northshore Mining Company also operates the Peter Mitchell Pit in the headwaters of the Partridge River, upstream of Polymet, resulting in potential combined impacts. In addition, PolyMet will appropriate water for the Processing Plant from Colby Lake (which is part of Partridge River drainage), raising the possibility of decreases in lake discharge and lake levels from present conditions. Short-term peak discharges from the Mine Site can be mitigated by control of outflow from sedimentation and treatment basins, if necessary, to limit potential impacts on stream geomorphology. During reclamation, there will be a period of time when the PolyMet mine pits will be filling with water and the flow to the Partridge River will be reduced as water accumulates in the mine pits. The cumulative impact of greatest concern is the potential for combined peak dewatering from Polymet and Northshore Mining, or combined reduction in base flow caused by abandoned pits filling with water.

This evaluation will combine these impacts to the surface water runoff throughout the watershed and evaluate the overall changes to flows along the Partridge River and to lake levels on Colby Lake and Whitewater Reservoir. The impacts of these flow changes on the stream stability and the lake shorelines will also be evaluated.

2.0 Hydrologic Evaluation

2.1 Assessment

A quantitative assessment of cumulative impacts due to changes in flow will be performed for the Upper Partridge River. This assessment will focus on flow changes in the immediate vicinity of the proposed project. The hydrologic modeling will continue downstream to the USGS gaging station located upstream of Colby Lake.

Following this quantitative assessment, a qualitative assessment will be made of resources further downstream to evaluate whether cumulative impacts may occur at greater distances. This qualitative assessment will include Colby Lake and Whitewater Reservoir.

2.2 Base Condition

An evaluation of the geomorphology of the Partridge River in the vicinity of the project was conducted in 2004. It found that the upper Partridge River was in good condition in the reaches evaluated, suggesting that historic mining upstream of Polymet has not resulted in channel stability problems. Therefore it is proposed to take the present condition as the baseline condition.

2.3 Hydrologic Analysis

The cumulative impacts of the PolyMet mine site on the hydrology of the Partridge River will be performed using the XP-SWMM computer model[1]. This model can analyze unsteady flow conditions that may occur in flat areas and in storm sewer systems. The model can also include the impacts of evaporation and groundwater inflows over long periods.

The XP-SWMM analysis will be developed to simulate the overall watershed to the gage upstream of Colby Lake. This model will be calibrated to gage data to represent existing conditions (see Section 2.4). To analyze the cumulative impacts, model data from the mine site XP-SWMM models (RS21) will be incorporated into the overall watershed model. Additional information from other potential sources (other mining activities, timber harvest, residential development, road construction, etc) will also be incorporated for the evaluation of cumulative impacts.

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[1] The United States Environmental Protection Agency's Storm Water Management Model (SWMM), with a computerized graphical interface provided by XP Software (XP-SWMM), was chosen as the floodplain computer-modeling package for this study. XP-SWMM uses precipitation and watershed information to generate runoff that is routed simultaneously through complicated pipe, channel, and overland flow networks. Simultaneous routing means that flow in the entire system is modeled for each time increment simultaneously, then the model moves on to the next time increment, and so on. Simultaneous routing allows the model to account for flows in pipes, flows detained in ponding areas, the effects of backwater conditions (such as backflow through pipes), and the complexity of routing overflows in directions different than the pipes convey the piped flows. XP-SWMM can simulate either single design events or continuous historic rainfall.

The time period that is evaluated for the mine site comparison to existing conditions will be long enough to include average, wet, and dry climatic conditions. The XP-SWMM model is an unsteady flow model that will be used to evaluate the effects of precipitation, evaporation, wetland and pond storage, and groundwater inflow to the mine pit(s). Modeling requirements and proposed methodology are briefly described below. Further details on the assumptions made and the modeling methods will be provided in the report for this study.

The tributary watershed upstream of Colby Lake will be divided into approximately 30 subwatersheds at crossings that restrict the flow and at the confluence of major streams. Watershed divides will be based on the best available topography including Mesabi project topographic mapping, but a large part of the watershed will be delineated using the USGS quadrangle maps.

Watershed input data consists of area (acres), impervious percentage (%), slope (ft/ft), and width (ft) for each subwatershed. All land use practices within a watershed impact the quantity of runoff generated. Each land use contributes a different quantity of runoff due primarily to the amount of impervious areas. The impervious areas input into the XP-SWMM computer model must, by definition, be hydraulically connected to the drainage systems being analyzed. The direct or connected impervious percentage includes driveways and parking areas that are directly connected to the storm sewer system. Rooftops draining onto adjacent pervious areas would not be treated as effective impervious areas. Since the system being analyzed is primarily comprised of natural drainage swales and very little storm sewer, the majority of the impervious surfaces will likely be considered unconnected (i.e., draining to a pervious area). Watershed "width" in XP-SWMM is used along with velocity and channel length to compute the time of concentration. Watershed "width" in XP-SWMM is defined as twice the length of the main drainage channel, with adjustments made for watersheds that are skewed (i.e. the areas on both sides of the main drainage channel are not equal). Watershed width will be calculated using Arc View scripts developed by Barr Engineering. In accordance with the SWMM user's manual (*Storm Water Management Model; Version 4 User's Manual* 1988), the width parameter may be used for peak runoff calibration.

Additional required input data includes runoff infiltration rates, depression storage losses, and overland flow roughness factors:

- Infiltration is the movement of water into the soil surface. For a given storm event, the infiltration rate will tend to vary with time. At the beginning of the storm, the initial infiltration rate is the maximum infiltration that can occur because the soil surface is typically dry and full of air spaces. The infiltration rate will tend to gradually decrease as the storm event continues because the soil air spaces fill with water. For long duration storms the infiltration rate will eventually reach a constant value, the minimum infiltration rate. The Horton infiltration equation will be used to simulate this variation of infiltration rate with time. Infiltration parameters will be based on published data and may be modified during model calibration. Sources for this data may include: Hydrologic Analysis and Design, McCuen, 1989; Relative Infiltration and Related Physical Characteristics of Certain Soils, Free, Browning, and Musgrave, USDA Technical Bulletin 729, 1940; Hydrology for Engineers, Linsley, Kohler, and Paulhus, 1958; Hydrology Handbook, ASCE Manual of Engineering Practice No. 28, 1949; and XP-SWMM manuals. Stockpile infiltration rates will be developed as part of the Stockpile Design Report (RS49). Soil types will be approximated using available soils maps. The impacts of large wetland areas in the watershed that are located on top of bedrock formations may significantly alter the infiltration capacity. Therefore, infiltration will be one of the parameters that is used for calibration, as described below.
- Depression storage inputs, the areas that must be filled with water prior to generating runoff from both pervious and impervious areas, will be set within the general range of published values. It represents the initial loss caused by such things as surface ponding, surface wetting, and interception. The model handles depression storage differently for pervious and impervious areas. The impervious depression storage is replenished during dry simulation periods by evaporation. The water stored as pervious depression storage is subject to both infiltration and evaporation. The pervious and impervious depression storage inputs will be based on published data and may be modified during model calibration.

- Overland flow is the surface runoff that occurs as sheet flow over land surfaces prior to concentrating into defined channels. In order to estimate the overland flow or runoff rate a modified version of Manning's equation is used by XP-SWMM. A key parameter in the Manning's equation is the roughness coefficient. The shallow flows typically associated with overland flow result in substantial increases in surface friction. As a result the roughness coefficients typically used in open channel flow calculations are not applicable to overland flow estimates. These differences will be accounted for by using an effective roughness parameter instead of the typical Manning's roughness parameter, as published in *HEC-1 User's Manual*, September 1990 and in *Engineering Hydrology: Principles and Practices* (Ponce, 1989). These overflow flow parameters may be modified during model calibration.

Subcatchment infiltration will be coupled to groundwater so that the unsaturated zone can interact with the infiltration from the watershed surface. Decreased infiltration increases surface runoff. For example, the water table can rise to the ground level from excessive infiltration.

Initial groundwater interaction parameters will be estimated based on typical values representative of the watershed and they will be modified during model calibration to obtain a good fit to existing flow data (see Section 2.4). The groundwater interaction options include:

- Evapotranspiration from both the upper and lower zone may also be simulated.
- Dynamic groundwater table. If it rises to the surface, the upper zone disappears and infiltration is stopped. If it drops below the bottom elevation of the conduit, groundwater outflow will cease.
- Groundwater evaporation/transpiration parameters. Evapo-transpiration from the upper zone represents soil moisture lost via cover vegetation and by direct evaporation from the pervious area of the subcatchment. Evapo-transpiration from the lower zone is typically small compared to other terms.
- Potential Evaporation available for subsurface water loss; the difference between total evaporation input to the model and evaporation used by the surface routing.
- Wilting Capacity; the soil moisture content at which plants can no longer obtain enough moisture to meet transpiration requirements; they wilt and die unless water is added to the soil.
- Maximum depth over which significant lower zone transpiration occurs, ft [m]. Lower zone evapotranspiration occurs after upper zone evapo-transpiration by removing the remaining fraction linearly as a function of depth to the water table. If the water table drops below this depth no lower zone evapotranspiration occurs.
- The amount of water a well-drained soil holds after free water has drained off, or the maximum amount it can hold against gravity, expressed as a moisture content fraction.
- Field capacity; must be greater than the wilting point (since it occurs at lower tensions), and less than 0.9 times the porosity.
- Groundwater infiltration percolation parameters. Percolation represents the flow of water from the unsaturated zone to the saturated zone, and is the only inflow for the saturated zone.
- Fraction of maximum Evapo-transpiration rate assigned to the upper zone.
- Initial moisture content
- Hydraulic conductivity vs. moisture content curve-fitting parameter, dimensionless. This parameter can be estimated from an exponential fit of hydraulic conductivity to soil moisture, assuming such data is available. This parameter is a sensitive calibration parameter for movement of unsaturated water into the saturated zone.
- Coefficient for unquantified losses, in./hr [cm/hr]. Deep percolation represents a lumped sink term for unquantified losses from the saturated zone. The two primary losses are assumed to be percolation through the confining layer, and lateral outflow to somewhere other than the receiving water. The model provides for a first order decay, typical of water table recession curves.
- Global groundwater outflow calculation parameters. Groundwater discharge represents lateral flow from the saturated zone to the receiving water. To this end, a general equation is provided to formulate the groundwater flow.

The routing data that is required by the model includes: (a) pipe locations, sizes, types, materials, and elevations; (b) natural channel cross-sections; (c) storage basin elevation, volume, and outflow characteristics; and (d) surface flow characteristics (overland flow upstream of the channels). We will use the best available data to estimate these characteristics (U.S.G.S quadrangle maps, field data collection, available plans, etc.). The watershed contains large areas of wetlands, and storage in these wetlands may have a significant impact on the total flows. The storage that is available in upstream wetlands will be investigated based on previous studies and may also be adjusted during the calibration process. Information on stream configuration and crossings will be obtained from readily available plans and limited field observations.

Climatic input data consists of rainfall, temperature, wind speed, water surface evaporation, and snowmelt parameters. These data are used by the model to generate a snowpack, watershed runoff (due to rainfall and snowmelt), and estimate water surface fluctuations resulting from evaporation:

- Daily precipitation data will be obtained from National Weather Service (NWS) rainfall gages closest to the mine site. Data from several stations may be required since the period of record is likely not continuous; therefore the station that is closest to the mine site will be used for each period.
- Continuous simulations, like those required for this study, require a complete time series of daily maximum and minimum temperatures. We anticipate using temperature data from the same NWS station as the daily precipitation, or the site closest to the mine site that has available data.
- The wind speeds are used for melt determination during periods of rainfall on snow cover conditions (according to the SWMM user's manual). The higher the values of wind speed, the greater are the convective and condensation melt terms (we have found that the SWMM model is not very sensitive to modifications of the wind speed). Average monthly wind speeds will be obtained from the NWS station that is closest to the mine site that has available data. During continuous simulations of longer than a single year, one monthly average wind speed value will be entered for each month of the year. For example, in a 10-year simulation there would be 10 individual monthly average wind speed values for March. These 10 values will be averaged to produce one overall average wind speed for March, which will be input into the model.
- Evaporation plays several vital roles in continuous simulations. It important in estimating the amounts of depression storage available prior to a given storm event and therefore ultimately plays a key role in subwatershed runoff estimates. Secondly, evaporation impacts the surface water elevations of a pond and the volume of water in the various ponds. This in turn affects the volume available to store the runoff prior to conveying any excess to the next downstream basin. Average monthly evaporation rates will be obtained from the Meyer Model estimates from other previous studies conducted at nearby sites. The Meyer Evaporation Formula is basically an empirical method that was found to give reasonable results when used in a water balance application. The model uses daily precipitation, temperature and other monthly climate data (average daily wind speed and relative humidity).

  An average monthly evaporation rate is required for all the months in a continuous XP-SWMM model. This rate is subtracted from the rainfall and snowmelt intensities at a given time step and is also used to replenish the depression storage. The evaporation used in the runoff generation will be handled in a similar way as the wind speed. During continuous simulations of longer than a single year, one monthly average evaporation value will be entered for each month of the year. For example, in a 10-year simulation there would be 10 individual monthly average evaporation values for June. These 10 values will be averaged to produce one overall average evaporation rate for June, which will be input into the model.
- For continuous modeling, the precipitation depths from the NWS station are used along with the hourly temperatures determined from the daily minimum and maximum temperatures to determine if the precipitation is rainfall or snowfall. If the estimated temperature is below a specified dividing temperature (e.g., 35° F), the precipitation is treated as snowfall and will be stored in the model as a snowpack. This temperature (35 ° F) has been shown to be the dividing line between equal probabilities of rain and snow (SWMM user's manual).

XP-SWMM utilizes the interpolated hourly temperatures in the snowmelt computations. The snowmelt is generated using a degree-day type equation during dry weather and Anderson's NWS equation during rainfall periods (SWMM user's manual). Before any melt can occur, the snow must be heated to a base temperature. The computed snowmelt is then handled in a similar manner as rainfall (i.e., the model allows depression storage and infiltration losses prior to generating runoff). There are numerous input parameters required for snowmelt modeling. Since no field data is available, the parameters will be set within the range of values published in the SWMM user's manual and may be modified during model calibration (see the SWMM user's manual for additional information about snowmelt modeling).

2.4 Model Calibration

The XP-SWMM model will be calibrated to available flow data in order to estimate probable streamflow changes. The predicted change in flow characteristics will be estimated at appropriate stream reaches near the Polymet Mine site.

There are limited streamflow data for the upper Partridge River. There are two United States Geological Survey stream gauging stations on the Partridge River with long term flow records: one above Colby Lake at Hoyt Lakes (#04015475) and one near Aurora downstream of Colby Lake (#04016000). From 1978 to 1988 the U.S.G.S. operated gaging station #04015475 on the Partridge River just upstream from the confluence with Colby Lake. During this period Reserve Mining Company (the predecessor to NorthShore Mining Company) was not pumping to the Partridge River so this record will probably be usable for calibration. This assumption will be verified, especially with respect to the impacts of any overflows that may have occurred from Reserve Mining Co. pits.

There is also limited flow data (13 months) from DNR's 2004 East Range Hydrology Study. NorthShore discharge data are available for this time on at least a monthly basis.

The calibration will attempt to match the major trends in average daily water flows over the period of analysis, testing the goodness of fit with various modifications.

Hydrologic modeling will include the effects of past and present actions (through the date of monitoring) including:

- Existing Cliffs Erie and discharges from pits (as of date of monitoring)
- Modification of land use (including wetland loss) by past mining practices within the upper Partridge River watershed
- Existing discharge from Northshore Mining Company Mine and Crusher area
- Typical timber harvest activities on SNF, state and county lands and private lands.

2.5 Long-Term Simulations

If the model can be calibrated to the gaging record with reasonable accuracy, the flow record will be extended using meteorological data. The streamflow record will be adjusted to remove the effect of known pumping or pit overflow discharges. This extended record will be long enough to analyze both wet and dry climatic conditions, although modeling limitations may require running the model in several segments of the overall record to limit the run time. The error of estimate associated with use of the model will be displayed and discussed in light of its intended use in the EIS.

The extended flow record will be used to create a synthetic, local streamflow record for points of interest near the PolyMet site. The relationship of the sub-model to the overall model calibration will be checked to a limited extent using individual streamflow measurements done by PolyMet in 2004. The points of interest will help define the impact to flows and flood levels near the site, and will likely include:

1. Dunka Pit Road crossing
2. Railroad crossing downstream of Dunka Pit Road
3. 2004 water quality sampling location PM-16 / Stream classification site 2

4. Sensitive reach identified in the Partridge River Level 1 Rosgen Geomorphic Survey RS26

These points of interest locations are identified on Figure 2. The synthetic local streamflow record will be summarized in relevant flow statistics, including 7Q10, 1.5-year, and, if reasonable, 10- and 100-year flow estimates. The latter may be estimated by single-event simulation using standard estimates of extreme rainfall events; either the 24-hour SCS Type II storm or the 10-day snowmelt, whichever appears to be the critical event based on gaging station data from the Partridge River. The average daily flows that are computed for the average, wet, and dry climatic conditions will be presented in the report.

The hydrologic models will be modified to include actions since the date of the monitoring and potential future actions including:

- Net hydrologic effects of PolyMet Mine Site discharges to Partridge River and appropriations for PolyMet
- Long-term flow management of PolyMet mine pit during and after filling of pit
- Any potential changes in water discharge from Northshore Mining Company discharges in Partridge River watershed
- Any reasonably foreseeable changes to timber harvest activities on SNF, state and county lands and private lands.

Comparisons between the existing conditions and the modified future conditions will be conducted and the results summarized in the report.

3.0 Cumulative Impact Analysis

3.1 Partridge River

The streamflow data as described above will be augmented by available geologic, soils, and ecological data and summarized to describe the condition and sensitivity of the Partridge River in the study area. The river will be classified in terms of sensitivity to streamflow change, using the Rosgen classification approach.

The threshold of significance for the cumulative impact assessment for the upper Partridge River will be the likelihood of major change in stream morphology as defined by the Rosgen classification method (Rosgen, 1994) or other applicable method. This analysis will be based on stream reconnaissance completed in 2004 by PolyMet as a base condition and augmented by available geologic, soils, and ecological data to describe the sensitivity of the stream in the study area. The predicted change in flow characteristics will be estimated at the different stream reaches. The possibility of significant changes in stream morphology and ecology due to flow changes will be evaluated, based on the Rosgen methodology, existing information, and applicable research.

Where significant impacts are predicted, the EIS will suggest and evaluate mitigative measures such as controls on rate or volume of discharge or modifications to the water management plan to redirect water to less-sensitive stream locations. It will also evaluate the need for additional data collection to be addressed in the permitting processes.

3.2 Colby Lake and Whitewater Reservoir

After completion of the quantitative analysis of the upper Partridge River, we will conduct a semi-quantitative evaluation of the probable cumulative impacts on Colby Lake and Whitewater Reservoir. This assessment will not be as rigorous as the Partridge River assessment since cumulative impacts on water levels and lake outflow are expected to be well within the range of historic conditions.

Colby Lake and Whitewater Reservoir served as a water source for the former Erie Mining Company and LTV Steel operations, from 1950 through May, 2000. Polymet is expected to appropriate less water from Colby Lake, with less impact on water levels and lake outflow than occurred under the previous mining operations. A minimum Colby Lake water level of 1439 ft msl was set by provision in the company's water appropriation permit, 49-0135. Minnesota Power presently holds this permit, with the same provision.

Proposed Polymet water withdrawal from Colby Lake and impacts on water levels will be evaluated and compared with historic effects from Erie and LTV Steel. The recent record of lake levels and outflow for Colby Lake will also be summarized for comparison to existing conditions. The effect of the project on the lake in view of the operating plan and recent experience will be evaluated. Potential changes in either the proposed project or the operating plan for the Colby Lake and Whitewater Reservoir outlet will be suggested as appropriate. The need for additional hydrologic data to monitor cumulative effects on Colby Lake and Whitewater Reservoir will be discussed in the report.

4.0 Investigation Report and Schedule

The results of the streamflow and lake level analyses will be summarized and incorporated into the Cumulative Streamflow and Lake Level Impacts Report. The report will describe the methodology and results from the hydrologic and hydraulic modeling, calibration procedure and results, and long-term flow simulations; the predicted impacts to future flows; the resulting impacts to the Partridge River, Colby Lake, and Whitewater Reservoir; and conclusions and recommendations. Documentation supporting the analyses and results will be included in tables, figures, and appendices, as appropriate.

The report will list the assumptions made and the modeling methods will be explained.

The anticipated schedule is to start the analyses immediately and conduct the calibration by July 6th. Modeling information from other tasks will be incorporated into the model after they are completed. Long-term simulations will be conducted approximately 10 weeks after receipt of the Stockpile Design Report (RS49). The target date for submittal of the draft report is 16 weeks after receipt of the Final Mine Site Plan (RS17).

Streamflow and Lake Level Changes: Cumulative Impacts Work Plan for the PolyMet NorthMet Mine Site

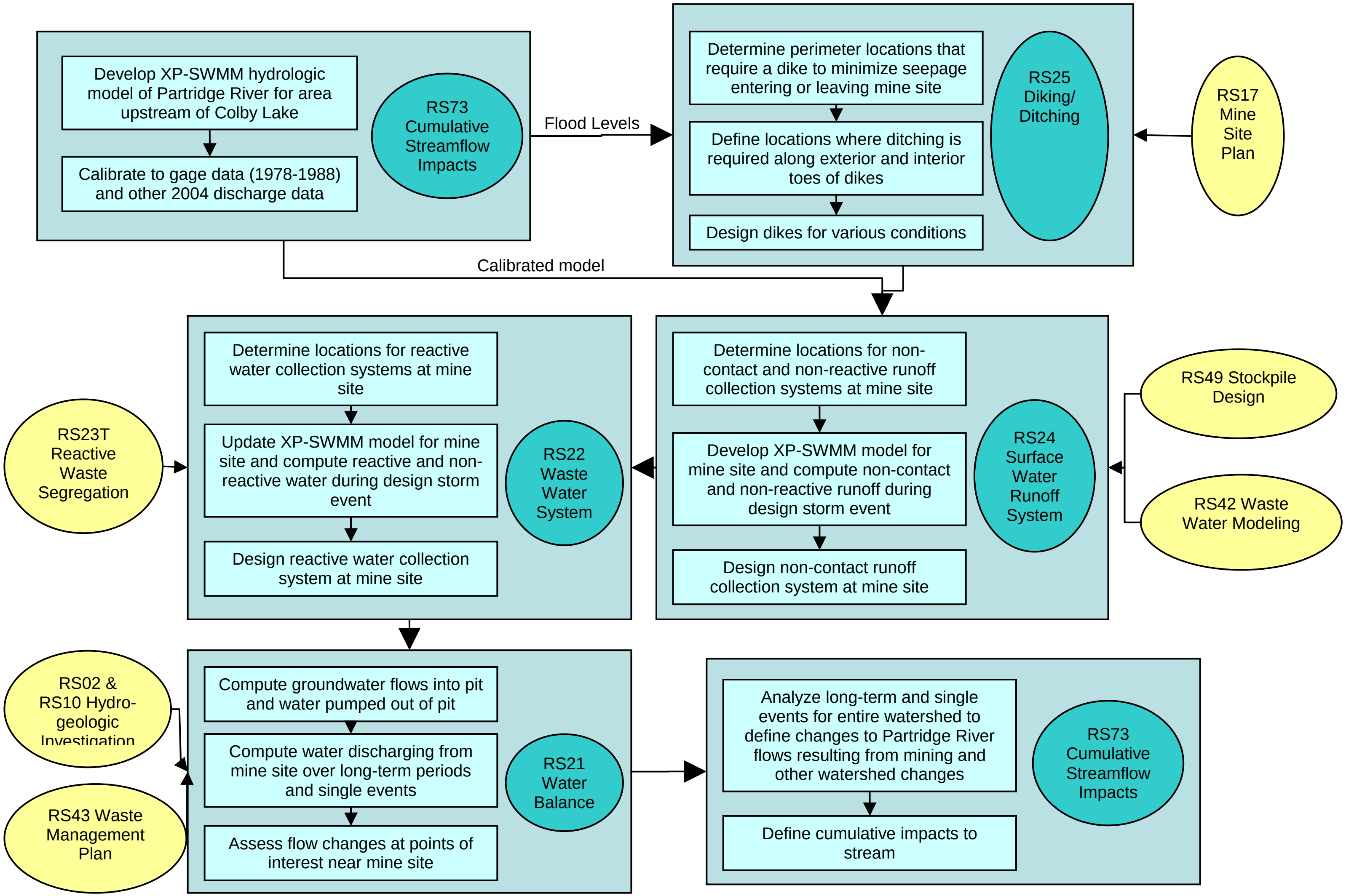
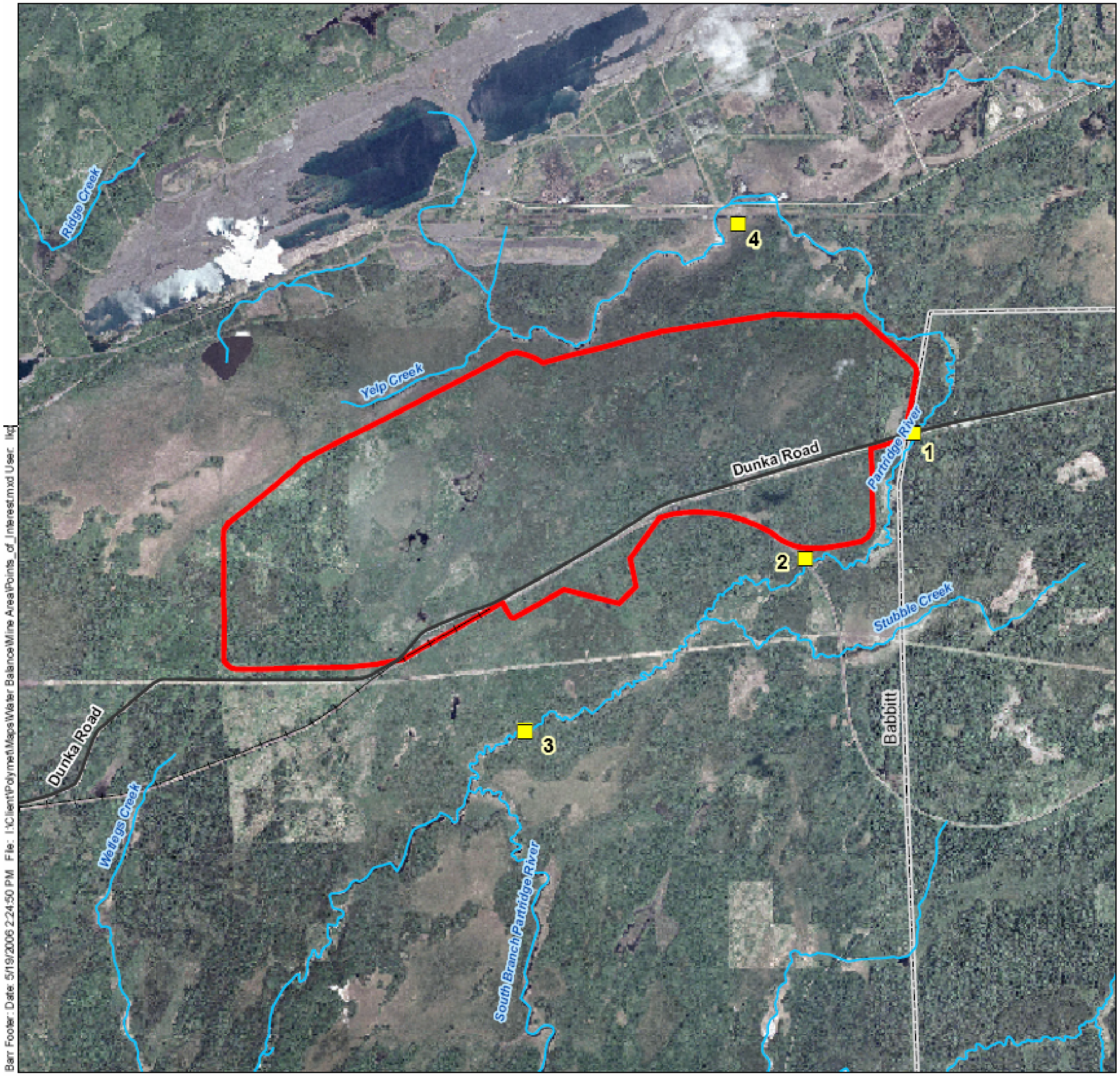


Figure 1
MINE SITE WATER MANAGEMENT

NOTES: This flow chart provides a general idea of the various tasks, the work plans provide more details. Predecessor tasks ○ are only listed at the first occurrence. Closure and reclamation will also be evaluated.



LEGEND

- 🟨 Point of Interest
- 🟥 Project Areas
- — USGS Streams
- ─┼─ Railroads
- ▬ Roads
- ☐ Municipalities

Figure 2
POINTS OF INTEREST
LOCATIONS

Barr Footer: Date: 5/19/2006 2:24:50 PM File: I:\Client\Polymet\Maps\Water Balance\Mine Area\Points_of_Interest.mxd User: lkp