Mine Waste Water Management for the PolyMet NorthMet Mine Site

RS22 Technical Detail Report
Draft-02

PolyMet Mining, Inc.

October 17, 2007

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Table of Contents

Executive Summary ... 1

1.0 Introduction ... 6
1.1 Mine Site Water Management ... 6
1.2 Preceding, Simultaneous, and Future Reports ... 7
1.3 Objectives ... 8
1.4 Report Outline ... 10

2.0 Process Water Infrastructure Design Criteria ... 11
2.1 Ditches ... 11
2.2 Ponds ... 12
2.3 Sumps ... 13
2.4 Pumps and Pipes ... 15
2.5 Overflow Contingencies ... 16

3.0 Mine Pit Process Water ... 17
3.1 Mine Pit Inflows ... 17
3.1.1 Precipitation and Runoff ... 17
3.1.2 Groundwater ... 18
3.2 Mine Pit Dewatering ... 19
3.3 East and Central Pit Filling ... 21

4.0 Waste Rock Stockpile Yield ... 23
4.1 Stockpile Water Balance ... 23
4.1.1 Yields from Open Liner ... 25
4.1.2 Total Yield from Active Stockpiles ... 25
4.1.3 Total Yield from Reclaimed Stockpiles ... 34
4.1.4 Liner Leakage and Foundation Underdrain System ... 40
4.1.5 Stockpile Sump Design ... 42
4.2 Construction Runoff Collection ... 47
4.2.1 Construction Runoff Collection Overflow Contingencies ... 48

5.0 Ore Handling Area Yield ... 50
5.1 Lean Ore Surge Pile ... 50
5.2 Rail Transfer Hopper ... 51
5.3 Haul Roads ... 52
5.4 Process Water Pond and Sump Design ... 53
5.4.1 Process Water Ponds ... 54
5.4.2 Lean Ore Surge Pile Sumps ... 54
5.4.3 Ore Handling Area Pond and Sump Overflow Contingencies ... 55

6.0 Overburden Area Runoff 57
6.1 Overburden Storage and Laydown Area 57
6.2 Overburden Portion of the Category 1/2 Stockpile 58
6.3 Ditch Design 59
6.4 Process Water Pond Design 60
6.4.1 Overburden Area Pond Overflow Contingencies 61
7.0 Process Water Management Plan 62
7.1 Conveyance System Alignment 62
7.2 Conveyance System Design 64
7.2.1 Pipeline Material 64
7.2.2 Pipeline Contingencies 66
7.2.3 Individual Pipeline Designs 68
8.0 Central Pumping Station and Treated Water Pipeline to Tailings Basin 73
8.1 Central Pumping Station 73
8.2 Pipeline to Tailings Basin 77
8.2.1 Alternative Routes Considered 78
8.2.2 Pipeline Design 79
8.2.3 Pipeline Operation 83
9.0 Maintenance and Monitoring 86
10.0 References 87

List of Tables

ES-A Design Criteria for Process Water (PW) Infrastructure

Table 3.1.1-A Snowmelt Data for USGS Gage1 on Partridge River above Colby Lake

Table 3.1.1-B Average Annual and Peak Annual Inflow Rates in Pits

Table 3.1.1-C Rainfall for Given Recurrence Intervals of 24-Hour Storms (Huff and Angel, 1992)

Table 3.2-A Preliminary Pit Sump Specifications

Table 3.2-B Preliminary Pump and Pipe Specifications for Pit Dewatering

Table 3.3-A Water Balance for East and Central Pit Filling

Table 4.1-A Comparison of Stockpile Parameters

Table 4.1-B Annual Stockpile Process Water Volumes and Flow Rates[1]

Table 4.1-C Uncovered Stockpile Storm Event Yields: Surface Runoff plus Liner Drainage

Table 4.1.2.2-A Uncovered Stockpile Annual Yield Percentages: Surface Runoff plus Liner Drainage

Table 4.1.3-A Evapotranspirative Cover Stockpile Yield Percentages

Table 4.1.3-B Membrane Cover Stockpile Yield Percentages

Table 4.1.5-A Preliminary Waste Rock Stockpile Sump Specifications

Table 4.1.5-B Preliminary Stockpile Overflow Pond Specifications

Table 4.1.5.1-A Comparison of Preliminary Sump Requirements for the Category 1/2 Stockpile[1]

Table 4.2-A Stockpile Construction Area Surface Runoff

Table 5.4-A Preliminary Ore Handling Area Pond and Sump Specifications

Table 6.4-A Preliminary Overburden Area Pond Specifications

Table 7.2.3.1-A Preliminary Pump and Pipe Specifications for Category 3/4 Construction Area Surface Runoff and Overburden Surface Runoff (Pipeline 1)

Table 7.2.3.3-A Preliminary Pump and Pipe Specifications for Category 3/4 Process Water and Rail Transfer Hopper Surface Runoff (Pipeline 3)

Table 7.2.3.5-A Preliminary Pump and Pipe Specifications for Category 1/2 Stockpile Process Water (Pipeline 5)

Table 7.2.3.6-A Preliminary Pump and Pipe Specifications for Category 1/2 Construction Area Surface Runoff (Pipeline 6)

List of Figures

Figure 1.1-A	Year 1 Process Water Management
Figure 1.1-B	Year 5 Process Water Management
Figure 1.1-C	Year 10 Process Water Management
Figure 1.1-D	Year 15 Process Water Management
Figure 1.1-E	Year 20 Process Water Management
Figure 1.2-A	Mine Site Water Management Studies
Figure 3.1.1-A	Mine Pit Inflows
Figure 3.2-A	Pit Dewatering Typical Cross Section
Figure 3.3-A	East And Central Pit Filling
Figure 4.1-A	Active Stockpile Water Balance
Figure 4.1-B	Reclaimed Stockpile Water Balance
Figure 4.1.3.3-A	U.S. Forest Service Soils Map
Figure 4.1.4.2-A	Typical Foundation Underdrain (from RS49)
Figure 4.1.4.2-B	Plan View of Underdrain/Sump Interaction
Figure 4.1.5.1-A	Conceptual Category 1/2 Sump Layout
Figure 4.1.5.1-B	Category 1/2 Material Drainage Sump Liner (Drawing 12 of RS49)
Figure 4.1.5.2-A	Generalized Stockpile Liner Configuration (Figure 2 from RS23T)
Figure 4.1.5.2-B	Conceptual Category 3 & 4 Sump Layout
Figure 4.1.5.2-C	Category 3/4 Material Drainage Sump Liner (Drawing 12 of RS49)
Figure 5.0-A	Ore Handling Area Yield Volumes
Figure 5.2-A	Rail Transfer Hopper
Figure 6.1-A	Overburden Storage Annual Process Water Surface Runoff Volume
Figure 6.2-A	Overburden Portion of the Category 1/2 Stockpile Annual Runoff
Figure 7.1-A	Year 1 Process Water Pump Systems
Figure 7.1-B	Year 5 Process Water Pump Systems
Figure 7.1-C	Year 10 Process Water Pump Systems
Figure 7.1-D	Year 15 Process Water Pump Systems
Figure 7.1-E	Year 20 Process Water Pump Systems
Figure 7.1-F	Process Water Components
Figure 7.1-G	Water to be Treated
Figure 7.1-H	Cross Section Along Dunka Road
Figure 7.2-A	Average Annual Flows for the Six Pipelines within the Mine Site
Figure 8.1-A	Preliminary Layout of the WWTF and CPS Site
Figure 8.1-B	Process Flow Diagram of the WWTF
Figure 8.1-C	Schematic of the Central Pumping Station
Figure 8.2-A	Pipeline Alignment
Figure 8.2.1-A	Alternative Pipeline Alignments

Figure 8.2.2.4-A Typical Road/Pipeline Cross Section – Flat Topography
Figure 8.2.2.4-B Typical Road/Pipeline Cross Section – High Ground to the North
Figure 8.2.2.4-C Typical Road/Pipeline Cross Section – Low Areas
Figure 8.2.2.4-D Typical Road/Pipeline Cross Section – Road Access Required

List of Appendices

Appendix A RS22 Work Plan

Appendix B Groundwater Modeling

Appendix C U.S. Forest Service Soil Map Descriptions (Draft)

List of RS Documents Referenced

RS02 Hydrogeological Investigation – Drill Hole Monitoring and Data Collection – Phase 1

RS18 Mine Plan

RS21 Mine Water Balance

RS23T Reactive Waste Rock, Lean Ore, and Deferred Ore Segregation

RS24 Mine Surface Water Runoff

RS25 Mine Diking/Ditching Effectiveness Study

RS49 Stockpile Design

RS52 Closure Plan

RS73 Cumulative Streamflow Impacts. This was broken into two documents:

- RS73A Streamflow and Lake Level Changes: Model Calibration Report
- RS73B Streamflow and Lake Level Changes: Model Results

Executive Summary

This report provides details on estimation of the quantity of process water at the Mine Site, the preliminary design of systems for collection and conveyance of process water within the Mine Site, and preliminary design of a pump station and pipeline for the conveyance of water from the Mine Site to the Tailings Basin. The Mine Site water management system will be developed incrementally throughout the life of the mine as water management is required. The overall system capacity for each type of structure will be based on the year that the pit and stockpile configurations contribute the maximum quantity of process water (Critical Year). Individual segments of the water management system will be added when needed. This document addresses process water management at the Mine Site, and is part of a series of RS documents addressing overall water management at the Mine Site. RS52 addresses process water in closure of the Mine Site.

Criteria developed for the design of process water infrastructure are listed in Table ES-A and described in more detail in this document.

Process water includes runoff and groundwater that has contacted disturbed surfaces and may not meet water quality limits. The components of process water are shown in Figure ES-A and include the following:

- Surface runoff and liner drainage from active waste rock, lean ore, and overburden stockpiles;
- Liner drainage from reclaimed waste rock stockpiles;
- Water from dewatering the mine pits; and
- Surface runoff from the Rail Transfer Hopper, Overburden Storage and Laydown Area, haul roads, and areas cleared for stockpile construction.

The overall quantity of process water at the Mine Site is directly related to the amount of area actively mined, filled, or being used that is not yet reclaimed. Figure ES-B shows the annual volume of process water produced at the Mine Site by Mine Year, based on the configuration of stockpiles and pits in the five-year design increments from the Mine Plan (RS18). Annual volumes are estimated to vary between 1,211 acre-feet (Year 1) and 3,124 acre-feet (Year 15). Annual peak flows occur during spring snowmelt and range between 1,940 gallons per minute (gpm) (Year 1) and 3,392 gpm (Year 10). The pump and pipe networks at the Mine Site, Wastewater Treatment Facility (WWTF), and Central Pumping Station (CPS) were designed based on the high estimate of peak annual volumes during the snowmelt event. Water quality predictions were based on the low estimate of annual volumes, which predict higher concentrations of constituents. Calculation of make-up water demand for the Plant Site also used the low estimate of annual volumes of process water. Determination of the zero discharge aspect of the project used the peak annual volumes during the snowmelt event and storm events.

ES-A Design Criteria for Process Water (PW) Infrastructure

Structure	Objectives	Design Criteria and Assumptions[A]
Ditches: Overburden Runoff	Convey overburden process water runoff by gravity to process water ponds	1. Sized for runoff from 10-yr 24-hr event.
Ditches: Haul Road Runoff	Convey haul road runoff by gravity to process water ponds	1. Sized for runoff from 100-yr 24-hr event.
Ditches: Category 1/2 Stockpile Sump Overflow	Convey Category 1/2 sump overflow by gravity to the West Pit for collection	1. Perimeter dikes to contain yield from 100-yr 24-hr event. 2. Ditches and dikes lined the same as the Category 1/2 sumps.
Ponds: Overburden Runoff	Provide flood storage and reduce TSS	1. Ponds to contain runoff from 25-yr 24-hr event.
Ponds: Haul Road Runoff	Provide flood storage and reduce TSS	1. Ponds to contain runoff from 100-yr 24-hr event.
Ponds: Rail Transfer Hopper Runoff	Provide flood storage and reduce TSS	1. Ponds to contain runoff from 100-yr 24-hr event.
Sumps: Category 1/2 Stockpile	Provide storage for collection and conveyance	1. Contain yield from 0.9 inch rainfall by gravity. 2. Perimeter dikes to route overflow to West Pit in lined ditches. 3. Pump capacity based on average annual 30-day spring snowmelt flows (average annual peak runoff volume). 4. Sump lining with a single composite system designed in RS49. 5. Runoff coefficients based on SCS curve numbers.
Sumps: Category 3 and 4 Waste Rock Stockpiles and Lean Ore Surge Pile	Provide flood storage in sumps or on stockpile foundation liner for collection and conveyance	1. Contain yield from 10-yr 24-hr event by gravity. 2. Perimeter dikes to convey 100-yr 24-hr runoff through a lined ditch to an overflow pond or to contain this volume on drainage layer over the liner less than 1 foot deep, pumped out within 2 weeks. 3. Pump capacity based on average annual 30-day spring snowmelt flows (average annual peak runoff volume). 4. Sump lining with a double composite system designed in RS49. 5. Runoff coefficients based on SCS curve numbers. 6. Increased outflow coefficients for Lean Ore Surge Pile, due to potential for an open liner.
Sumps: Mine Pit	Provide storage of runoff and groundwater inflows for collection and conveyance	1. Contain runoff from average annual 30-day spring snowmelt. 2. Lowest pit level to act as sump overflow. 3. Temporary in nature and continually moved with mine progression.
Pumps and Pipes	Convey process water to Wastewater Treatment Facility or Central Pumping Station	1. Pump rate based on average annual spring snowmelt yields. 2. Pipes designed to maintain velocities less than 5 feet per second.
Overflow Contingencies	Temporary systems to manage water greater than design capacities of infrastructure	1. Contain process water during events greater than design storm, if possible. 2. Include contingencies for power outage during such events.

[A]Design Criteria were applied to the Critical Year, the year with the most total yield (runoff and drainage) for that structure for adequate sizing. Additional description of rationale for selection of design criteria is given in the appropriate section in this document.

TSS: Total Suspended Solids. yr: year. hr: hour.

Figure ES-A Process Water Components

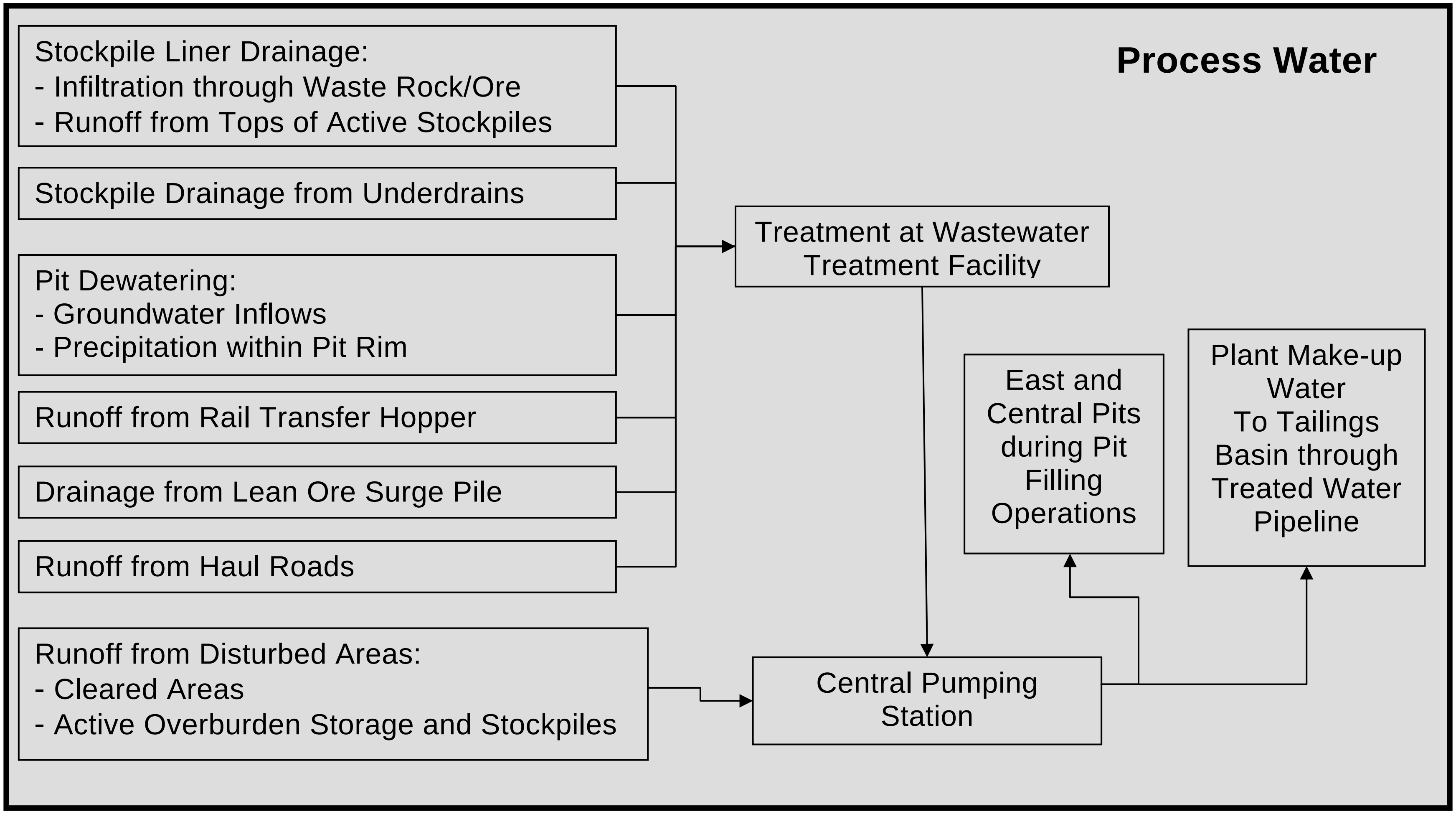
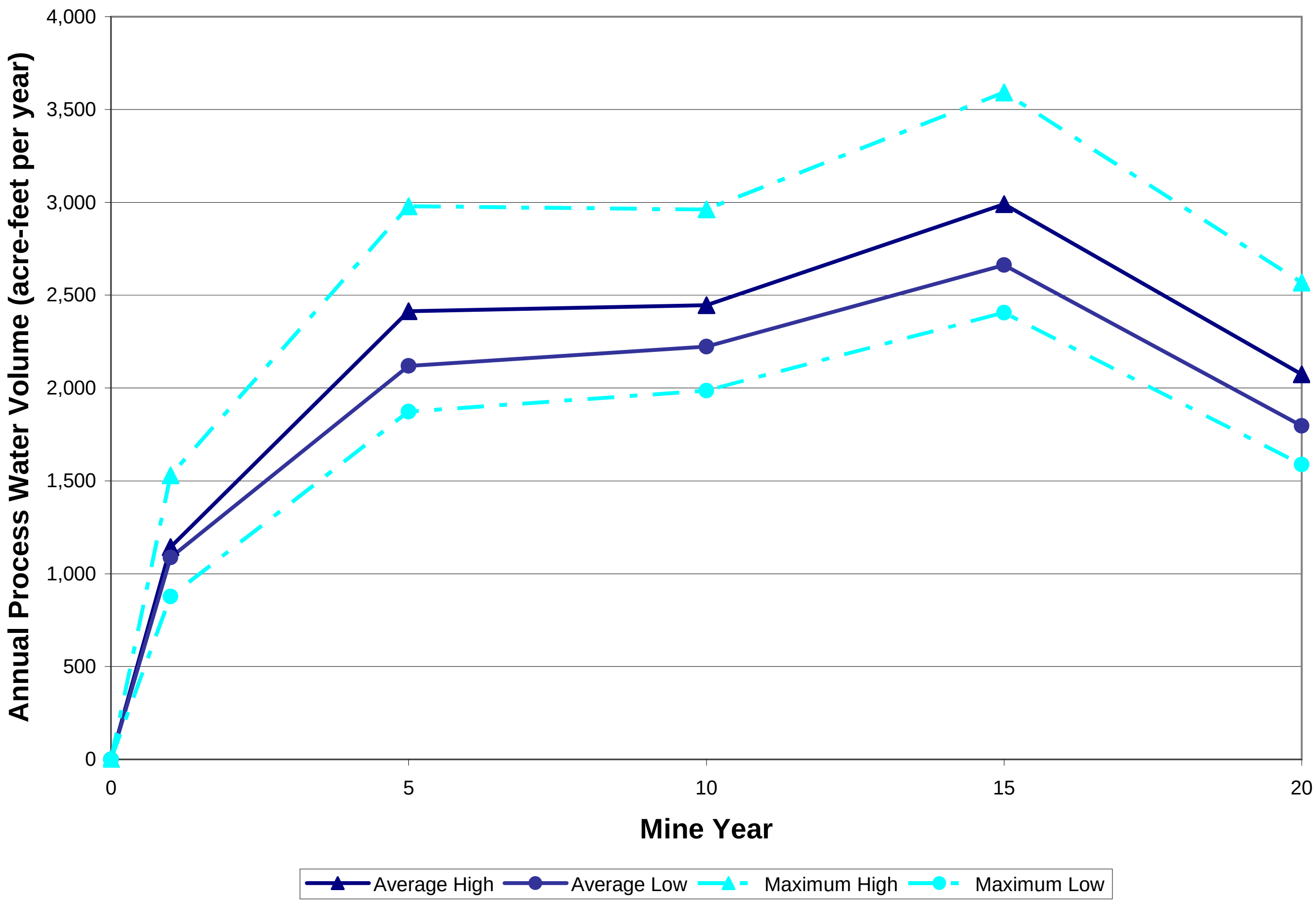


Figure ES-B Annual Volume of Process Water Produced at the Mine Site



Note: These estimates were derived from 30 years of precipitation records for the Mine Site: average precipitation was 28.2 inches, wettest was 41.8 inches (maximum high), and driest was 20.3 inches (maximum low). The average high and low estimates represent the range of annual volumes expected from average precipitation; the difference between the high and low estimate depicts the range of stockpile drainage estimates (see Section 4.0).

1.0 Introduction

PolyMet Mining, Inc. (PolyMet) is in the process of environmental review for its NorthMet project, located near Babbitt, 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 for the proposed project. This document summarizes the preliminary design of process (waste) water management systems at the Mine Site.

1.1 Mine Site Water Management

There are two major types of runoff that will occur on the Mine Site, process water (wastewater) and stormwater (non-contact water). Precipitation that falls on natural or reclaimed surfaces and is expected to meet water quality limits after sedimentation ponds remove suspended solids is referred to as stormwater. Process water includes runoff[1] and groundwater that has contacted disturbed surfaces and may not meet water quality limits. This includes runoff from cleared construction areas, drainage from unreclaimed stockpiles, liner drainage from reclaimed stockpiles, and pit water. Process water may require treatment for removal of metals or other substances at the Waste Water Treatment Facility (WWTF) prior to being routed through the Treated Water Pipeline to the Tailings Basin for use as plant make-up water or for pit filling in later years. Figures 1.1-A through 1.1-E show the process water management systems at the end of Mine Years 1, 5, 10, 15, and 20, respectively.

Stormwater runoff from undisturbed (natural) areas will be diverted away from process areas (stockpiles, pits, haul roads, Rail Transfer Hopper, Lean Ore Surge Pile, etc.) using a combination of dikes and ditches that will route stormwater into sedimentation ponds, then outlet to the Partridge River (discussed in RS24).

Minnesota mining rules for nonferrous metallic mineral mining require water that comes in contact with "reactive mine waste" to be collected to "prevent the release of substances that result in adverse impacts on natural resources." Potentially reactive mine waste (i.e. waste rock) at NorthMet will be

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[1] Runoff is defined in this report as the total volume of stormwater or process water that collects above ground from the surface. According to this definition, the runoff from active stockpiles is process water and the runoff from reclaimed stockpiles is stormwater. Runoff from active stockpiles includes the total yield from surface runoff, liner drainage, and leakage through the liner (see Section 4). Runoff from reclaimed stockpiles includes flows from the top of the cover and interflow that infiltrates into the cover and exits the stockpile without contacting the waste rock (see RS24).

stockpiled in four waste rock stockpiles and one surge pile at the Mine Site, as described in RS18. These stockpiles have been designed with a foundation and liner system to direct process water by gravity to a series of collection sumps, which will be pumped through a pipe network to the WWTF for treatment, if necessary, and then to the Central Pumping Station (CPS). Once stockpiles are reclaimed, the runoff from the tops and sides of the stockpiles will be classified as stormwater and will be routed to sedimentation ponds through a system of ditches prior to being routed to local receiving waters. Process water will continue to be collected off the foundation and liner systems after stockpile reclamation. All groundwater flows to mine pits and precipitation that falls within the mine pits will be collected in sumps and routed to the WWTF and then to the CPS after treatment. Inflows to the pits will be minimized by dikes that manage surface water flow and shallow groundwater flow into the pits. Process water will also be collected from unreclaimed portions of the overburden portion of the Category 1/2 stockpile and the Overburden Storage and Laydown Area, the Rail Transfer Hopper, the Lean Ore Surge Pile, haul roads, and construction areas and routed to the CPS after treatment, if required.

All process water system components have been designed to route process water by gravity flows to sumps or process water ponds that are designed to contain water from a component specific "design event". The design event chosen for each component was based on the expected quality of water handled by the component and the overflow potential of the component. This allowed matching the level of protection applied to the component to the water quality handled by the component and the potential for overflows by choosing larger "design events". Water from the sumps and process water ponds will be conveyed to the WWTF, if needed, and then to the CPS.

1.2 Preceding, Simultaneous, and Future Reports

Mine site water management has been evaluated in five separate studies. Figure 1.2-A illustrates the interaction between these studies and includes the following reports:

- Mine Surface Water Runoff (RS24) evaluates the collection and routing of stormwater from the Mine Site.
- The Mine Diking/Ditching Effectiveness Study (RS25) evaluates the diking and ditching system around the perimeter of the Mine Site and the dikes around the pits that will be constructed to minimize surface water flow and shallow groundwater flow in the pits.
- The Mine Water Balance (RS21) incorporates data from these three studies (RS22, RS24, and RS25) into the overall water balance at the Mine Site.

- The Cumulative Streamflow Impacts (RS73) evaluates the cumulative effects of the Mine Site development on the quantity of Partridge River and other downstream river and lake flows.

Mine design from other reports was also used and incorporated into the design of process water management systems, including the Phase 1 Hydrogeological Investigation (RS02), the Mine Plan (RS18), the Stockpile Design Report (RS49), Reactive Waste Rock, Lean Ore, and Deferred Ore Segregation (RS23T), which evaluated stockpile liner and cover designs. The Closure Plan (RS52) evaluates process water management needs during and following closure of the Mine.

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.

1.3 Objectives

The objectives of this report are based on the May 2, 2006 Work Plan that was developed in discussions with the agencies. Appendix A is a copy of this Work Plan.

Process water from the Mine Site will be collected in a system that is separate from the stormwater collection system, and will be conveyed to the Plant Site for makeup water or conveyed the East and Central Pits for filling following any treatment that is necessary. This report will provide a conceptual design for the ditches, pipes, and ponds that convey this process water throughout the Mine Site.

The primary objective of this report is to ensure that all process water that does not meet water quality limits is collected for treatment and reuse. The report must also provide volume estimates to predict water quality concentrations and to design the WWTF and CPS.

The collection system must perform so that water that escapes the collection system does not create adverse impacts to surface water or groundwater quality. The design of monitoring systems to ensure that this performance objective is met is briefly addressed in this report; a preliminary monitoring plan was provided in the PolyMet Project Description and will be finalized as part of the permitting process.

This system will be designed to accommodate all potential sources of process water, based on available data. Conservative assumptions have been made due to uncertainty of the amount and nature of the reactive material. The design also considers the critical temporal phase for process water volumes, depending on whether the stockpiles are active, covered (reclaimed), or partially covered (reclaimed).

The objectives of this report have changed from those listed in the May 2, 2006 Work Plan that was developed based on discussions with the agencies:

- The Work Plan was developed assuming that the facility would increase discharges to the Partridge River, which would require detailed information on the magnitude of the increased flows to define the potential impacts. Therefore detailed modeling of the Mine Site was proposed. The NorthMet project is now proposing a reuse/recycle strategy, with no process water discharge proposed to surface waters of the State. Stormwater from the entire Mine Site will continue to be routed to the Partridge River during mining operations (see Section 1.1 for the difference between process water and stormwater). Therefore, flows in the Partridge River are expected to stay the same or decrease slightly because a portion of the Mine Site runoff will become process water that is sent to the Plant Site. Therefore, detailed hydrologic analysis of the Mine Site water management systems is no longer warranted for this analysis.

- The Work Plan did not include groundwater modeling as part of RS22. Changes in mine operations have progressed to include backfilling of the East and Central Pits as part of mine development. Therefore, groundwater modeling has become an integral part of the process water management system and is included in this report.

- The Work Plan assumed that design would be based on the critical mine year, the year that produced the largest volume of process water, and that other years would not be evaluated. The critical mine year was used for design of the process water management system. However, the flows for mine years 1, 5, 10, 15, and 20 were also evaluated to define the volume of flows that require treatment and to determine which year would be the critical year.

Based on these revisions, the objective of this document was revised to define the overall quantity of process water that will be produced and to manage that quantity according to the objectives listed above.

1.4 Report Outline

This report is organized by the particular area of the Mine Site where process water is managed, as follows:

1) Design Criteria (Section 2), including
    a. Ditches,
    b. Ponds,
    c. Sumps,
    d. Pumps and Pipes, and
    e. Overflow Contingencies.

2) Mine Pit Process Water (Section 3), including
    a. Mine Pit inflows and dewatering, and
    b. East and Central Pit filling.

3) Waste Rock Stockpile Drainage (Section 4), including
    a. The stockpile water balance, and
    b. Stockpile construction surface runoff collection.

4) Ore Handling Area Drainage (Section 5), including
    a. Lean Ore Surge Pile drainage,
    b. Rail Transfer Hopper runoff, and
    c. Haul Road runoff.

5) Overburden Area Runoff (Section 6), including
    a. Overburden Storage and Laydown Area runoff, and
    b. Overburden portion of the Category 1/2 Stockpile runoff.

6) Process Water Management Plan (Section 7), including
    a. Conveyance system alignments, design, and overflow contingencies.

7) Central Pumping Station (CPS) and Treated Water Pipeline to Tailings Basin (Section 8),

8) Maintenance and Monitoring (Section 9), and

9) References (Section 10).

Tables and Figures are located after the report text.

2.0 Process Water Infrastructure Design Criteria

This section only provides the design parameters supporting the process water infrastructure design throughout this report. Design parameters are provided for ditches, ponds, sumps, pumps and pipes, and overflow contingencies. This section was developed as an overall summary of the design criteria and does not include any of the details of the designs; specific details about these designs and how these criteria were developed are provided in later sections.

2.1 Ditches

There are three categories of process water ditches that convey gravity flows: overburden runoff, haul road runoff, and Category 1/2 stockpile sump overflow. These ditches collect surface runoff from active (uncovered) portions of the overburden areas and haul roads and any overflow from the Category 1/2 stockpile sumps. The following design criteria were used in ditch design:

Critical year: The design was based on a critical year, which is the Mine Year producing the highest quantity of process water for each ditch network.

Design event: 10-year, 24-hour event for overburden runoff and 100-year, 24-hour event for haul road runoff and Category 1/2 stockpile sump overflow.

Design storm precipitation: 3.36 inches for the 10-year, 24-hour event and 5.2 inches for the 100-year, 24-hour event

Design storm yield[2]:

- Overburden - 61 percent of 10-year, 24-hour precipitation based on the design event and surface runoff properties similar to compacted dirt roads with Type C soils (SCS Curve Number [CN] = 87).

[2] Yield is the total volume of surface runoff and drainage from the infrastructure. For infrastructure without a liner (i.e., overburden, haul roads, rail transfer hopper, etc.), this only includes surface runoff, but for infrastructure with a liner (i.e., waste rock and lean ore stockpiles and the ore surge pile), this includes any drainage collected on the liner or leakage collected from the underdrains. Details for each infrastructure are included later in the report.

- Haul road - 76 percent of 100-year, 24-hour precipitation based on the design event and gravel roads with Type C soils (CN = 89),
- Category 1/2 sump overflow – Conveyance of the volume in excess of a 0.9-inch rainstorm (collected in the sump), up to the volume from the 100-year, 24-hour storm event. See stockpile sump design criteria below for a detailed description of the runoff calculation for the stockpile.

Cross sectional area: Trapezoidal channel with 3 (horizontal):1 (vertical) side slopes and 1 foot of freeboard.

Liner system: None for overburden or haul road runoff ditches. Category 1/2 sump liners (see Section 2.3) will be used in the Category 1/2 sump overflow ditches.

2.2 Ponds

There are four types of process water ponds for collection of flood storage: overburden runoff ponds PW-1 and PW-7, haul road runoff ponds PW-2 and PW-4, Rail Transfer Hopper runoff pond PW-3, stockpile overflow ponds PW-5 and PW-6. Ponds PW-1 through PW-4 and PW-7 collect surface runoff by gravity from active (uncovered) portions of the overburden areas, haul roads, and the Rail Transfer Hopper. Ponds PW-5 and PW-6 collect overflow by gravity in excess of the sump storage from the Category 3 Lean Ore and Category 4 stockpiles (PW-5) and the Category 3 stockpile (PW-6). All process water ponds are designed to contain flood storage by gravity in the event of a power outage during a storm event. Excess process water from sump storage at the Category 1/2 stockpile will overflow by gravity into the West Pit, so no overflow pond is proposed for the process water from that stockpile. The following design criteria were used in pond design:

Critical year: The design was based on a critical year, which is the year producing the highest quantity of process water for each pond.

Design event: 25-year, 24-hour event for overburden runoff (PW-1 and PW-7) and 100-year, 24-hour event for haul road runoff (PW-2 and PW-4), Rail Transfer Hopper runoff (PW-3), and stockpile sump overflow (PW-5 and PW-6)

Design storm precipitation: 4.1 inches for the 25-year, 24-hour event and 5.2 inches for the 100-year, 24-hour event

Design storm yield:

- Overburden runoff - 66 percent of 25-year, 24-hour precipitation based on the design event and characteristics similar to compacted dirt roads from Type C soils due to the mixture of rock and soils (CN = 87). Surface runoff within the pond boundary was calculated as 95 percent of precipitation based on wetlands (CN = 98).
- Haul road runoff - 76 percent of 100-year, 24-hour precipitation based on the design event and gravel roads with Type C soils (CN = 89). Surface runoff within the pond boundary was calculated as 95 percent of precipitation based on wetlands (CN = 98).
- Rail Transfer Hopper - 95 percent of 100-year, 24-hour precipitation based on the design event and paved surfaces (CN = 98). Surface runoff within the pond boundary was calculated as 95 percent of precipitation based on wetlands (CN = 98).
- Category 3 and 4 stockpile sump overflow **-** The volume in excess of the 10-year, 24-hour storm event (collected in the sump), up to the 100-year, 24-hour storm event was calculated for overflow pond storage. Surface runoff from within the overflow pond boundary was calculated as 95 percent of precipitation based on wetlands (CN=98). See stockpile sump design criteria for a detailed description of the runoff calculation from the stockpiles.

Liner system: No liner will be used for the overburden ponds PW-1 and PW-7 (see Section 6.4). The Category 1/2 sump liners (see Section 2.3) will be used to line the haul road ponds, PW-2 and PW-4, and the stockpile sump overflow ponds PW-5 and PW-6. The Category 3/4 sump liner design (see Section 2.3) will be used for the Rail Transfer Hopper pond PW-3.

2.3 Sumps

There are four different types of sumps for collection of process water: Category 1/2 waste rock stockpile sumps, Category 3 and 4 waste rock stockpile sumps, Lean Ore Surge Pile sumps, and mine pit sumps. These sumps collect liner drainage from active (uncovered) and reclaimed portions of the stockpiles and Lean Ore Surge Pile, and surface runoff from active (uncovered) portions of the waste rock stockpiles, Lean Ore Surge Pile, and mine pits. The following design criteria were used in sump design (see Section 4 for details regarding stockpile runoff and Section 3 for pit runoff):

Critical year: The design was based on a critical year, which is the year producing the highest quantity of process water for each sump.

Design event:

- Category 1/2 stockpile sumps - sump design was based on collection and conveyance of the stockpile yield from a typical 0.9 inch rainfall, with overflows routed by gravity in lined ditches to the West Pit. Surface runoff within the sump boundary was calculated as 100 percent of precipitation.
- Category 3 and 4 stockpile sumps - sump design was based on collection and conveyance of the stockpile yield from a 10-year, 24-hour storm event. Flows greater than the 10-year event will be routed by gravity to an overflow pond (see Section 2.2) or stored on the stockpile foundation liner. Yields in excess of the 100-year, 24-hour event would temporarily be pumped to the East Pit or conveyed to the Wastewater Treatment Facility (WWTF) according to an emergency operating procedure as described in Section 4.1.5. Surface runoff within the sump boundary was calculated as 100 percent of precipitation.
- Lean Ore Surge Pile sumps - sump design was based on collection of the stockpile yield from a 10-year, 24-hour storm event. Flows greater than the 10-year event will be collected on the stockpile foundation liner up to the 100-year, 24-hour storm event with a maximum depth of one foot on the liner for the 100-year event. Stockpile yield in excess of the 100-year, 24-hour event would be pumped or conveyed to the East Pit or the WWTF. Surface runoff from within the sump boundary was calculated as 100 percent of precipitation.
- Mine pit sumps - sump design was based on collection of 0.5 inches of runoff, which equals about 15 percent of the average annual 30-day spring snowmelt runoff of 3.2 inches. Excess runoff would be collected in the lowest level of the pit. Surface runoff from within the sump boundary was calculated as 100 percent of precipitation.

Design storm precipitation: 3.36 inches for the 10-year, 24-hour event, 5.2 inches for the 100-year, 24-hour event, and 3.2 inches of runoff from the spring snowmelt event (based on 1978-1988 gage data)

Design storm yield:

- Category 1/2 stockpile sumps – Total yield of 25 percent of 0.9 inch precipitation for active (uncovered) portions of stockpiles. This is calculated with surface runoff characteristics similar to gravel roads with Type C soils (CN = 89) and minimal infiltration thru the waste rock to the liner during the storm event (see Section 4.1.2.3). Reclaimed portions of the stockpile result in 2.3 percent of 0.9-inch precipitation reaching the liner based on the infiltration rate from the SCS Curve Number for Type B soils on grassland in fair condition (CN = 69) and applying an SCS Curve Number from a gravel road with Type A soil (CN = 76) to the initial infiltrated volume to compute the quantity reaching the liner.

- Category 3 and 4 stockpile sumps – Total yield of 68 percent of the design storm precipitation based on active (uncovered) portions of the stockpiles. This is calculated with surface runoff characteristics similar to gravel roads with Type C soils (CN = 89) and 8 percent of the infiltration thru the waste rock to the liner during the storm event (see Section 4.1.2.3). Reclaimed portions of the stockpiles result in 10.8 percent of precipitation reaching the liner from reclaimed portions of Category 3 and Category 3 Lean Ore Stockpiles; and 6.1 percent of precipitation reaching the liner from the reclaimed portions of the Category 4 stockpile. As mentioned above, liner drainages were based on SCS Curve Numbers for the respective cover type. Category 3 stockpile covers used the infiltration rate from a grassland in fair condition with Type C soils (CN = 79) and applied the SCS Curve Number from a gravel road with Type A soil (CN = 76) to the infiltration volume to compute the quantity reaching the liner. The Category 4 stockpile covers used the infiltration rate from a grassland in fair condition with Type D soils (CN = 84) and applied the SCS Curve Number from a gravel road with Type A soil (CN = 76) to the initial infiltrated volume to compute the quantity reaching the liner for each stockpile.
- Lean Ore Surge Pile sumps - 100 percent of precipitation based on the possibility of having exposed liners conveying all stockpile yield to the sumps.
- Mine pit sumps - 100 percent of total snowmelt runoff based on the 3.2 inches of snowmelt runoff typical for this area (based on 1978-1988 gage data).

Design depths: Category 1/2, 3, and 4 waste rock stockpile sumps and the Lean Ore Surge Pile sumps were designed with an average depth of 6 feet. The mine pit sumps were designed with an average depth of 10 feet.

Liner system: Category 1/2, 3, and 4 waste rock stockpile sump liners were designed in RS49. The Lean Ore Surge Pile sump liners will be the same as Category 3 Lean Ore and 4 sump liners, and the mine pit sumps will be unlined.

2.4 Pumps and Pipes

The pumps and pipes were designed according to the following criteria:

Pump rates: Based on average annual yield from a 30-day snowmelt runoff of 3.2 inches (based on 1978-1988 gage data from the Partridge River). Process water during snowmelt yield from waste rock stockpiles and the Lean Ore Surge Pile was based on the high estimate with a range from 4 to 110 percent of the 3.2 inches, as described in Section 4. Snowmelt yield for pit pumps was based on a rapid snowmelt, assuming 40 percent of the snowmelt happens in one day, with 3 days to remove it.

Maximum pipe velocities: 5 feet per second (fps)

Maximum pressures: 265 pounds per square inch (psi)

2.5 Overflow Contingencies

All process water infrastructure has been designed based on a design storm. The critical infrastructure, infrastructure with the most storage capacity, for storage of process water includes process water ponds and the Lean Ore Surge Pile sumps. However, during the life of the mine, there may be occasions in which an event greater than these design events occurs. Contingencies have been developed to minimize environmental impacts if an event greater than the design events occur. The overflow contingency plan will be developed according to the following criteria:

- For storm events greater than the design events, all process water that has contacted waste rock or lean ore will be contained to the extent possible (see contingencies in each Section).
- Overburden runoff and stockpile construction area runoff, which only requires treatment for TSS, will be allowed to overflow from the process water ponds into the pits or stormwater ponds.
- Sump overflow ditches will direct water by gravity from the Category 1/2 sumps to the West Pit.
- Unlined overflow ditches will direct water by gravity from the Category 3 waste rock overflow pond to the East Pit.
- Road dewatering trucks with diesel pumps will be used to pump water, prioritized in order of reactivity, from the Lean Ore Surge Pile sumps S-6 and S-7 and the process water ponds PW-5 (Category 3 Lean Ore and Category 4 overflow pond), PW-3 (Rail Transfer Hopper runoff), PW-4 (haul road runoff), and PW-2 (haul road runoff), in that order. This water will be directed to the WWTF or pits, depending on system capacity at the WWTF.
- The CPS pond will overflow by gravity to the West Pit.
- The WWTF will either shutdown systems prior to overflow (i.e., upstream pumps), overflow by gravity to the West Pit, or a combination.

3.0 Mine Pit Process Water

This section describes calculations of the quantity of mine pit inflows, the collection and conveyance of water from within the pits, and filling of the East and Central Pits between Year 12 and Year 20. The diking and ditching system designed to keep water out of the pits by minimizing lateral movement of surface water and shallow groundwater within surficial deposits is discussed briefly in this document; further details on the design and evaluation can be found in RS25. Figures 1.1-A through 1.1-E show the process water management systems, including the pump and pipe networks that dewater the pits, in Mine Years 1, 5, 10, 15, and 20, respectively. Further information on the conveyance of mine pit process water to the CPS and through the Treated Water Pipeline is discussed in Section 6.0.

3.1 Mine Pit Inflows

Inflows to the pits include direct precipitation and runoff, and groundwater.

3.1.1 Precipitation and Runoff

Dikes and ditches will be used to minimize surface runoff from surrounding areas from entering the mine pits, as described in RS25. Therefore, precipitation and runoff will be restricted to that within the pit rims. The amount of runoff accumulating within the pits during rainfall events will vary depending on the intensity and volume of the event. Small rainfall events will wet rock surfaces but may not produce any runoff. Large rainfall events will produce runoff. Snow that falls within the pit perimeter will accumulate until the temperatures increase above freezing. Orientation and shading will affect snowmelt differently on various slopes and areas within the pits.

When considering the accumulation of runoff, two separate volumes need to be considered: average annual precipitation and single storm event precipitation.

As described in RS73A, precipitation data for the Mine Site was compiled from precipitation records from 16 weather stations within a radius of approximately 30 miles from the Mine Site. This precipitation data was used to create a spatially distributed rainfall grid for the Partridge River watershed, which includes the Mine Site. Based on data from the 30-year period from October 1, 1971 through September 30, 2001, average annual precipitation for the Mine Site is approximately 28.2 inches, and annual average runoff from combined rainfall and snowmelt is about 11.3 inches. The remaining 17 inches of precipitation is lost to infiltration and evaporation. Based on discussions with staff at the Minnesota Department of Natural Resources (MDNR), runoff within mine pits is typically similar to natural runoff. This is likely due to the high retention capacity of the rock that reduces the pit runoff and the high rate of runoff from surrounding wetlands under natural watershed conditions. Therefore, 11.3 inches of annual runoff was assumed to occur during the six months where temperatures are above freezing (combining rainfall and snowmelt runoff).

The peak runoff volume from a single event in an average year typically occurs during spring snowmelt in this area. The average runoff rate during the typical 30-day spring snowmelt event was used as input to size the pumps for dewatering the pits. The historical annual average snowmelt runoff amount of 3.2 inches was defined by comparing USGS gaged flows on the Partridge River over a 10-year period between 1978-1988 with records of air temperature and precipitation from this same timeframe, as listed in Table 3.1.1-A.

Based on these runoff assumptions, the predicted average annual inflow rates and peak inflow rates for the three pits are listed in Table 3.1.1-B and illustrated in Figure 3.1.1-A.

In addition to average annual precipitation and snowmelt runoff data, rainfall statistics from various storm events for this area were obtained from the *Rainfall Frequency Atlas of the Midwest* by Huff and Angel (1992) as listed in Table 3.1.1-C.

3.1.2 Groundwater

Groundwater contributes the largest volume of inflow of water to the pits. Dikes will be used to minimize shallow groundwater flows entering the pit from the surficial deposits, as detailed in RS25. Groundwater inflows from surficial deposits, the Duluth Complex, and the Virginia Formation were predicted using the industry standard finite difference groundwater modeling code MODFLOW. A detailed description of the groundwater modeling that was completed for this project is provided in Appendix B to this report and is summarized here.

A three-dimensional model was constructed for the 100-square mile area encompassing the proposed mine pits. Data collected as part of the Phase I, Phase II, and Phase III Hydrogeologic Investigations, which provided information on the hydraulic conductivity of the Duluth Complex, the Virginia Formation and the surficial deposits, was incorporated into the model (see RS02, RS10 and RS10A). The model was calibrated to groundwater levels in both the bedrock aquifers and the surficial deposits. Several transient model realizations simulating the pits in various stages of development (i.e. Years 1, 5, 10, 15 and 20) were constructed based on the proposed mine plan. Groundwater inflow rates to the pits were predicted in each model realization. A sensitivity analysis was performed to address uncertainties in model parameters.

In general, groundwater inflows will increase during Years 1 through 11 as the mine pits expand wider and deeper. The predicted flows are listed in Table 3.1.1-B and shown in Figure 3.1.1-A. Starting in Year 12, when the East Pit begins to be backfilled, groundwater flows to the East Pit decrease as the water level in the pit rises and the inward groundwater gradients are reduced. In Year 20, the East and Central Pits are combined into one pit. This pit has a negative net groundwater inflow rate, indicating that there is a net loss of water to the groundwater system. This is mainly due to the dewatering of the West Pit, which creates a cone of depression that extends to the East and Central Pits and beyond. Additional detail regarding pit dewatering is provided in the following sections and in Appendix B.

3.2 Mine Pit Dewatering

Mine pit inflows will be directed to sumps within the pits where process water can be collected and pumped to the surface. The mine pit sump areas and pump capacities were designed to minimize delay to mining operations during the typical spring snowmelt event.

Water management within the pit will occur as part of mine development, with the pit floors sloped toward the sumps. The sumps will be excavated as part of mine operations. Pumps in the excavated sumps would either be submersible pumps, which can handle small amounts of suspended sediments and require a basin deep enough to submerge the entire unit, or floating pumps on a platform or raft above the sump. These pumping systems could include one single large pump or several smaller pumps, depending on an optimization analysis. High density polyethylene (HDPE) pipes would connect these pumps to additional pumps at the rim of the pits that would convey the water to the WWTF, as shown in Figure 3.2-A. Pipe configurations for pit dewatering are shown on Figures 1.1-A through 1.1-E, according to Plan Year. These configurations were based on future pit development, minimizing the need to move pipes to the extent possible. In locations where a pipe crosses a haul road, culverts will be used to protect the pipe from the weight of the trucks.

Several different types of hoses were also evaluated for use in pit dewatering instead of HDPE pipe. The main benefit of hoses is their flexibility; because of this their use was noted in the NorthMet Project Description for use within the pits. However, HDPE pipe is preferred over hoses, because the high friction coefficients of the hoses required significantly larger pumps to overcome the loss in power. In addition, HDPE can withstand higher pressures than the hoses, requiring fewer pumps overall. HDPE can tolerate acidic water and is preferred in this climate due to its ability to withstand freeze and thaw cycles. Further discussion of pipe material selection is included in Section 7.2.1.

Hoses may be used periodically where design allows to provide operational flexibility. For example, a short section of hose may still be used to connect pumps to HDPE pipe. If this is necessary, high pressure rubber hose would likely be used to minimize friction loss.

The size and location of the sumps and pumps will change as the pits expand in size and depth, requiring periodic upgrading of the pumps. Pump capacities are based on peak annual flows from the snowmelt event, assuming a rapid spring snowmelt (40 percent of the snowmelt occurring within one day). The pumps were designed to handle groundwater inflows and the average annual runoff volumes from a snowmelt event, removing approximately 100 percent of the groundwater inflows and 40 percent of the total snowmelt runoff (1.28 inches) within 3 days; the volume from this snowmelt event is approximately equivalent to the runoff volume expected in the pits during the 5-year 24-hour storm event. The sumps were designed with capacity to hold the remaining volume from this snowmelt runoff event plus groundwater inflows not conveyed immediately by the pumps.

In the event that a storm exceeds the sump and pump capacity, the lowest level of the pit will be used as an emergency sump, with mining operations relocated to upper levels or delayed until water levels are pumped back to sump capacity. During extreme events, pit dewatering may temporarily be stopped to minimize overflow of stockpile sumps. The preliminary pit sump sizes are listed in Table 3.2-A, according to the mine year.

The pipes associated with these pumps were sized to maintain average velocities less than 5 feet per second to minimize friction losses and surge pressures (water hammer) in the pipes. The preliminary pump and pipe specifications for pit dewatering are listed in Table 3.2-B by mine year. The pump sizes were evaluated for each mine year, because, as the pits deepen, larger pumps will be needed to overcome the change in static head.

Pumps will be further evaluated in final design to determine the best way to optimize the total number, size and operating cost of pumps needed, and whether to upgrade each Plan Year with a single large pump that can overcome all of the static and dynamic head or split the power needed into numerous small pumps. Pumps will be re-used in the changing system as much as possible.

Available power will be another factor to evaluate in the final pump size determination. The more pumps used along the conveyance system, the more locations power distribution is needed. Conversely, the larger the pump is, the less it would have to run to empty water from the sump.

The in-pit pumps range in size from 2 to 60 horsepower in Year 1 up to 600 horsepower in Year 20, due to the increasing depth of the pits. In-pit pipes range from 6 inches in diameter in Year 1 up to 18 inches in diameter in Year 20.

Pumps from the pit rim to the WWTF also increase in size over the years, due to the increased flows in the pits. These pumps range from 2 to 60 horsepower in Year 1 up to 225 horsepower in Year 15. Pipe diameters also increase from the rim of the pit to the WWTF from 6 inches in Year 1 up to 22 inches in diameter in Year 20.

3.3 East and Central Pit Filling

Mining activities will be completed in the East and Central Pits prior to completion of mining in the West Pit. Once mining activities have ceased in the East and Central Pits, Category 1/2 waste rock and water will be used to fill these pits. According to RS18, mining in the East Pit is projected to be completed in Year 11, with backfilling starting in Year 12. Mining in the Central Pit will be completed in Year 13.

The quantity of rock available to place in the East and Central Pits changes with each year of operation, depending on the quantity of Category 1/2 rock excavated during that year. During pit filling, it may be beneficial to keep the water elevation at or below the surface of the waste rock to avoid work in the water and to aid in complete filling of the pit volumes with waste rock. The desired water level was assumed to be a maximum of 5 feet below the current rock level, to minimize oxidation of the waste rock.

The total volume of Category 1/2 waste rock in Years 12 through 20 is more than what is needed to fill these pits to the desired level. Pit filling has been modeled to evaluate the available waste rock and quantity of water needed for each progressive year. Because the Category 1/2 stockpile is projected to be completely reclaimed by Year 15, the model assumes that approximately half of the total Category 1/2 waste rock from Mine Years 12, 13, and 14 and all Category 1/2 rock mined after Year 14 will be placed in these pits.

The water that is available to fill the East and Central Pits includes direct precipitation, in-pit runoff, and groundwater from within these two pits and, when needed, additional water will be pumped from the CPS. Water quality estimates in RS31 indicate that the West Pit runoff and groundwater will need to be pumped to the WWTF for treatment prior to being pumped to the East Pit. During East and Central Pit filling operations, dewatering may still be needed during periods of high precipitation to allow placement of the waste rock under optimal conditions. Table 3.3-A lists the amount of water needed each year of pit filling operations to keep the water level within 5 feet of the top of waste rock. Figure 3.3-A illustrates the percent of East and Central Pit runoff, groundwater, and rock used during pit filling operations, and the amount of additional water needed from the CPS on an annual basis.

By the end of Year 20, the East and Central Pits will be one large pit. Notice that Figure 3.3-A does not show this pit filled completely at the end of Year 20. During closure, a wetland treatment system will be constructed over this backfilled pit, with the remaining pit volume filled with overburden material, vegetation, and additional water, as described in RS52.

Mining of the West Pit will not cease until the end of Year 20, immediately prior to closure. West Pit filling is described in RS52.

4.0 Waste Rock Stockpile Yield

This section describes the water balance of the waste rock stockpiles and presents preliminary design for management of a range of expected flow rates and volumes from these stockpiles and from stockpile construction areas. It also includes a discussion of the overall stockpile water balance; yields[3] from open liners, active stockpiles, and reclaimed stockpiles; liner leakage and the underdrain system; stockpile sump design; and collection of stockpile construction runoff. Average annual precipitation values used to calculate the range of annual flows from stockpile process water drainage are presented in Section 3.1.1 along with storm event statistics. Conveyance systems for process water are discussed in Section 7.0.

4.1 Stockpile Water Balance

A water balance was calculated for stockpile drainage to estimate the range of annual process water runoff and liner drainage expected from the stockpiles and to quantify the volume of process water produced from specific design storm events. As described in RS49, the waste rock will likely have a large initial water retention rate because the moisture content of the rock will be significantly lower than its field moisture capacity immediately after it is mined. This will result in the waste rock stockpiles permanently retaining a large amount of the stockpile infiltration during the first several years. Therefore, the water balance includes a significant but conservative loss (retention) of water into the stockpiles prior to being reclaimed, as described in further detail in Section 4.1.2.2.

The annual water balance predicted for the stockpiles is based on information obtained from previous studies of test piles at the Dunka Mine (Eger, Melchert, Wagner, 1999, Eger, Antonson, Udoh, 1990) and test piles from the AMAX exploratory shafts (Eger, Lapakko, 1985) in northern Minnesota and at the Cluff Mine in Saskatchewan (Nichol, Smith, Beckie, 2005). The test stockpiles had characteristics that were different than the proposed stockpiles, such as the shape, percent of area with flat grades, height of the piles, type of rock in the pile, side slopes, and cover types. Table 4.1-A compares several stockpile parameters from each of these studies to the proposed stockpile parameters. With the exception of the Cluff studies, all of these studies were completed in the

[3] Yield is the total volume of surface runoff, liner drainage, and leakage from the stockpiles. For active (unreclaimed) stockpiles, the total yield is water from the top of the liner, the underdrains, and surface runoff (all process water). For reclaimed stockpiles, the total yield of stormwater is water from the top of the cover, and the total yield of process water is infiltration collected on the top of the liner and water collected in the underdrains. See RS24 for details on stormwater.

immediate vicinity of the proposed Mine Site, with the same bedrock material. Because of the differences portrayed on Table 4.1-A, a range of expected yields have been developed that include surface runoff from a stockpile, infiltration through a stockpile, and drainage (infiltration collected by liner drains and underdrains) from a stockpile. These overall yields were used as a basis to compute a range of expected flows prior to the availability of field data from the Mine Site.

Two different volumes of yield were considered for this analysis: annual yield and storm event yield. Each of these quantities account for runoff and liner drainage from covered stockpiles and liner drainage from uncovered stockpiles and open liners. A range of annual yields were established to bracket flow rates and to estimate the water balance at the Plant Site; to design the stockpile pumps and pipes, mine pit pumps and pipes, CPS, and Treated Water Pipeline; to define the concentration of contaminants in the drainage from liners, to define the volume of water that requires treatment, and to analyze the water quality for the Mine Site, Tailings Basin, and pit filling operations. The storm event yields were used to size the stockpile and mine pit sumps, sump overflow ponds and ditches, process water ponds, and process water ditches. Design of the WWTF uses both the annual yields and the storm event yields.

Each computation that required volumes considered the critical volumes from the range that was established for annual yield, which may be the low value or the high value in the range provided. For example, the high predicted annual volumes provide conservative values for quantifying the volume of water that will require treatment. The low estimate of predicted annual volumes provides a conservative basis to estimate the water quality concentrations reaching the liner and stockpile runoff from unreclaimed surfaces. Volumes were also evaluated during various seasons to estimate the changes to the pumping and treatment systems. The highest seasonal flow, snowmelt, was analyzed to determine the maximum seasonal flow expected during the life of the mine for designing stockpile pumps, mine pit sumps, the WWTF, the CPS, and the Treated Water Pipeline.

Figures 4.1-A and 4.1-B provide schematic diagrams of the water balance parameters for the waste rock stockpiles, active (uncovered) and reclaimed (covered). Table 4.1-B presents the range of total annual process water volumes and flow rates estimated from the stockpiles based on data presented in Section 4. Table 4.1-C presents process water yields from uncovered stockpiles calculated for various storm events based assumptions presented in Section 4. The volumes and flow rates presented in Table 4.1-B and Table 4.1-C conservatively assume conveyance of all process water to the sumps, with no leakage occurring through the liner.

4.1.1 Yields from Open Liner

The total process water volume from an open liner during any single rainfall event was assumed to equal the precipitation; a conservative estimate that will be used for design of the water management system to minimize environmental impacts during large storm events. Foundation slope and very low infiltration rates of the liners will result in nearly all precipitation draining to the sump. Infiltration and evaporation have been assumed negligible on open liners during single storm events.

Waste rock stockpile liners will be constructed during the summer and fall seasons, when conditions allow. Construction of stockpile liners would not be feasible during frozen conditions, so a sufficient amount of liner would need to be constructed prior to winter to allow continued stockpiling of rock throughout the winter and spring. Open liners would be susceptible to damage during freeze-thaw cycles, and yield would be substantial if open liners are present during spring runoff. Therefore, a minimum thickness of waste rock will be placed on the liner shortly after construction, prior to the winter season, to protect the liner from freeze-thaw cycles, protect the Mine Site from substantial spring yield, and allow continued winter and spring operations. Therefore, it was assumed that open liners would not be present during the spring snowmelt event for calculation of seasonal process water yield.

Open liners were not used for computation of annual yields because it was assumed that open liners will be temporary, and waste rock will be placed shortly after construction, prior to winter operations. Therefore, average annual drainage from stockpiles was assumed to be either from active or reclaimed portions of the stockpiles.

4.1.2 Total Yield from Active Stockpiles

As described in RS49, waste rock will be placed on the liners in lifts, with a total lift height of 40 feet. All process water from active stockpiles will be collected on the liner, including any surface runoff and infiltration that reaches the liner.

4.1.2.1 Grading of Active Stockpiles and Surface Drainage

The tops of active stockpiles will be constantly changing as new lifts are added, and drainage patterns will vary. Reclamation will occur on lower lifts while construction of upper lifts continues, so management of the process water on the upper lifts will be required to prevent mixing with stormwater generated off the lower levels. The most effective way to prevent co-mingling of process water and stormwater would be to minimize process water surface runoff from the active upper lifts while managing stormwater as surface runoff from the lower reclaimed areas. Management of stormwater is described in RS24, and management of both stormwater and process water is also described in Draft-02 of RS49.

In general, grading of the tops of active stockpiles will be gradual and no special grading will be done to minimize infiltration into the waste rock. According to stockpile research (see Section 4.1.2.2), little to no surface runoff is likely to occur due to the coarse nature of the material. Although surface flows are not expected on a regular basis, they could occur during major storm events, so provisions to accommodate these flows have been made. As described in RS18, temporary dikes will be constructed along the perimeter of the stockpile top where trucks are hauling, which will minimize surface runoff over the sides. Therefore, in general, flow paths on the tops of active stockpiles will direct surface flows away from the perimeter to ditches down the access road or a riprap-lined channel down the sides of the stockpiles. Surface runoff from active benches would be directed along the bench to a riprap-lined channel or to the access road. Flows from these systems would be directed to the stockpile sumps (discussed in Section 4.1.5).

As described in RS49, active stockpiles will be designed to encourage infiltration on the tops and enhance surface runoff on the sides and benches of active lifts. Because the recently mined rock will have a high capacity for moisture retention when initially placed on the stockpile, precipitation that infiltrates into the stockpile (prior to the top of the stockpile being covered) will be stored in the voids of the stockpile until the stockpile reaches void capacity or preferential flow paths develop. Therefore, by taking advantage of the moisture retention capacity of the rock, an effective approach is provided to preventing the mixing of process water from the upper lifts with stormwater on lower reclaimed surfaces.

Typically the outer benches of an active stockpile will be uncovered for the longest period, so the moisture retention capacity of the rock could be reached while the stockpile remains active (see Section 4.1.2.2). Therefore, maximizing surface runoff along the benches will be encouraged to minimize the amount of water reaching the stockpile liner after the retention capacity of the rock has been met. As described in RS49, ditches will be constructed along the benches connecting to the road ditches or riprap-lined channels running down the slopes. The design of process water ditches running down the slope will consider the location of the stormwater collection system and design of the stockpile cover, so as not to interfere with stockpile reclamation and to avoid mixing process water with stormwater.

The top of the stockpile will typically be the most recently-mined rock with the largest potential to retain water; therefore it would be most beneficial to utilize the retention capacity of the top of the stockpile (the newly mined rock) prior to covering in order to minimize the potential for mixing stormwater from lower lifts and process water from active upper lifts. In some locations, the tops and lower benches may also be designed to encourage infiltration and evaporation by grading the stockpile towards a defined ponding area and sump, allowing infiltration and evaporation to occur. Any additional surface water in these areas will then be directed to riprap-lined channels or a temporary pump and pipe leading to the stockpile sump.

4.1.2.2 Range of Annual Yields from Active Stockpiles

As described in Section 4.1, annual yields from active stockpiles are used to estimate the water balance for the Plant Site; to design the stockpile pumps and pipes, mine pit pumps and pipes, WWTF, CPS, and Treated Water Pipeline; to define the concentration of contaminants in the drainage from liners; to define the volume of water that requires treatment; and to analyze the water quality for the Mine Site, Tailings Basin, and pit filling operations. The range of annual process water yields from active (uncovered) stockpiles is described in this section.

As shown on Table 4.1-A, the analysis of previous studies from the AMAX and Cluff studies found that the average total volume from uncovered stockpiles ranged from 44 to 58 percent of precipitation, with individual annual pile yields ranging from 28 to 66 percent of precipitation from uncovered stockpiles from the AMAX study (control piles FL1, FL4, and FL6). The high variability in yields was likely due to the wide range of grain sizes (from boulders to clay) of the rock, the resultant variability in the development of preferential flow paths and the moisture retention capacity of the material. Average total ranges of active stockpile volumes were used for PolyMet computations (up to 58 percent of annual precipitation), because the large proposed stockpile footprints (ranging from 54 to 565 acres) are likely to moderate the extremes found in these studies of smaller stockpiles, which ranged from 0.016 acres to 0.11 acres.

Table 4.1.2.2-A presents the total annual yield from active stockpiles as percentages of annual precipitation, listing estimated ranges once the waste rock is at field capacity (based on data from the test stockpiles) and the estimated number of years to reach field capacity (based on the estimated moisture retention capacity). As a conservative estimate, the annual process water volumes that were developed for each Mine year ignored the assumption that the stockpiles may not produce yields for several years while the rock is below field capacity after it is mined and initially stockpiled. However, the annual yields were reduced with an increase in height of the stockpiles to reflect the

available storage of water in the pile as the stockpile grows in height. The process water annual volumes shown in Table 4.1-B, which are partially based on the percentages from Table 4.1.2.2.-A assume that flows will occur within the first year. Because active stockpiles are placed on liners that extend beyond the base of the stockpile and are not yet covered, all outflows from the stockpile are collected on the liner and become liner drainage.

Information from the AMAX and Cluff mines indicate that uncovered stockpiles take close to 1 year to start producing the full volume of yield (based on 13- to 16-foot-high stockpiles compared to the 80- to 240-foot-high proposed stockpiles). This appears to occur because there is an initial moisture deficit in the waste rock and because the preferential flow paths had not yet developed. As the stockpiles increase in height, the volume of rock with available retention capacity will increase proportionally.

The Cluff study by Nichol, Smith, and Beckie (2005) calculated a median residence time of 4.4 years for natural rainfall to move through their 16-foot tall stockpiles, although water moved through the pile at a much faster rate through preferential flow paths. For example, Marcoline, Smith, and Beckie (2006) reported average pore water velocities of approximately 5 feet per year for the 16 foot stockpiles in the Cluff studies, with preferential flow velocities as high as 6 to 13 feet per day. Preferential flow paths were defined as a system of interconnected voids or pore spaces where infiltration flows to the liner and exits the system as drainage very quickly, bypassing or short-circuiting natural infiltration patterns through the pile. Development of preferential flow paths appears to be related to total quantity and intensity of rainfall and construction methods, mainly compaction, employed.

Although the AMAX study did not specify the length of time to produce yield, discussions with Paul Eger, one of the two authors of the report, determined that drainage was observed from the stockpile a few days after construction (2007). However, the full volume of yield was not observed for almost a full year. This difference in rate of drainage is likely due to the initial development of preferential flow paths prior to the movement of the wetted front through the pile.

Based on the extrapolation of time for the 13-foot-tall stockpiles (AMAX) and the 16-foot-tall stockpiles (Cluff) to produce yields, each 40-foot lift would take approximately 3 years of infiltration volume prior to producing significant drainage on the liner, assuming that preferential flow paths continue to develop linearly with the height of the stockpile. This estimate may vary depending on the precipitation cycle and the time to develop preferential flow paths. However, it indicates that the active stockpiles may produce very little drainage, a fraction of what they are capable of, during mining operations if they are covered within the first few years of exposure.

In addition, the estimate of the moisture retention capacity was compared to the water holding capacity of the waste rock (amount of water required to bring the rock from the wilting point to field capacity). The field capacity and wilting point of the waste rock have not been tested; however it is likely that the rock will be below field capacity as it is mined because of the lower groundwater levels around the pit as it is dewatered. Hewett (1980) calculated the retention capacity of material on the 80-foot-high LTV Steel Mining Company (LTVSMC) stockpiles and the 13-foot-high AMAX stockpiles. According to Hewett (1980), retention capacities for the LTVSMC stockpiles ranged from 6 to 25 percent by volume of the till cover, 0.11 to 0.89 percent by volume of the waste rock based on a model of the material, and 3.5 percent by volume of the waste rock based on the limiting values for the material. AMAX stockpiles had estimated retention capacities that varied from 6 to 25 percent by volume for the till cover, 20 to 25 percent by volume for topsoil, 0.4 to 3.1 percent by volume for the waste rock based on a model of the material, and 3.5 percent by volume of the waste rock based on the limiting value for the material.

By comparison, the specific retention (water holding capacity) for pit run sand is approximately 2 percent by volume and glacial till is about 19 percent by volume (Eger, Antonson, Udoh, 1990). Another source by Hewett (1981) lists boulders with a 10 percent grain size of 10 inches as having a specific retention of 3.5 percent by volume, while medium sand has a specific retention of 11 percent by volume. The modeled moisture retention capacity of the stockpiles by Hewett (1980) of 0.11 to 0.89 percent by volume falls well below the 3.5 percent by volume published for boulders; therefore it seems reasonable to predict higher retention capacities for the stockpiles than calculated in that report. In comparison, RS49 specifies that in Golder's experience, the difference between the moisture content of the waste rock and its field capacity is generally in the range of 1 to 5 percent by weight, depending on material specific properties.

Based on the estimated residence time of precipitation in the 16-foot-high Cluff stockpiles by Nichol, Smith, and Beckie (2005), the calculated average pore water velocities by Marcoline, Smith, and Beckie (2006), and the estimated moisture retention capacity of stockpiles higher than 80 feet, any estimated liner drainage is expected to be minimal prior to being covered. However, considering the preferential flow velocities calculated by Marcoline, Smith, and Beckie (2006), surface flow and preferential flow paths may direct water horizontally to the lower benches at a rate exceeding the reclamation of stockpile lifts. Therefore the high estimate of the annual range shown on Table

4.1.2.2-A allows for some horizontal flows to lower benches that circumvent the natural vertical flow. Circumvented flows may be higher on sloped sides although annual flows for active stockpiles were not separated for slopes and benches or for surface runoff (which is estimated to be minimal) and liner drainage.

During winter, flows from the test piles were noted to be negligible. According to Eger and Lapakko (1985), the AMAX stockpiles did not have any flows from any of the stockpiles in January, and very little flow in November, December, February and March, although the flow that did occur in those months was not quantified. At the Erie Mine (aka LTVSMC), Hewett (1980) reported some flows in November for all the stockpiles, in December from Seep 1, and again in March at all the stockpiles. The Cluff stockpiles measured outflow in November, then again in March, with frozen conditions in between. The interior of the stockpiles did not typically freeze in the winter and water from summer precipitation likely continued to flow when the outlets were not frozen. As a result, Table 4.1.2.2-A provides a range of runoff from active stockpiles in the winter between zero and 10 percent of annual precipitation to account for the period between November through March when there may be intermittent flows.

As discussed in Section 3.1.1, the snowmelt event produces the largest runoff volume in this area and was therefore used for design of the pump and pipe networks (discussed in Section 4.1.2.3). The snowmelt runoff was estimated to range from a minimum of zero for very high stockpiles where infiltration will dominate to a maximum of ten percent above natural snowmelt runoff conditions due to surface compaction and collection of infiltration from the liner. The snowmelt event occurs over 30 days, and the preferential flow paths may not be limited in capacity; therefore snowmelt runoff volumes similar to natural condition runoff may occur. Any snowmelt surface runoff that flows down the sides of the uncovered stockpile will be recovered on the liner and is included in this total volume; snowmelt runoff volumes were not quantified as surface runoff or infiltration, as the amount from each would vary based on whether the stockpile is frozen or not. Regardless of its path, all snowmelt from an active stockpile is considered process water and will report to the liner and WWTF. Data is not available from previous stockpile research for estimated snowmelt runoff on uncovered waste rock stockpiles; data from previous research was only available for total annual volume.

Under natural watershed conditions, the snowmelt runoff is typically about 28 percent of the total annual runoff as calculated in RS73B. The total annual stockpile yield (12-16 inches) is estimated to be higher than natural watershed runoff in this watershed (11.3 inches), and the percent of the total annual stockpile yield from snowmelt is estimated to range from less than 28 percent to slightly more than 28 percent.

4.1.2.3 Range of Storm Event Yields from Active Stockpiles

Storm event yields from active stockpiles is used to size the stockpile and mine pit sumps, sump overflow ponds and ditches, process water ponds, process water ditches, and the WWTF. The range of process water yields (total stockpile drainage) from active (uncovered) stockpiles is described in this section.

Storm event yields are a factor of total precipitation, intensity of the rainfall, and the moisture deficit and rate of infiltration of the ground surface. Larger storm events typically produce higher runoff rates when the soil moisture deficit is met or the intensity of the storm is greater than the infiltration rate of the soil. Storm event yields for active portions of the stockpiles were calculated using the SCS Runoff Curve Number (CN) method, which estimates surface runoff and peak discharges in small watersheds. The total yield from active stockpiles was computed by adding the surface runoff to the estimated infiltration that reaches the liner (described below). The results of this analysis were then compared to published literature for storm event yields from stockpiles in Northern Minnesota.

Although the SCS method is most appropriate for urban settings, it provides curve numbers representing appropriate runoff conditions for rural, agricultural, and arid and semiarid rangeland uses including gravel roads and grassy areas. The SCS Curve Number method is the most commonly used method for calculating rainfall runoff in the United States and is the core of many hydrologic models.

Curve numbers are chosen based on the hydrologic soil group, land use, cover type, and hydrologic condition. The hydrologic soil group is a classification of soils by infiltration rates, assuming bare soils after prolonged wetting. Most of the soils in the United States are classified into a hydrologic soil group, which is typically documented in a Soil Survey or specified according to texture.

Two Curve Number analyses were performed to evaluate movement of water through the stockpile to the liner. Because there is no Curve Number for stockpiles as a cover type or land use, the initial Curve Number analysis uses one Curve Number to reflect surface runoff and infiltration into the stockpile (infiltration equals precipitation minus surface runoff). Because all the water that enters the stockpile is either retained in the voids of the stockpile or moves through the stockpile to the liner, a second Curve Number analysis was conducted to simulate movement of water through the stockpile to the liner. The second calculation uses the infiltration quantity that enters the stockpile from the first calculation as input for the precipitation value for the second calculation.

All the water contacting active portions of a stockpile is considered process water, including surface runoff, if any, and infiltration that is captured on the stockpile liner. Therefore the runoff calculated from this first Curve Number analysis only reflects a portion of the total yield of the stockpile. It should also be noted that this initial calculation estimates runoff from the stockpile as the output. As shown in stockpile research discussed in Section 4.1.2.2, it is very infrequent that actual surface runoff is observed on active stockpiles. The runoff quantity calculated in this first SCS runoff equation will likely move through the pile as a combination of surface runoff and preferential flow through the stockpile rather than along natural infiltration paths.

The Curve Number analysis for active stockpiles used a Type C soil for gravel roads with a Curve Number of 89 to reflect movement of water along the top surface of the stockpile. Type C soils have low infiltration rates when thoroughly wetted and consist mainly of soils with a layer that impedes downward movement of water. This soil type was chosen due to the compaction that will be obtained from stockpile construction, which will impede the downward movement of water.

The amount of infiltration likely to follow natural infiltration paths was calculated as the difference between the total amount of precipitation minus the “runoff,” or total output from the first SCS runoff equation. This quantity of infiltration will consist of drainage to the liner and water retained by the stockpile material. Drainage to the liner was then estimated using a second SCS calculation based on the infiltrated volume from the first calculation.

Liner output from material within the stockpile will occur at a different rate than the surface runoff. Therefore, a second Curve Number of 76 was used based on a Type A soil and a cover type of a gravel road to estimate runoff to the stockpile liner. Type A soil was chosen based on a high rate of infiltration, even when thoroughly wetted, and a low runoff potential. This soil type consists of deep, well to excessively well drained sand, sandy loam or gravel with a high rate of water transmission, simulating conditions likely to develop within the stockpile. The volume of infiltration from the first SCS runoff equation was used as the precipitation in this second SCS equation. The resulting runoff from the second equation is the amount process water expected to reach the liner along natural flow paths from the storm event. The two quantities of process water, “surface runoff” and liner drainage, calculated by these two applications of the SCS runoff equation represent the total quantity of process water expected during a storm event. The separation of surface runoff and liner drainage from these calculations is not intended to accurately model each component on an individual basis, but rather to provide an estimate of the *total* process water predicted from a storm event from active portions of a stockpile.

The total quantity of storm event yields from active stockpiles will vary with height and total quantity of precipitation, similar to the annual yields. Process water yield from active stockpiles ranged from 68 percent of precipitation for the 10-year, 24-hour storm event to 82 percent of precipitation for the 500-year, 24-hour storm event, as shown in Table 4.1-C.

The results from these analyses were compared against stockpile studies in northern Minnesota. In *Hydrology of Stockpiles of Sulfide Bearing Gabbro in Northeast Minnesota*, Hewett (1980) isolated many individual storm events to estimate storm yields on uncovered AMAX stockpiles and uncovered Dunka stockpiles. For Dunka watershed EM-8, storm yield ranged from 0 to 66 percent of precipitation. However, it should be noted that this site may be partially influenced by exterior wetland flows. Storm yields from natural streams were also provided in this report as ranging from 5 to 90 percent, according to separate research by others.

Hewett also reported that AMAX stockpiles, which were isolated from surrounding areas and were fully lined to capture all flows, showed storm yields ranging from 0 to 96 percent of precipitation. Evaluation of the storm events and their yield from the AMAX stockpiles resulted in similar percentages of yield for similar rain events as predicted for the proposed stockpiles. Approximately half of the 16 AMAX rainfall-yield records are comparable to predicted yield estimates using calculations for the proposed stockpiles. Two-thirds of the AMAX storm events that did not correspond to predicted yield estimates for the Mine Site were for storms less than 1.2 inches.

Of the eight AMAX storms between 0.1 inches and 1.2 inches, half of the storms resulted in yield calculations very similar to predicted values for the Mine Site, and half of the storms resulted in yield quantities significantly higher than those predicted for the Mine Site. The four AMAX storms between 0.1 and 1.2 inches that were not in agreement with those predicted for the Mine Site ranged from 0.65 to 1.1 inches with measured yields ranging from 53 to 100 percent of precipitation, whereas the predicted yields ranged from 16 to 30 percent of precipitation for the Mine Site. No explanation was given as to why the AMAX yields from these four storms were so unusually high for the small amount of rainfall.

Of the remaining four AMAX storms that did not match predicted yield estimates for the Mine Site, two of these storms ranged from 1.5 to 1.7 inches with measured yields from 67 to 70 percent of precipitation, and predicted yields are 43 and 48 percent for the Mine Site. The remaining two storms had rainfalls of 2.4 and 3.6 inches with measured yields of 41 and 8 percent, and predicted yields of 59 and 70 percent at the Mine Site. Overall, the AMAX storm yield calculations are inline with the predictions using the SCS Curve Number method described for the Mine Site.

Monitoring of initial stockpiles constructed at the Mine Site is recommended for various heights to refine the range of likely yield so that subsequent stockpile water management infrastructure, such as future sumps or pumps and pipes, can be designed based on actual site data. If monitoring shows that there is more process water than assumed in Section 4 of this report, infrastructure can be increased; if monitoring shows that there is less process water than expected, future infrastructure can be reduced in size. Additionally, the moisture retention capacity and field capacity should also be tested to calculate the number of years before the waste rock will produce flow. Stockpile monitoring will be defined during permitting.

4.1.3 Total Yield from Reclaimed Stockpiles

PolyMet plans to cover stockpiles progressively. The timing of cover placement will have a large impact on the total quantity of process water from the stockpile, as described in Section 4.1.2. However, computations for reclaimed stockpiles assume that the waste rock has little to no excess moisture retention capacity remaining (i.e. volumes are based on long-term stockpile conditions).

As shown on Figure 4.1-B, the total volume from reclaimed stockpiles will include:

- Surface runoff from the top of the vegetated cover that is routed into the stormwater system.
- Interflow that infiltrates into the cover and flows along the cover, exiting the stockpile cover without coming in contact with the waste rock. This water will be routed into the stormwater system.
- Precipitation that infiltrates into the cover but does not pass through the cover and is either transpired through vegetation on the cover or evaporated from the cover surface. This water will not be seen in any collection system.
- Infiltration through the cover that is retained by the waste rock. This water will not be seen in any collection system.
- Infiltration through the cover that drains through the waste rock and is collected on the liner and routed to the sump.

- Leakage through the liner, which is discussed in Section 4.1.4.

4.1.3.1 Grading of Reclaimed Stockpiles and Surface Drainage

In general, the volume of process water that reaches the liner can be reduced by grading the tops of reclaimed stockpiles to drain quickly (increasing stormwater runoff and reducing infiltration); the volume of process water that infiltrates will be increased if the tops of reclaimed stockpiles are flat (allowing ponded water to infiltrate). Grading of the reclaimed stockpiles is described in RS49 and varies depending on the cover type. According to RS49, the covers will generally be sloped to direct surface runoff to grass-lined collector channels spaced at approximately 200 foot intervals. The collector channels will direct flows to riprap-lined channels down the sides of the stockpiles (described in RS49 and discussed in RS24). As discussed in RS49, ditches will be constructed on the benches with stormwater directed to the riprap channels. Stormwater runoff from reclaimed stockpiles is discussed in more detail in RS49 and RS24.

4.1.3.2 Process Water Estimates from Reclaimed Stockpiles

The cover types used on the stockpiles impacts the volume of stormwater runoff from stockpiles and the infiltration through the cover and waste rock that becomes process water. There are two different covers that will be used on the stockpiles, according to the waste category of the rock (as described in RS49) including an evapotranspirative (ET) cover and a membrane cover. Annual and storm event quantities of stormwater runoff is estimated in RS24.

Stockpile cover hydrology is largely a function of the soil type. The covers designed for the stockpile are described in further detail in RS49, with characteristics generally described in this report. Additional details on specific soil properties will be provided in permitting and final design of the stockpile cover.

4.1.3.2.1 Annual Yields for Reclaimed Stockpiles

Annual yields from reclaimed stockpiles are one component used to estimate the water balance for the Plant Site, to design the stockpile pumps and pipes, mine pit pumps and pipes, WWTF, CPS, and Treated Water Pipeline, to define the quantity of water that requires treatment, and to analyze the water quality for the Mine Site, Tailings Basin, and pit filling operations. The range of annual process water yields (liner drainage) from reclaimed stockpiles is described in this section.

The analysis of liner drainage from ET-covered stockpiles was primarily based on data from an AMAX stockpile (FL2) that was covered with topsoil and vegetated. The average annual total surface runoff plus liner drainage from this stockpile was approximately 30 percent of the precipitation; individual annual volumes ranged from 16 to 36 percent. Volumes from covered stockpiles were generally less than the volumes from uncovered stockpiles. This is likely due to the transpiration from vegetation on the cover and the capillary break between the fine-grained cover material and the coarse-grained stockpile material. A capillary break forms when water from the smaller pore diameter in the cover material does not release water to the larger pore diameter in the underlying waste rock, even when the fine material is saturated. Water is able to move freely between materials in areas with similar pore sizes, but capillary pressures restrict free movement of water between materials with very different grain sizes.

A measure of vegetative cover was provided for the first five years of the AMAX study. AMAX stockpile FL2 had 47 percent cover in 1979 (third year), 56 percent in year four, and 62 percent vegetative cover in year five (1981). Because this covered stockpile was only about 50% vegetated, precipitation losses included an annual transpiration rate of 4 inches (about 50% of the typical 8 inches) and the annual average evaporation rate of 10 inches. Runoff from the stockpile did not decrease on a rate reflecting the vegetative growth; FL2 had runoff rates of 34 percent of precipitation in 1979, 16 percent in year four, 31 percent in year five (1981), 35 percent in 1982, and 24 percent in 1983. There was no major decrease in annual yield from this stockpile in the last two years (year six-1982 and seven-1983) of the study that could be attributed to a significant increase in vegetative growth, and no data was provided on the percent of vegetative cover for years six and seven. Full vegetative cover and growth are an important component in the reduction of process water with ET covers. This study did not present the full potential of an ET cover due to the lack of full vegetation on the stockpile.

A MDNR study at LTVSMC titled "*Using Passive Treatment Systems for Mine Closure – A Good Approach or a Risky Alternative*" by Eger, Melchert and Wagner (1999) predicted a reduction in infiltration when stockpiles were covered with native soils, based on previous research: a 40% reduction when covered with uncompacted native soils and a 60% reduction when covered with compacted native soils. Reductions of 90% had been observed with a membrane cover. For comparison, the Cluff Mine stockpile data reported infiltration rates on covered stockpiles of about 14 percent of precipitation.

Unlike active stockpiles, reclaimed stockpiles will have been uncovered for several years and preferential flow paths may have already developed. Therefore, it is conservatively assumed that there is no reduction in flows due to moisture retention by the rock. Tables 4.1.3-A and 4.1.3-B present the range of estimated annual liner yield (process water) for both cover types. The range of estimated surface runoff (stormwater) from reclaimed stockpiles is presented in RS24.

Drainage values from test piles are primarily from flat surfaces. It is a well known hydrologic principle that the surface runoff from inclined surfaces increases with the slope. It is estimated that, compared to flat surfaces, surface runoff would increase by about 20% for side slopes with a rise of 1 foot over 2.5 to 3 feet, the proposed slopes on the stockpiles. There would also be a corresponding change in the liner drainage below sloped surfaces. Therefore, the values in Table 4.1.3-A present separate volume percentages for the stockpile side slopes. Surface runoff from membrane covers is expected to be consistent for various slopes due to the drainage layer on top of the membrane.

Winter flows from reclaimed stockpiles are expected to be similar to active stockpiles, because total vegetative transpiration will not occur over the winter months. Winter liner flows with a membrane cover are expected to be minimal because infiltration is still limited by the membrane barrier.

Snowmelt yields could be the same as the total runoff from the natural watershed, slightly lower because some of the water may infiltrate into the cover materials, or slightly higher due to a decrease in hydraulic conductivity of the cover materials. Annual volume percentages were estimated as a sum of summer and winter volumes, with snowmelt volumes included in the summer volumes. Maximum process water volumes during snowmelt were assumed to be limited by the maximum summer volumes. The minimum process water volumes from reclaimed stockpiles during the snowmelt event could be zero, because all of the snowmelt may runoff as stormwater (assuming the depth of freezing may be greater due to the consolidated cover material).

As a result of this analysis, the estimated annual process water volume collected in the sumps from reclaimed stockpiles ranges from 13 to 36 percent of precipitation for ET covers and from 1 to 5 percent of precipitation for membrane covers, as shown on Tables 4.1.3-A and 4.1.3-B. This process water yield estimate assumes all water is collected on the liner or in the foundation underdrains, with no loss, as discussed in Section 4.1.4. Process water volumes may decrease over time after reclamation, as discussed in RS52.

4.1.3.2.2 Storm Event Yield for Reclaimed Stockpiles

Storm event yield from reclaimed stockpiles is used to size the stockpile and mine pit sumps, sump overflow ponds and ditches, process water ponds, process water ditches, and the WWTF. Process water yields (liner drainage) from reclaimed stockpiles during storm events are described in this section.

Storm event process water yield was calculated based on the SCS Runoff Curve Number (CN) method in a two-stage process, similar to calculations for liner yield in active stockpiles, as described in Section 4.1.2.3. However, the analysis for reclaimed stockpiles varies according to cover type, and only includes the quantity of water that reaches the liner, which is a small portion of the overall stockpile water balance; therefore the sizing of process water infrastructure based on storm event yields is not nearly as dependent on yields from covered stockpiles as it is on the yield from active stockpiles.

All stockpile covers will be partially comprised of native soils from the Mine Site. A soils map, attached as Figure 4.1.3.3-A, includes both soil data and landform data at the Mine Site according to the U.S. Forest Services' Draft Superior National Forest Ecological Classification System with additional data assembled by Dr. David Grigal for the project EAW. A detailed description of the soil mapping units from Figure 4.1.3.3-A is included in Appendix C; and additional soils information at the Mine Site is provided in RS24 and RS25.

Based on the soils information provided and as described in RS24, curve numbers for grass in fair condition were assigned to the three cover types as follows:

- The ET cover was evaluated as a Type B soil, because 60 percent of the soils at the Mine Site represent these characteristics according to the soils map. Stockpiled overburden from the Mine Site will be used to develop this cover. The remaining 40 percent of the Mine Site consists of Type D soils, which are less appropriate for vegetative growth on the stockpiles. Additionally, Type D soils, when drained, take on characteristics of A, B, or C soils, depending on the adequacy of drainage and the type of soil. Use of Type B soils provides a higher rate of infiltration and liner yields than the Type D soils; silt loam and loam represent typical soil textures classified as Type B soils.

  An SCS Curve Number of 69 was used for the ET cover to reflect open space with grass cover in fair condition (50 to 75 percent cover) with a Type B soil. This computation separates the stormwater runoff (surface runoff and interflow), calculated by the SCS runoff equation, from the infiltration. All precipitation that did not runoff (including interflow) was assumed to infiltrate into the waste rock stockpile and become process water. A second SCS Curve Number was then assigned to this infiltration quantity to calculate yield that reaches the liner from within the stockpiles.

As described in Section 4.1.2.3, the second SCS Curve Number in these analyses utilizes a Type A soil, which assumes a high rate of infiltration and a low runoff potential. This soil type simulates predicted conditions within the stockpile. An SCS Curve Number of 76 was used based on a cover type of a gravel road, which is the closest cover type to relate to the stockpile material. This Curve Number was applied to the infiltration quantity from the first SCS runoff equation to get a second runoff quantity. This second runoff volume is the volume of process water expected to reach the liner and is the total quantity of process water expected from a storm event from reclaimed portions of the stockpile with an ET cover.

- The membrane cover was classified as a Type D soil due to the high runoff potential and restrictive infiltration characteristics of the membrane. Examples of a Type D soil range from clay loam, silty clay loam or clay. This soil type is also used when there is a high water table or bedrock at or near the surface. An SCS Curve Number of 84 was chosen based on the soil type and open space with grass cover in fair condition (50 to 75 percent cover). This computation separates the stormwater runoff (surface runoff and interflow), calculated from the SCS runoff equation, from the infiltration. All precipitation that did not runoff (including with the cover was assumed to infiltrate into the waste rock and become process water. A second Curve Number was then assigned to this infiltrated volume to calculate yield that reaches the liner from within the stockpiles.

  As described in Section 4.1.2.3, the second SCS Curve Number in the analysis utilizes a Type A soil, which assumes a high rate of infiltration and a low runoff potential. This soil type simulates predicted conditions within the stockpile. An SCS Curve Number of 76 was used based on a cover type of a gravel road, which is the closest cover type to relate to the stockpile material. This Curve Number was applied to the infiltration quantity from the first SCS runoff equation to get a second runoff quantity. This second runoff volume is the volume of process water that would reach the liner and is the relative quantity of process water expected from a storm event from reclaimed portions of a stockpile with a membrane cover.

  The total stormwater yield from a stockpile with a membrane cover is also very dependent on the number of defects or tears and punctures in the membrane. If installed properly, the membrane itself should not transmit water. However, as described by Eger et al, it is unreasonable to assume that a membrane is completely intact (1990). Generally, the better the installation and the thicker the liner, the less leakage that would be expected to occur. This analysis conservatively assumes that leakage through the membrane cover occurs, estimated to be average about 10 percent of precipitation for storm events between the 10-year 24-hour and the 500-year 24-hour storm. This assumption likely overestimates the yield to liner with a membrane; however, the percentage is much lower than that calculated for active stockpiles. The total volume of process water from a stockpile with a membrane cover is less than 10 percent of the volume of process water from an active stockpile; therefore sizing of the stockpile sumps, pumps, and ponds were calculated based primarily on the yields from the active stockpiles and were much less dependent on the yields from the reclaimed portions of the stockpiles.

- The combination ET and membrane cover was calculated using the ET cover on the side slopes, classified as a Type B soil, and a membrane cover on the tops and benches, which is classified as a Type D soil. The volume of process water to reach the liner from a storm event with a combination of ET/membrane cover was calculated combining the analyses discussed for each cover type above.

Based on this analysis of cover types, runoff quantities, both stormwater and process water, were calculated for use in sizing various infrastructure for the Mine Site.

4.1.4 Liner Leakage and Foundation Underdrain System

Design of the liner system and foundation underdrain system were described in RS49. This section evaluates the estimated leakage through the liners and the water collection in the foundation underdrains.

4.1.4.1 Liner Leakage

The estimated leakage through a stockpile liner is based on the liner permeability and the percent of precipitation that reaches the liner. As mentioned above, the liner system designed for each stockpile is dependant on the waste rock category, as discussed in Draft-02 of RS49 and RS23T, which also presents liner design and performance as well as details of leakage calculations. Estimates of liner leakage rates for active and reclaimed stockpiles were based on simulations using the Hydrologic Evaluation of Landfill Performance (HELP) model, which was developed by the United States Environmental Protection Agency (USEPA) to estimate drainage from and leakage through liner systems. The HELP model produced a synthetically-generated 100-year weather record for Duluth, Minnesota to model liner leakage based on permeability rates for the proposed liners. These leakage volumes are conservative estimates to define potential impacts to water quality in the Partridge River, as described in RS74.

As described in Sections 4.1.2 and 4.1.3, drainage through the stockpile to the liner is significantly reduced once portions of the stockpile are reclaimed; therefore the leakage through the liner is also significantly reduced once reclaimed (i.e., less water through the pile equals less water through the liner). The significance of this reduction to liner leakage rates is discussed in further detail in RS74. The liner leakage rates were developed according to the reactivity of the stockpiles and are as follows for open stockpiles (before reclamation with cover systems):

- Category 1/2 stockpile will have a subgrade prepared with the upper one foot engineered to a maximum permeability of 5 x 10^{-7} centimeters per second (cm/s) and an overliner drainage layer. Modeling of this liner system results in an average annual leakage through the liner of about 464 gallons per acre per day (gal/acre/day) along active portions of the stockpile.

- Category 3 stockpile will have a subgrade prepared with the upper one foot engineered to a maximum permeability of 1 x 10^{-5} cm/s, overlain by a geomembrane liner and an overliner drainage layer, which results in a liner leakage rate of approximately 2 gal/acre/day.

- Category 3 lean ore, Category 4 waste rock, and Category 4 lean ore surge pile will have a subgrade prepared with the upper one foot engineered to a maximum permeability of 1 x 10^{-6} cm/s, overlain by a geomembrane liner and an overliner drainage layer. With this liner system, these stockpiles will have a liner leakage rate of less than 1 gal/acre/day, which is currently under further evaluation for RS49 and RS74.

Table 4.1-B presents the range of total annual process water volumes and flow rates estimated from the stockpiles based on the estimated percent yields provided above in Sections 4.1.2.2 and 4.1.3.2 and in Tables 4.1.2.2-A (uncovered), 4.1.3-A (with ET cover), and 4.1.3-B (with membrane cover). The annual process water volumes and flow rates presented in Table 4.1-B conservatively assumes conveyance of all process water to the sumps, with all water captured by the liner or underdrains.

4.1.4.2 Foundation Underdrain System

In order to facilitate foundation drainage, an underdrain system will be designed for each stockpile, as illustrated in Figure 4.1.4.2-A. This underdrain system was designed in RS49. According to Draft-02 of RS49, this underdrain system will consist of corrugated polyethylene pipes spaced at a nominal distance of 100 feet. The underdrains will be designed so that they are above the groundwater elevations as much as possible, to avoid continual pumping of groundwater. Any water collected by the foundation underdrain system will be conveyed to a series of small sumps separate from, but generally located next to the stockpile process water sumps, as shown in Figure 4.1.4.2-B.

These underdrain sumps will be designed to overflow into the stockpile process water sumps, but the underdrain discharges to the sumps will be separated from the liner system discharges to allow monitoring of water quality and quantity from each system independently. The underdrains are not designed specifically to capture leakage through the liner, in the unlikely event that it occurs. However, the potential exists that leakage could occur and these underdrains could serve to capture some of that leakage.

4.1.5 Stockpile Sump Design

The stockpile foundations and liner systems will be constructed so that water that reaches the liner will drain by gravity into process water sumps. Stockpile foundation and sump liner systems were designed in RS49. Stockpile sumps will be lined, as described in RS49, according to the reactivity of the stockpile. The Category 1/2 stockpile sumps are located between the stockpile and the West Pit and have been designed for a typical event (0.9 inches of rainfall) with all overflow conveyed along similarly-lined ditches to the West Pit. The Category 3 and 4 stockpiles and sumps are mainly located along Dunka Road where space is limited. The Category 3 and 4 stockpile sumps have been designed for a less frequent event (10-year, 24-hour), and higher flows, up to the 100-year, 24-hour storm event, will be allowed either to back up into the stockpile area or collect in adjacent lined overflow ponds.

Preliminary sump areas are listed in Table 4.1.5-A with preliminary overflow pond specification listed in Table 4.1.5-B. The pump and piping systems necessary for collection and conveyance of process water from these sumps are discussed in Section 7. All sumps were designed with an average depth of 6 feet.

Construction of a lined sump requires adequate foundation drainage to prevent excessive pore pressure from developing under the liner. Further geotechnical investigation and hydrologic monitoring will be required after foundation elevations are established to facilitate this design. If groundwater or surface water is expected to remain near the surface once pit dewatering begins, these sumps may need to be constructed partially above the surrounding ground surface. Although more expensive, other design approaches that may be used, if required, include excavation of the underlying material down to a suitable grade and rebuilding the foundation with a well-drained material, installing underdrains to facilitate a dry foundation, constructing dikes or trenches with subsurface leakage control measures, as described in RS25, or some combination of these methods.

4.1.5.1 Category 1/2 Sumps

Category 1/2 sumps have been designed to contain process water runoff and drainage from active and reclaimed stockpiles during a 0.9 inch typical rainfall event with the flood level below the lowest stockpile liner elevation. According to the 30 years of precipitation data compiled for the Mine Site (see Section 3.1.1), the 0.9-inch rainfall event typically occurs about 4.5 times a year and is equivalent to the 2-month, 12-hour storm for this area. Stockpile storm event yields, as described in Section 4.1.2.3 (active stockpiles) and Section 4.1.3.2.2 (reclaimed stockpiles) calculated for the design storm of 0.9 inches were used for the design of the Category 1/2 sumps.

The spatially distributed weather records compiled for RS73, RS74, and RS49 between 1971 to 2001 were analyzed to calculate the appropriate design storm for the Category 1/2 stockpile sumps; compilation of this weather record is described in RS73A. The size and frequency of typical storm events at the Mine Site were evaluated for this 30-year record. The average storm event for the 30-year period was 0.17 inches with an average of about 193 storm events each year. Of all the storm events evaluated, the 0.9 inch storm was chosen for sizing the Category 1/2 stockpile sumps because that size storm occurred or was exceeded about 4.5 times per year. Any overflow from these sumps is directed to the West Pit and has potential to impact mining operations if not appropriately managed, and this frequency of overflow was acceptable for operations.

For volumes in excess of the design storm, lined dikes will be constructed around the perimeter of the sumps to direct higher flow volumes from each sump into a similarly-lined ditch that routes overflow into the West Pit by gravity for temporary collection prior to pumping to the WWTF. Figure 4.1.5.1-A illustrates the sump and ditch conceptual design for Category 1/2 stockpile sumps. A riprap-lined spillway or pipe system will be constructed at the rim of the West Pit to prevent severe erosion of surface soils at the rim where flows enter the pit. The Category 1/2 sumps and the overflow ditches will be lined with a single composite liner system consisting of an upper 60-mil HDPE geomembrane over a one-foot thick soil liner as shown in Figure 4.1.5.1-B and described in RS49.

The size of the design storm was analyzed to evaluate the feasibility of the small, typical storm event compared to the standard storm event, the 10-year 24-hour event, used for the sumps on the remainder of the stockpiles.

- The 10-year 24-hour storm event was analyzed, which has a 10 percent chance of being exceeded in any given year; the 10-year storm event is the event used for sizing the Category 3 and 4 stockpile sumps. Table 4.1.5.1-A lists the difference in volume and area required for using the 10-year event for Category 1/2 stockpile sumps rather than a 0.9-inch rain storm, which would typically occur about 4.5 times a year. The change in size of the Category 1/2 sumps for the 10-year event would be an increase of over 80 percent of the area required for the 0.9 inch storm.

- The 100-year 24-hour storm event was also evaluated to manage all the water without overflow to the West Pit. The preliminary requirements for each sump based on this storm event is also listed in Table 4.1.5.1-A. The change in size of the sumps for the 100-year event would be an increase of approximately 97 percent of the area,

This analysis indicates that these overflow ditches will typically be used about 4.5 times per year. Increasing the sump areas to a larger storm event would alleviate regular use of these overflow ditches. However, regardless of the storm event used to size these sumps, lined overflow ditches to the West Pit would still be required to accommodate flows up to the 100-year 24-hour storm event, as required by the design criteria. These lined ditches to the West Pit represent the contingency in place for power outages and storm events greater than the design storm up to the 100-year storm event with 1 foot of freeboard for additional flows. With this contingency in place, it is not cost effective to provide larger sumps without sufficient justification. The overflow from these sumps will be carried by these ditches to the edge of the pits, and then conveyed down to the sumps in the pits. All of the water from the pit sumps is collected and conveyed to the WWTF for the same method of treatment as from the Category 1/2 stockpile sumps.

One question that was raised about designing the sumps for such a small storm event was regarding the concern about the accumulation of sediment in the sump that could wash through the stockpile. None of the research studies reviewed made any mention of sediment buildup in the collection pipes or systems from the stockpiles evaluated. Paul Eger, MDNR researcher involved in several of the research studies on Dunka and AMAX stockpiles, stated that sediment accumulation in the collection systems was never an issue in his research (personal communication, 2007). Many of the Dunka stockpile collection seeps were at the base of the pile, and the accumulation of sediment or sediment moving out the seep streams was never an issue.

Mr. Eger also discussed sediment in reference to the AMAX stockpiles which were constructed on top of an impervious liner sloped to a 6-inch perforated plastic pipe. These stockpiles did not include any drainage layer as proposed, although there was a layer of pit run fill between the hypalon liner and the waste rock to act as a protective buffer for the liner. MDNR observed these stockpiles from 1977 through 1983, and to Mr. Eger's knowledge, there was never an issue with sedimentation in the pipes or in the sumps, which included a ¾-inch pipe and a 5-gallon bucket for the water quality sampling station. Sediment flow in the Category 1/2 sumps will be evaluated during further design, but it is not believed to be a controlling factor for sizing the sumps.

4.1.5.2 Category 3 and 4 Sumps

Category 3 and 4 sumps will be designed to contain process water runoff and drainage from active and reclaimed stockpiles during a 10-year 24-hour rainfall event with the flood level below the lowest stockpile liner elevation. Stockpile storm event yields, as described in Section 4.1.2.3 (active stockpiles) and Section 4.1.3.2.2 (reclaimed stockpiles) calculated for the 10-year, 24-hour design storm were used to design the Category 3, Category 3 Lean Ore, and Category 4 sumps.

Dikes will be constructed around the perimeter of the sumps with capacity for the 100-year process water yield. To minimize uncontrolled overflows from the sumps, one of two methods or a combination of the two methods will be employed to manage overflow:

- The 100-year 24-hour event volumes in excess of the sump capacity will either be allowed to back up into the drainage layer of the liner up to a maximum depth of 1 foot for a period of less than 2 weeks; or

- The 100-year 24-hour event volumes in excess of the sump capacity will be conveyed by gravity through a lined ditch or pipe to lined overflow ponds PW-5 and PW-6.

The decision as to which method or how much of each method will be employed will require further design and analysis of the stockpile foundation design and site grading, and will be determined during permitting. The current design assumes the 100-year volume in excess of the sump capacity will be conveyed to the overflow ponds. This is considered a conservative assumption in that the amount of wetland impacts could be reduced by reducing the size of the overflow ponds and allowing storage of the excess water within the stockpile.

The drainage layer is depicted in Figure 4.1.5.2-A, which was taken from RS23T. Figure 4.1.5.2-B illustrates the conceptual sump and dike design for Category 3 and 4 stockpile sumps with overflow contained within the stockpile. Table 4.1.5-B lists the preliminary design specifications for the overflow ponds assuming capacity for stockpile runoff in excess of the sump capacity up to the 100-

year storm event yield volume. Conveyance into the overflow ponds may be gated to contain high volumes on the stockpile liner or route high volumes to the ponds.

An emergency operating procedure has been developed to manage process water volumes in excess of the design event for the overflow ponds PW-5 and PW-6. This contingency plan includes an overflow ditch from the Category 3 overflow pond PW-6 to the East Pit, and a plan of action for the Category 3 Lean Ore / Category 4 overflow pond PW-5. This plan of action includes use of road dewatering trucks with temporary diesel pumps to operate during events greater than the design event (100-year storm). This plan will include procedures to maintain water levels below the capacity of the pond, pumping to either a tanker truck or a temporary tank for storage until process water volumes are down to manageable levels in the overflow pond. Pit dewatering will temporarily be stopped during these conditions to allow drainage of stockpile sumps to the WWTF and minimize sump overflow. Due to the nature of the water and the overflow routes of other process water systems, maintaining water levels below the capacity of PW-5 and PW-3 (Rail Transfer Hopper runoff) will have precedence over pumping from any other system (i.e., pit dewatering, overburden runoff, haul road runoff, etc.)

The liners on Category 3 and 4 stockpiles will be open only prior to waste rock placement, which is expected to occur shortly after construction of the liner. The sumps were not designed for open liners, because open liners will be short lived; however, during periods with an open liner, the sumps have capacity to contain the entire precipitation volume from the 2-year, 24-hour rainfall event with the entire precipitation volume from a 25-year, 24-hour rainfall event contained on the foundation liner within the drainage layer.

These sumps will be lined as designed in RS49, consisting of a double composite liner system with an upper 60-mil HDPE primary liner, a geonet leak collection and recovery system (LCRS), and a 60-mil HDPE secondary liner overlying a one-foot thick soil liner as shown in Figure 4.1.5.2-C.

4.1.5.2.1 Category 3 and 4 Sump Overflow Ponds

As described in Section 4.1.5.2, two process water ponds will be constructed to collect gravity overflow from the Category 3 stockpile (PW-6) and the Category 3 Lean Ore and Category 4 stockpiles (PW-5) up to the 100-year, 24-hour storm event. Process water will either be contained within the stockpile foundation liner or routed to these process water ponds. Water into these ponds may be controlled by a gated inlet, which could backup water on the stockpile liner instead of allowing flow to the ponds. Water would be conveyed either through a lined ditch or a pipe to these ponds.

In general, these overflow ponds will be partially excavated and partially filled above the natural ground, designed to ensure that the pond bottom is above the expected groundwater elevations once the pit dewatering begins. These overflow ponds may require periodic pumping during high precipitation periods when precipitation minus evaporation within the pond boundary is positive, even when there are no overflows from the sumps.

These overflow ponds will only contain sump overflow water during events exceeding the 10-year, 24-hour storm. Overflow pond water will be conveyed to the WWTF as soon as the capacity is available in the pipeline system, taking priority over the stockpile sumps. Additionally, any water reaching the overflow ponds will be significantly diluted due to the size of the event. Therefore, based on the infrequent use, short duration of time needed, and diluted nature of the water, the overflow ponds will be designed with the same liners as the Category 1/2 sumps, which are shown in Figure 4.1.5.1-B and include an upper 60-mil HDPE geomembrane over a one-foot thick soil liner.

The pond dikes and slopes will be vegetated to limit erosion. The pond dike design will be conducted once the foundation grading design is completed and pond elevations can be established. The pond elevations will allow the collection ditches and pipes to convey by gravity into the ponds, and will be low enough so that additional storage can be provided above the ground. This additional storage also increases sediment-trapping efficiency.

4.2 Construction Runoff Collection

As described in RS49, waste rock stockpile construction will consist of excavating the footprint down to suitable material and building the foundation on that material. This will result in a large area of disturbance while construction of the foundation is occurring. Construction runoff is considered process water because it has contacted disturbed surfaces, and will therefore be collected and conveyed to the CPS.

Although this water will be collected, these construction areas may fall under the jurisdiction of the NPDES construction stormwater permit or industrial stormwater permit, which will be determined during permitting. Regardless of permit coverage, design and containment of these construction areas will meet or exceed the requirements of the Minnesota Pollution Control Agency’s (MPCA’s) stormwater permit programs. As shown on Figures 1.1-A through 1.1-E, dikes will be built to block surface runoff from leaving the stockpile construction areas. A small sedimentation pond will be built within the construction zone to collect the surface runoff and route it to the CPS. Annual process water volumes for construction water are estimated to be approximately the same as natural runoff rates (40% of annual precipitation) and spring snowmelt runoff volumes are estimated to remain the same as historic natural runoff rates (3.2 inches).

Because it will not require treatment beyond sedimentation and will only be temporary, construction water may be collected and conveyed separately from the stockpile process water. This may result in temporary parallel process water pipes while construction is progressing. The construction water may be combined with the unreclaimed overburden stockpile runoff. It will be routed to the CPS for conveyance to the Tailings Basin.

The construction sequence for Category 3 and 4 stockpiles will require clearing the areas several years in advance of excavating and building the foundation and liner. The construction area for the next 5 year period is relatively small, and construction will proceed in approximately five year increments, with the construction area dikes being built about five years in advance of stockpile progression for Category 3 and 4 stockpiles.

The Category 1/2 stockpile will progress much faster because it must hold a larger volume of waste rock and will ultimately have a much larger footprint than the Category 3 and 4 stockpiles. The area for full buildout of each 5 year increment is substantially larger than the other stockpiles, and it will take a significant amount of time and effort to complete all of the clearing, grubbing, and stockpile construction in the footprint that needs to be re-worked. Progression of the Category 1/2 stockpile footprint will likely proceed about half the distance of the next five year increment, as opposed to the full footprint that is extended for the Category 3 and 4 stockpiles. Therefore, as shown on Figures 1.1-A through 1.1-C, half of the following Plan Year's footprint is surrounded by dikes for the Category 1/2 stockpiles and the full footprint is surrounded by dikes for Category 3 and 4 stockpiles.

Table 4.2-A lists the estimated average annual runoff volume (acre-feet) and the equivalent average annual runoff rate (gallons per minute) for stockpile construction runoff at the end of each Mine Year 1, 5, and 10. All stockpile foundations will be constructed by Mine Year 10. The pump and piping system necessary for collection and conveyance of this water is discussed in Section 7. Due to the temporary nature of these systems and because construction runoff will only be collected during non-frozen conditions, hoses and portable pumps may be used in some instances.

4.2.1 Construction Runoff Collection Overflow Contingencies

The process water from the stockpile construction areas will only require treatment for TSS to meet water quality standards, which makes it no different than construction stormwater on other construction sites that are routed into natural drainage systems following treatment for sedimentation.

However, there are safeguards in place, as discussed in Section 4.2, to collect and convey the stockpile construction area process water to the Tailings Basin rather than diverting it into the natural stormwater system. The reason this water is collected is due to the proposed reuse/recycle strategy, with no process water discharge proposed to surface waters of the state under normal operations. This water is considered process water, because it has contacted disturbed surfaces and may not meet water quality limits for TSS.

The primary safeguard for collection and management of the stockpile construction areas is the dikes constructed to contain the process water areas from adjacent stormwater areas. These dikes will be sized depending on the size of the construction area and the size of the containment pond required within the dikes. Generally construction areas will have a maximum of a 5-year life prior to build-out of that construction area, so ponds will be sized to store the runoff from the construction footprint from the 5-year, 24-hour storm. Pumps and pipes will convey this water to the CPS for further conveyance to the Tailings Basin.

For storm events in excess of the design storm (5-year, 24-hour event), process water from construction areas will overtop the construction area dikes and follow natural drainage patterns into the stormwater management system. Construction process water does require treatment for TSS removal, and conveyance through the stormwater system will reduce TSS in the proposed sedimentation ponds. The MPCA requires a temporary sediment control pond during construction be sized for the minimum of 2-year, 24-hour storm runoff or 1,800 cubic feet per acre draining to the pond, so the 5-year, 24-hour storm runoff volume would meet and exceed this requirement and provide additional storage and treatment in the event of a potential overflow. In addition, any overflow to the stormwater management system will be treated in the stormwater sedimentation ponds for TSS up to the 100-year flood volume for the stormwater system (see RS24).

In summary, the stockpile construction area dikes and ponds will only allow overflow to the stormwater system during storms in excess of the design storm, which was chosen as more conservative than that required as part of MPCA's stormwater permitting program.

5.0 Ore Handling Area Yield

This section describes the estimated range of process water volume from the Ore Handling Area located at the Mine Site and the preliminary design of a system to collect that drainage. Average annual precipitation values used to calculate the range of annual yield from the Ore Handling Area are presented in Section 3.1.1. Conveyance of all process water systems is discussed in Section 7.

The Ore Handling Area includes the Lean Ore Surge Pile and the Rail Transfer Hopper. The haul road ditches and process water ponds are also described in this section, because the majority of the runoff from these areas will flow by gravity to the area near to and associated with the Rail Transfer Hopper.

Surface runoff and drainage collected from the Lean Ore Surge Pile and the Rail Transfer Hopper which has contacted ore and lean ore is considered process water. Runoff from the haul roads will be considered process water, because the native surface material has been disturbed. Process water from the Ore Handling Area and the haul roads will be collected and routed to process water ponds to reduce the total suspended solids (TSS) and then pumped to the WWTF.

Figures 1.1-A through 1.1-E show the process water management systems that collect and convey water from each of these process areas at the end of Mine Years 1, 5, 10, 15, and 20, respectively. Process water from each of these areas will be kept separate from stormwater. Figures 1.1-A through 1.1-E in RS24 show stormwater management systems that divert water away from each of these process areas at the end of Mine Years 1, 5, 10, 15, and 20, respectively.

Total annual process water runoff volumes from the Ore Handling Areas are shown in Figure 5.0-A.

5.1 Lean Ore Surge Pile

Stockpile yield was described in Section 4. As described in RS49, one difference between the Lean Ore Surge Pile and waste rock stockpiles is that the Lean Ore Surge Pile is for temporary storage of lean ore, and the size and shape will change throughout the life of the mine. This surge pile will be completely removed and the area reclaimed at the end of Mine Year 20, as detailed in RS52. Because this surge pile will be completely removed at the end of mining, no cover system will be installed on the Lean Ore Surge Pile.

Unlike other stockpiles, the Lean Ore Surge Pile will continually have material added and removed. As described in RS49 and Section 4 of this document, the moisture content of the lean ore will normally be below field capacity, resulting in a large reduction (uptake) of precipitation reaching the liner, because the rock will be continually replaced with newly-mined rock. However, the stockpile yield from individual storm events will be much higher when a significant amount of the lean ore has been removed and the liner is not covered with rock. PolyMet has committed to maintain a nominal depth of rock over the liner to minimize damage to the liner and to aid in drainage interception and retention of the water by the newly-mined waste rock.

As shown in Figure 5.0-A, the annual process water volume from the Lean Ore Surge Pile was calculated for Mine Year 1, 5, 10, 15, and 20. Process water estimates are based on footprint areas from RS18, which indicates a full 40-foot lift on the footprint at the end of each time period. This results in a consistent annual process water volume between 56 and 74 acre-feet per year and an average annual process water rate between 35 and 46 gallons per minute. Process water from the Lean Ore Surge Pile will be collected in Sumps S-6 and S-7 prior to being pumped to the WWTF.

It should be noted that in addition to the figures presented in RS18, Table 3.A of that document also lists yearly Lean Ore Surge Pile balances. This table indicates that the Lean Ore Surge Pile varies significantly from year to year, including years of little to no ore on the liner. Operationally, it would be beneficial for PolyMet to maintain some amount of lean ore on the liner at all times for protection of the liner and to slow the rate of drainage to the sump. Removal of the ore to the Rail Transfer Hopper will be from the west to the east. When the surge pile is getting low, keeping the remaining material towards the south border would minimize the amount of water in the sump to the extent possible or delay water draining to the sump. As described in Section 4.1 for waste rock, the lean ore may also retain water due to its low moisture content compared to its field capacity (especially during the first few years).

5.2 Rail Transfer Hopper

As described in RS18 and the NorthMet Project Description, the Rail Transfer Hopper is used for loading ore into rail cars. The layout of this Rail Transfer Hopper consists of a raised platform on which haul trucks enter and exit the area and from which they dump ore into a hopper over a pan feeder, which drops the ore into rail cars, as shown in Figure 5.2-A. There will be a sloped concrete floor within the Rail Transfer Hopper, directing runoff to the south. The runoff will cross the rail spur on sloped concrete panels to a concrete-lined swale on the south side of the railroad spur. This concrete swale will be sloped to the west to allow process water to flow to the lined process water pond PW-3 prior to being pumped to the WWTF for treatment. Design of process water pond PW-3 is described in Section 5.4. Process water volumes from the Rail Transfer Hopper during individual storm events were assumed to equal 95 percent of precipitation, based on an SCS Curve Number of 98 for paved parking lots and storm sewers, which assumes some loss of precipitation to initial wetting, depression storage and evaporation. Annual process water volumes were estimated based on precipitation and surface runoff rates provided in Section 3.1.1.

Due to the nature of the work and probability of ore spillage, all surface runoff from the Rail Transfer Hopper will be considered process water except the side slopes that lead up to the raised platform, which will be vegetated. As shown on Figure 5.2-A, the proposed side slopes to the raised platform (haul approach area) are 1.5H:1V, so additional erosion protection may be needed to stabilize these slopes. Erosion protection could include reinforced slopes consisting of riprap revetment, turf reinforcement mats, erosion control blankets, or some other method to stabilize the slope during a major storm event.

Small ditches at the base of these vegetated slopes will collect surface runoff from these sloped areas for conveyance to the perimeter stormwater ditches (discussed in RS24).

5.3 Haul Roads

The haul roads will either be constructed to split surface runoff to both sides by crowning (peaking) the road in the middle of the road, or by directing all surface runoff to one side by super-elevating the high side. Depending on the height of these roads above the natural grade, ditches will either be built in the road section or adjacent to the road. These process water ditches will only collect surface runoff from the road cross-section, because stormwater from adjacent areas will be intercepted and re-directed before entering the road section. This may mean construction of parallel ditches in some areas, one for process water and one intercepting adjacent stormwater. This will minimize the size of the process water ditches and the amount of water requiring treatment from haul road drainage.

The quality of the water coming off haul roads will be related to the amount of ore spillage occurring on the roadways. Haul roads will generally be kept clear of material for safe travel of the vehicles and as part of best management practices; however, trucks will be traveling on these roadways with large amounts of Category 1/2, 3, and 4 waste rock and lean ore. Therefore, drainage from the haul roads will be collected in process water ditches to a lined pond. Water quality will be monitored at the pond to determine the nature of the water and the method of treatment needed.

Process water runoff from haul roads was calculated as 76 percent of precipitation for 100-year, 24-hour design storm, based on an SCS Curve Number of 89 for gravel roads with Type C soils. The ditches and process water ponds for the haul roads are designed to accommodate the 100-year storm.

Due to the small size of the catchment areas along the haul roads, these ditches will contribute an average annual volume of runoff between 57 and 75 acre-feet at an average annual runoff rate between 35 and 47 gallons per minute, depending on the Mine Year, as shown on Figure 5.0-A. Annual runoff rates were based on precipitation rates provided in Section 3.1.1.

Due to a natural topographic divide through the center of the Mine Site, haul road drainage will be directed to two separate process water ponds. The majority of the road drainage will be collected in process water pond PW-4, west of the Rail Transfer Hopper. The road drainage east of and along the access road into the Mine Site, immediately east of the Lean Ore Surge Pile, will be directed to the process water pond PW-2, located south of Dunka Road and the railroad. All drainage from the haul roads will be directed to these process water ponds prior to being pumped into the pipe system to the WWTF. The design of these haul road ponds is discussed below in Section 5.4.

In some cases, haul road runoff may be directed to the mine pit and included in mine dewatering.

5.4 Process Water Pond and Sump Design

As described above, three process water ponds (PW-2, PW-3, and PW-4) and two sumps (Sump S-6 and S-7) will be constructed to collect drainage from the Ore Handling Area for conveyance to the WWTF. Preliminary design data for the process water ponds and sumps are listed in Table 5.4-A. The primary purpose of the process water ponds and the sumps is to provide storage for gravity flow of process water volumes and to minimize the pump capacity and cycling. In designing these systems for containment of the 100-year, 24-hour storm yields, the process water ponds for the haul roads (PW-2 and PW-4) and the Rail Transfer Hopper (PW-3)[3] will have the added benefit of reducing total suspended solids (TSS), which will limit the amount of sediment in the pumping and piping system. The primary purpose of the sumps is to provide storage for gravity flows, moderate the pump flows, and reduce the cycling of the pumps, although some sediment will be removed by their size and retention of the water.

As described in Section 4.1.5, construction of a lined sump or pond requires adequate foundation drainage to prevent excessive pore pressure from developing under the liner. Although there are no wetlands mapped in the location of these three proposed ponds, additional geotechnical investigation and hydrologic monitoring is needed to determine the depth of groundwater once foundation grades are established during final design. If groundwater or surface water is expected to remain near the surface once pit dewatering begins, the storage volume necessary for flood attenuation could be affected by groundwater levels. This may require the ponds and sumps to be constructed partially above the surrounding ground surface by constructing dikes. Although more expensive, other design approaches that may be used, if required, include excavation of the underlying material down to a suitable grade and rebuilding the foundation with a well-drained material, installing underdrains to facilitate a dry foundation, constructing dikes or trenches with subsurface leakage control measures, as described in RS25, or some combination of these methods.

5.4.1 Process Water Ponds

Design of the process water ponds PW-2, PW-3, and PW-4 is based on the criteria established to contain runoff by gravity from the 100-year, 24-hour storm event, resulting in required storage volumes of 8.3, 4.2, and 17.4 acre-feet, respectively, based on runoff coefficients discussed in Sections 5.2 and 5.3. Storage of this water will have the added benefit of reducing TSS concentrations in the process water.

In general, the ponds will be partially excavated and partially filled above the natural ground, designed to ensure that the pond bottom is above the expected groundwater elevations once pit dewatering begins.

Due to the nature of the runoff water, the Rail Transfer Hopper process water pond PW-3 will be constructed with the same liners that have been designed for the Category 4 stockpile sumps, as described in Section 4.1.5.2. The haul road process water ponds PW-2 and PW-4 will be constructed with the same liner as designed for the Category 1/2 stockpile sumps, described in Section 4.1.5.1.

The pond dikes and slopes will be vegetated to limit erosion. The pond dike design will be conducted once the foundation grading design is completed and pond elevations can be established. The pond elevations will allow the collection ditches to be conveyed by gravity into the ponds, and will be low enough so that additional storage can be provided above the ground. This additional storage also increases sediment-trapping efficiency. The outlet for these ponds will be a pump and piping system to convey this process water to the WWTF, as described in Section 7.

5.4.2 Lean Ore Surge Pile Sumps

The Lean Ore Surge Pile is different from the waste rock stockpiles because it will likely have periods of open liner throughout the mine operations. Due to the potential for an open liner on the Lean Ore Surge Pile, the two sumps associated with this surge pile (Sumps S-6 and S-7) have been designed with more overall capacity than the waste rock stockpile sumps. This was achieved by increasing the yield coefficients used in sizing the sumps for the Lean Ore Surge Pile to 100 percent of precipitation to reflect the potential for these periods of open liner, which will increase the quantity and timing of runoff from within the footprint.

These two sumps were designed to contain the entire precipitation volume from an open liner during the 10-year 24-hour event. Perimeter dikes around the sumps will connect the stockpile foundation and liner to the sump to contain the 100-year 24-hour precipitation volume from an open liner in the drainage layer of the stockpile liner, as shown in Figure 4.1.5.2-A.

5.4.3 Ore Handling Area Pond and Sump Overflow Contingencies

The process water from the Ore Handling Areas will all likely require treatment to meet water quality standards. The design storm for these facilities, the 100-year, 24-hour event, only has a 1 percent chance of being exceeded in any given year, or an 18 percent chance of being exceeded during the 20-year life of the Mine Site. Although these facilities have been designed according to a significant design storm, there may be occasions during the life of the mine that the design storm is exceeded, resulting in runoff exceeding the capacity of the facilities. Contingencies have been developed to minimize environmental impacts in the event a larger storm occurs.

For storm events in excess of the design storm, process water from Ore Handling Areas will overtop the dikes surrounding the process water sumps and ponds. The pumping networks draining these sumps and ponds were only sized for the snowmelt event; therefore pumping must be increased through a second pump system. Although it would not be cost-effective to have a second permanent pump and pipeline network in place in the event of an extended power outage or larger storm event, an emergency operating procedure has been developed to manage process water under these circumstances. This contingency plan includes a plan of action for the Ore Handling Area sumps and ponds. This plan of action includes use of the road dewatering trucks with temporary diesel pumps to operate during events greater than the design event or under circumstances of extended power outages associated with heavy rainfall. This plan will maintain water levels below the capacity of the sumps and ponds, pumping to either a tanker truck or a temporary tank for storage in the tank or pit until process water volumes are down to manageable levels.

Under circumstances of design events exceeding sump and pond capacity or extended power outages during heavy rainfall, it is likely that the WWTF may reach capacity and shut down the pumping network leading to it. In these circumstances, pumped process water may be temporarily pumped into the pits, with mining operations in the lower levels temporarily shut down until water in the pit sumps are back to manageable levels. Water will continue to be pumped from the Ore Handling Area sumps and pumps, prioritized according to the level of reactivity, as follows: Lean Ore Sumps S-6 and S-7 and process water ponds S-5 (Category 3 Lean Ore and Category 4 overflow pond), PW-3 (Rail Transfer Hopper runoff), PW-4 (haul road runoff), and PW-2 (haul road runoff), in that order. Pit dewatering may be temporarily stopped during these conditions to allow lowering of the water in these sumps and process water ponds to manageable levels.

In the unlikely event of rainfall greatly exceeding the 100-year storm event and containment under the emergency contingency plan, all overflows from the Ore Handling Areas would exit the Mine Site from stormwater sedimentation ponds, ultimately flowing to the Partridge River. Under these conditions, process water overflow from these systems would occur as follows:

- ***Lean Ore Surge Pile Sumps S-6 and S-7*** overflow would flow into the stormwater networks south of the sumps. Overflow from S-6 would drain by gravity to the stormwater sedimentation pond OS-4 and be conveyed from the site. Overflow from S-7 would drain by gravity to the stormwater sedimentation pond OS-7 and be conveyed from the site.

- ***Rail Transfer Hopper Pond PW-3*** would overflow into the stormwater network south of the pond, be conveyed to stormwater sedimentation pond OS-4, and exit from the site.

- ***Haul Road Pond PW-4*** overflow would overflow into the stormwater network south of the pond, be conveyed to stormwater sedimentation pond OS-4, and exit the site.

- ***Haul Road Pond PW-2*** would overflow into the stormwater overflow ditch from stormwater sedimentation pond OS-7 and be conveyed around the northeast side of the railroad tracks, exiting the site.

6.0 Overburden Area Runoff

This section describes the estimated range of process water volume and the collection and conveyance of runoff from the overburden storage area and overburden portion of the Category 1/2 Stockpile including design of process water ditches and ponds. Average annual precipitation values used to calculate the range of annual runoff volume from the Overburden Area are presented in Section 3.1.1. Conveyance of all process water systems is discussed in Section 7.

Even though overburden is native surface material from other portions of the Mine Site, surface runoff from Overburden Areas is considered process water because the native surface material has been disturbed. Runoff from unreclaimed Overburden Areas should not require treatment for dissolved substances and will be routed to the Tailings Basin or may be used for the filling of the East and Central Pits in Years 12 through 20, as described in Section 3.3. Runoff from reclaimed surfaces of the overburden portion of the Category 1/2 Stockpile is considered stormwater and will be routed into the natural system to the Partridge River.

As described in RS18, there are two separate areas designated for storage of overburden material at the Mine Site:

1) the Overburden Storage and Laydown Area is located immediately west of the Rail Transfer Hopper, and

2) the Overburden Portion of the Category 1/2 Stockpile.

The majority of surface water flowing to these areas from the surrounding watershed will either be captured in stormwater ditches that direct the flow away from the areas or will be diverted away from the area by dikes. Surface runoff from the unreclaimed sections of the overburden portion of the Category 1/2 Stockpile and the Overburden Storage and Laydown Area (i.e., process water) will be captured and directed to process water ponds for flood storage and reduction of TSS, and pumped through a pipe network to the CPS. Figures 1.1-A through 1.1-E shows the development of the Overburden Areas at the end of the mine Years 1, 5, 10, 15, and 20, respectively.

6.1 Overburden Storage and Laydown Area

The Overburden Storage and Laydown Area is a temporary storage stockpile with a temporary screening and sorting area, all of which will be removed at the end of mining operations as described in RS18 and RS52. Surface runoff towards this area from the north and west sides will be captured in a stormwater ditch prior to entering the operations area and will be directed south along its natural drainage route. Surface runoff from this storage area will be intercepted by two process water ditches that surround the area and will flow into process water pond PW-1 at the southwest corner of the laydown area. The Overburden Storage and Laydown Area is projected to have a total annual average runoff volume between 0 and 87 acre-feet, changing over time as shown in Figure 6.1-A. Average annual runoff volumes are based on precipitation values provided in Section 3.1.1.

Runoff coefficients of 61 and 66 percent of precipitation was used for the active (uncovered) sections of the Overburden Storage and Laydown Area for the 10-year, 24-hour storm event and 25-year, 24-hour storm event, respectively, based on an SCS Curve Number of 87 for dirt roads with compacted Type C soils. The expected mixture of rock and soils should have characteristics similar to Type C soils.

6.2 Overburden Portion of the Category 1/2 Stockpile

Overburden will be stockpiled in a separate portion of the Category 1/2 Stockpile. Surface runoff from active (uncovered) areas on the overburden portion of the Category 1/2 Stockpile will be routed to process water ditches along the east side of the stockpile to a process water pond PW-7. The outlet from this process water pond will be a pump and pipe system to convey this water to the CPS.

Surface runoff from the sides of reclaimed portions of the Category 1/2 stockpile will be routed along the benches to riprap-lined channels down the sides of the stockpiles to the stormwater ditches along the toe of the stockpile. With this design, only runoff from active (uncovered) sections of the stockpile will be collected as process water, minimizing the volume of process water sent to the CPS and minimizing the reduction in flows to the Partridge River.

The total annual surface runoff volume from the overburden portion of the Category 1/2 Stockpile is projected to be about 95 to 120 acre-feet, progressing as shown in Figure 6.2-A. Average annual surface runoff volumes are based on precipitation data provided in Section 3.1.1.

Runoff coefficients of 61 and 66 percent of precipitation were used for the active (uncovered) sections of the overburden portion of the Category 1/2 Stockpile for the 10-year, 24-hour storm event and 25-year, 24-hour storm event, respectively, based on an SCS Curve Number of 87 for dirt roads with compacted Type C soils.

6.3 Ditch Design

Process water ditches associated with the Overburden Areas include two ditches around the Overburden Storage and Laydown Area and one ditch around the Overburden Stockpile. Channel velocities in the ditches will be controlled with a combination of vegetation and riprap, depending on the expected velocities. Drop structures or some other engineered design will be used in areas with slopes greater than 3 percent to reduce the gradient and limit erosion. Riprap or other standard Best Management Practices will be installed in ditch sections where velocities are greater than 4 feet per second to limit the potential for erosion. Because the surface runoff from these areas should not require treatment for anything other than TSS, these channels were designed as trapezoidal open channels with capacity to contain the 10-year, 24-hour event.

Construction techniques used to construct these ditches may dictate a larger channel than is necessary for hydraulic capacity. Ditches with a bottom width smaller than 3 feet would be difficult to construct and maintain with heavy equipment. Ditch design parameters based on hydraulic capacity include:

- Preliminary design of the process water ditches for the Overburden Storage and Laydown Area indicates a bottom width of 3 feet with 3H:1V side slopes, and ditch slopes between 0.3 and 1 percent, based on existing topography. Ditch sections with slopes between 0.4 and 1 percent will require riprap up to 9 inches in diameter. Peak flows were calculated based on the Rational Method with a runoff coefficient (C) of 0.7. The rainfall intensities were selected from the Minnesota Department of Transportation (MNDOT) Intensity-Duration-Frequency (IDF) curves for the 10- and 100-year storm event in Zone 3 which covers Northeast Minnesota (MNDOT, 2000). The 10- and 100-year peak flow rates for the west ditch were estimated to be about 58 and 94 cfs, respectively, and the flows in the ditch along the east side of the storage area were estimated to be approximately 29 and 47 cfs for the 10- and 100-year peak flows. The designed ditch capacity is greater than expected flows for the 100-year peak flows; however, reducing the ditch cross section results in potential constructability issues. The channel capacity parameters may be revised in final design to reduce velocities, limit erosion potential, or for economic or constructability reasons.

- The preliminary design of the process water ditch for the overburden portion of the Category 1/2 Stockpile includes a 3-foot bottom width with 3H:1V side slopes, and a minimum ditch grade of approximately 0.1 percent. The expected maximum flows were evaluated for the ditch along the east side of the stockpile. The 10- and 100-year peak flow rates for the east ditch were estimated to be 94 to 251 cfs, respectively. The designed ditch capacity is greater than expected 10-year peak flows, although the 100-year peak flows will likely overtop the ditch (see Section 6.4.1). The channel slope, width, and side slope parameters may be revised in final design to reduce velocities, limit erosion potential, increase capacity, or for economic or constructability reasons.

6.4 Process Water Pond Design

Two process water ponds will be constructed to provide flood storage for process water runoff from the 25-year, 24-hour storm event from the Overburden Storage and Laydown Area (PW-1) and the overburden portion of the Category 1/2 Stockpile (PW-7) prior to pumping to the CPS. Due to the nature of the contributing materials, none of these ponds need to be lined. The preliminary design parameters of these process water ponds are listed in Table 6.4-A.

Construction of ponds for flood storage requires that the storage volume be available when a precipitation event occurs. Therefore, if groundwater or surface water is expected to remain at or near the surface in the location of the proposed pond, the additional storage volume needs to be obtained at higher levels. Further geotechnical investigation and hydrologic monitoring is needed during final design of PW-1 and PW-7 due to existing wetlands mapped in their proposed locations. This information will be used along with the foundation elevations to define the pond elevations. Although more expensive, other design approaches that may be used, if necessary, include excavation of the underlying material down to a suitable grade and rebuilding the foundation with a well-drained material, installing underdrains to facilitate a dry foundation, constructing dikes or trenches with subsurface leakage control measures, as described in RS25, or some combination of these methods.

In general, these process water ponds will be partially excavated and partially filled above the natural ground, depending on expected water levels, bedrock elevation, and storage requirements. The pond dikes and slopes will be vegetated to limit erosion. The pond dike design will be conducted once the stockpile grading design is completed and pond elevations can be established. They will be designed such that the process water ditches will be able to flow by gravity from the channels into the respective ponds, and additional storage can be provided above the ground. This also increases their sediment-trapping efficiency.

6.4.1 Overburden Area Pond Overflow Contingencies

According to the assumptions made of the water quality, process water from overburden areas will only require treatment for TSS to meet water quality standards, which makes it no different than construction stormwater on other construction sites that are discharged into natural systems following treatment for sedimentation. However, there are safeguards in place, as discussed in Section 6.4, to collect and convey overburden process water runoff to the Tailings Basin rather than diverting it into the natural stormwater system. The reason this water is collected is due to the proposed reuse/recycle strategy, with no process water discharge proposed to surface waters of the state under normal operations. This water is considered process water from active sections of the overburden portion of the Category 1/2 Stockpile and from the Overburden Storage and Laydown Area, because it has contacted disturbed surfaces and may not meet water quality limits for TSS.

The design storm for the overburden pond, the 25-year, 24-hour event, only has a 4 percent chance of being exceeded in any given year, or a 56 percent chance of being exceeded during the 20-year life of the Mine Site. Although these facilities have been designed according to an appropriate design storm given the nature of the water, there may be occasions during the life of the mine that the design storm is exceeded, resulting in runoff exceeding the capacity of the facilities.

For storm events in excess of the design storm (25-year, 24-hour event), process water from overburden areas will overtop the overburden pond dikes and follow natural flow patterns into the stormwater system. Overburden process water does require treatment for TSS removal. The MPCA General Permit for Construction Activity requires a temporary sediment control pond during construction be sized for the minimum of 2-year, 24-hour storm runoff or 1,800 cubic feet per acre draining to the pond for treatment of TSS, so the 25-year, 24-hour storm runoff volume would exceed this requirement and provide additional storage and treatment in the event of a potential overflow. Furthermore, any overflow to the stormwater management system will be treated in the stormwater sedimentation ponds for TSS up to the 100-year flood volume for the stormwater system (see RS24). In summary, the overburden area dikes and ponds will only allow overflow to the stormwater system during storms in excess of the design storm, which was chosen due to the low level of expected water quality impacts associated with the process water from this source.

Overburden process water pond PW-7, which is only in place until the footprint of the Category 1/2 stockpile is complete by Year 10, would overflow to the West Pit in storm events in excess of the 25-year 24-hour event. Overburden process water pond PW-1 would overflow into stormwater pond OS-5 and outlet to the Partridge River during storm events in excess of the 25-year event.

7.0 Process Water Management Plan

The process water management system was developed for five different Mine Years 1, 5, 10, 15, and 20, which correspond to Figures 1.1-A through 1.1-E. The pumping systems are portrayed on a separate set of Mine Plan drawings, Figures 7.1-A through 7.1-E. For each Mine Year, the total quantity of process water was calculated for each of the particular areas of the Mine Site, as described in the previous sections. This section describes how those flows are routed in the process water conveyance systems to the appropriate locations.

7.1 Conveyance System Alignment

Process water needs to be collected from each of the stockpile and pit sumps and ponds and conveyed to the WWTP, the CPS, or to the East and Central Pits during pit filling operations. Pipelines will collect process water from various sources based on similar quality of water, similar conveyance locations, and similar destination. The quantity of water routed to each destination changes each year. Figure 7.1-F shows the overall process water balance components for the Mine. Figure 7.1-G shows the quantity of process water to be treated at the WWTF each plan year by source.

The preliminary design categorized the process water into six different pipelines, as follows:

1. Category 3 and 4 stockpile footprint construction water, and surface runoff from active portions of the Overburden Storage and Laydown Area (Note: haul road runoff volumes were included in this pipeline as part of a contingency plan);

2. East Pit, Central Pit, and West Pit water from the northeast sump, and haul road runoff;

3. Category 3 and 4 active stockpile runoff and liner drainage, Lean Ore Surge Pile runoff and liner drainage, and Rail Transfer Hopper runoff;

4. West Pit water from the southwest sump;

5. Category 1/2 active stockpile runoff and liner drainage; and

6. Category 1/2 stockpile footprint construction water and runoff from active areas of the overburden portion of the Category 1/2 stockpile.

Pipelines 1, 2, and 3 will be routed along the north side of Dunka Road. These pipelines can be sized with extra capacity in case water quality concentrations indicate other preferred combinations. Pipelines

5 and 6 will be routed around the west side of the West Pit, and Pipeline 4 will be routed south, directly to the WWTF. These alignments needed to account for the location of the stormwater management infrastructure around the Mine Site. Figures 1.1-A through 1.1-E of RS24 illustrate the layout of the stormwater management infrastructure. Figure 7.1-H is a conceptual cross section along Dunka Road from the Category 3 Lean Ore Stockpile to the railroad to illustrate the layout of the stormwater ditches in relation to the process water systems. As shown on Figure 7.1-H, a containment berm will be constructed between the process water pipes and the stormwater ditch to contain any process water in the unlikely event that pipe leakage were to occur.

As shown on Figures 7.1-A and 7.1-B, the two pipelines conveying construction and overburden water (Pipelines 1 and 6) route process water directly to the CPS. The remaining four pipelines route process water to the WWTF, then to the CPS.

As described in earlier sections, the flows that were used to size the pumps and pipes were based on historic snowmelt runoff data from the Partridge River gage upstream of Colby Lake, and assume a spring snowmelt event of 3.2 inches from a combination of snowmelt and rainfall. The snowmelt event is the critical storm event in this area when considering total annual volumes. As discussed in earlier sections of this document, the process water ponds and sumps all have storage capacity to hold process water yield volumes by gravity flow that result from larger storm events. The stockpile sump pumps were designed based on a 30-day snowmelt, which is typical for the Mine Site region. The pit sump pumps were designed based on the assumption that 40 percent of the snowmelt (1.28 inches) could occur within one day, and removal of this water is required within 3 days to alleviate delays in operations.

These pipe and pump systems have been sized to maintain velocities less than 5 feet per second (fps) to minimize friction losses and surge pressures (water hammer) in the pipes. Pipe and pump size calculations were based on use of high density polyethylene (HDPE) pipe, which is currently available to PolyMet. This type of pipe can tolerate acidic water and is preferred in this environment due to its ability to tolerate freeze and thaw cycles. Pipes of other materials may be used in some segments, on a limited basis, but HDPE is preferred for the majority of the pipes. These pipes will generally be placed on top of the ground with fill placed on top of some sections of the pipe to add stability and additional ultraviolet radiation protection. Fill will be comprised of overburden material or LTVSMC tailings. Additional discussion of pipe material selection is included in Section 7.2.1.

Winter operations of the pumps and pipes are expected to be minimum, because there will be very little drainage in the winter. Any process water that reaches sumps and ponds will be monitored and allowed to collect until pumping is required. During winter operations, pipelines will typically be drained after each use to alleviate potential freezing.

Check valves and flow diversion structures will be designed at the junctions of some pipe networks to maintain flexibility in the direction of flows, depending on measured water quality levels and whether treatment is required. These diversion structures will likely be concrete manholes that will allow various pipeline connections.

7.2 Conveyance System Design

The location of conveyance pipes and the locations and names of pumps and sumps are shown on Figures 7.1-A through 7.1-E. Figure 7.2-A shows the average annual flows within each of the six pipelines discussed above.

7.2.1 Pipeline Material

The pipes used for stockpile drainage and pit dewatering will be comprised of high density polyethylene (HDPE) material, which is highly suited for this application due to chemical compatibility, durability, and its resistance to the effects of local weather. HDPE pipe generally has a life expectancy of approximately 50 years when transporting water. Potential reasons for degradation of the pipes could include freeze-thaw cycles, ultraviolet radiation (UV), water hammer (pressure pipes), structural integrity, and shear force. Each of these issues will be discussed individually, along with the planned methods of protection of the pipelines against these elements.

Freeze-Thaw cycles often deteriorate material either through absorption of liquids and expansion from ice formation or from freezing temperatures making the material brittle, and then cracking the material during thaw. According to ISCO, a pipe manufacturer, and several other HDPE suppliers, HDPE gains strength during lower temperatures and will not become brittle during freezing temperatures. The pipe will expand and contract to avoid breaking, even when frozen solid with water. It is ideal for use at sub-zero temperatures, with a brittleness temperature of less than -180 degrees.

UV Radiation could typically reduce the tensile properties of plastics over time. According to Zeus, a supplier of high performance fluoropolymer tubing, HDPE is regarded as fair with respect to UV-degradation with direct exposure. Additional protection can be obtained using carbon black, an additive used in protecting polyethylene from weathering. PolyPipe, a pipe manufacturer, states that their black HDPE pipe is suitable for long-term above ground storage with a 50-year maximum storage time, with long-term service in aboveground applications. However, the best form of protection against UV-radiation is obtained by covering the pipe. The pipe at the Mine Site will generally be placed on top of the ground with fill placed on top of some sections of the pipe to add stability and additional UV-radiation protection.

Water Hammer is one of the most common causes for breaks or leaks in piping. HDPE pipe has high elasticity properties which reduce surge pressures caused by constant changes in the forces within the pipe that cause water hammer. The flexibility of the material allows a water hammer to move through the pipe, buffering itself as the pipe grows from the shock waver. Additionally, HDPE pipe sections are commonly fused together, reducing the potential for leaks or bursts at joints, where failure commonly occurs in piping. According to ISCO and PolyPipe, the elasticity of HDPE pipes allow an increase in ratings for repetitive surge pressures up to 1.5 times the rated working pressures used for design of the pipe and occasional surge pressures up to 2 times the design operating pressure.

Structural Integrity can also be affected by continuously moving the pipes, which causes wearing of the pipe material. Once a pipe loses approximately 10 percent of its wall thickness, its structural integrity is at risk of failure. Most HDPE pipes are scratch and abrasion resistant, but minimizing movement of the pipe is the most effective solution to this problem. The majority of the pipes at the Mine Site are set in place once, with little or no movement required through the life of the Mine. The main exceptions to minimizing movement are with the pit dewatering pipes. A thorough quality assurance, quality control (QA/QC) inspection program will be established to inspect pipes during and following a move to identify problems with structural integrity of the pipes prior to use of the pipe. If a section of the pipe is identified as having a significant loss approaching 10 percent of its wall thickness, immediate repairs will be made to field-fuse the pipe prior to use.

Shear Force is the last potential circumstance that could cause a pipe failure or leakage. Shear force would include a major impact to the pipe either by something running into the pipe or falling on the pipe (i.e., struck by a vehicle or falling rock). These pipes will generally be set between the stockpiles and stormwater ditches in most locations. The safety berms on Dunka Road and the stormwater ditches adjacent to Dunka Road would separate major traffic from pipes paralleling Dunka Road. Safety berms and roadway ditches along the haul roads will protect the pipelines that parallel the haul roads. Additionally, the stockpile safety berms, 30 foot benches, liner berms, and stormwater and process water ditches along the stockpiles will aid in protection of pipelines against falling rock. Unlike other circumstances, shear force is the one issue that could cause a problem for pipe breakage at the Mine Site that can not be completely mitigated, although the small diameter of the pipes will also aid in protection against shear force.

7.2.2 Pipeline Contingencies

The Category 3, 3 Lean Ore, and 4, Lean Ore Surge Pile, and Rail Transfer Hopper process water are all captured in one pipe (Pipeline 3). As discussed in Section 7.2.3.3, this pipe ranges in size from 3 to 8 inches, with a cumulative pipe length of just over 4 miles, and conveys the most reactive water from the Mine Site to the WWTF. The East, Central, and West Pit process water, carried in Pipelines 2 and 4, also requires treatment for reactivity; Pipeline 2 runs parallel to Dunka Road and Pipeline 3. Pipeline 1, also running parallel to Dunka Road, only contains water from the stockpile construction areas, overburden areas, and potentially, the haul roads. An analysis was performed to evaluate the cost-effectiveness of secondary containment for these process water pipelines adjacent to Dunka Road, as requested.

There is no regulatory requirement or guidance for providing secondary containment for pipelines of this nature. This analysis evaluated the use of a secondary containment pipe, lined ditches, or unlined ditches. The advantages, disadvantages and costs of each option are discussed below.

7.2.2.1 Secondary Containment Pipe

A secondary containment pipe could be used to contain all three pipelines along Dunka Road (Pipelines 1, 2, and 3) into a larger pipe to contain any leakage. This pipeline would range from a 12 to 36 inch pipe to hold all three pipelines and contain the volume from a break in all three pipelines. This option would require less maintenance than a lined ditch and would affect a smaller area than the lined ditch. It would contain a leak from the pressurized pipes much better than a lined ditch would.

There are several disadvantages of this option. It would be difficult to determine which of the three pipelines was leaking, although each of them would have metered flow. The containment pipe would need to drain to a manhole, several of which would have to be located along the alignment to allow discharge from a potential leak. Narrowing down the exact location of the leak would be difficult without fully dismantling the containment pipe between manholes, requiring temporary shutdown of all of these pipe systems. This option would not mitigate the affect of shear force, which is the circumstance with the most potential for pipe failure. In the event of shear force, a single containment pipe would likely increase the problem by combining all three pipes into one larger pipe, ensuring failure of all three during an impact.

The cost of a containment pipe is also a major disadvantage, ranging from $1.3 to $1.5 million depending on the need to contain the volume from a break in the largest pipe or the volume from all three pipes within the containment pipe. The cost of using double-walled pipes for all three pipelines individually is even greater than this option of an additional single-walled pipe containing all three.

7.2.2.2 Lined Ditches

A network of lined ditches could be used to contain the three pipes and any leakage that occurs from them. This option would reduce the concern of a pipe break due to force, because the water would mostly be contained within the lined ditch. With a 3-foot bottom width and a minimum slope of 0.1 percent due to bedrock, preliminary calculations indicate this ditch would be approximately 1.5 feet deep for both a single pipe failure and a failure of all three pipes.

Disadvantages to this alternative include significant maintenance of the ditch. The ditch would be exposed to precipitation, which would decrease the holding capacity. In the case of a rupture, a lined ditch may not contain the overspray from a pipe like a containment pipe would. For these two reasons, the depth of the ditch would need to be increased as a margin of safety.

The cost to construct a lined ditch would range from $530,000 for a depth of 1.5 feet to $1.6 million for a depth of 3 feet. These costs only include excavation and liner costs, not accounting for any blasting that might be necessary to achieve gravity flow or riprap or other erosion control measures that might be necessary along steep sections of this ditch.

7.2.2.3 Unlined Ditches

These three pipes could be located in a network of unlined ditches that would collect process water in case of pipe leakage. This could include excavation of the ditches or construction of a small berm to enclose the pipe between the stockpiles or other structures. This alternative does not provide significant protection other than providing additional separation between the process water flows and the adjacent stormwater flows. The maximum flow from a burst of all three pipes along Dunka Road would be approximately 2,800 gpm during an average peak flow during spring snowmelt, with flow from the largest pipe to be approximately 1,900 gpm. At these rates, it would take 8 to 11 hours to fill a ditch area 45 feet wide by 4 feet high by 1,000 feet long. The narrowest area along Dunka Road where all three pipelines run parallel is between the railroad spur leading from the Rail Transfer

Hopper and Dunka Road; there is approximately 45 feet between the toe of the railroad spur and the stormwater ditch.

The primary advantage to this alignment is the cost, which would range from about $130,000 for a single berm on one side to approximately $335,000 for excavation of an unlined ditch 3 feet wide, 1.5 feet deep with a slope of 0.1 percent. The option with a single berm would require use of adjacent slopes from the railroad or other structures.

The primary disadvantage is the lack of protection to groundwater in the event of a pipe burst. Road dewatering trucks with portable pumps could be used to remove any water that ponds up from a leak. The soil through this area, as described in Section 4.1.3.3 is primarily Type B, which has a moderate infiltration rate when thoroughly wetted; therefore during a major pipe rupture, there could be a loss of water to infiltration if the slope does route these flows prior to infiltration.

7.2.2.4 Pipeline Contingency Recommendation

Although the loss of process water to a pipe burst is possible in the 20 years of operations, mitigation measures will be installed to reduce the risk to a minimal level. The use of HDPE pipe itself alleviates many of the concerns of pipe burst. The remaining mode of rupture, shear force, has been mitigated to the extent possible by the placement of the pipes in areas away from major fall hazards. This could be further mitigated by covering the pipe in section that still represent some fall hazard (i.e., along the base of stockpiles). There are options to provide additional protection for a rupture, such as secondary containment pipes, lined ditches, and unlined ditches, but the cost of these options compared to the small possibility of a rupture makes these alternatives unreasonable. All the pipelines will have flow meters or pump horsepower meters monitoring the pumps at the collection point (ponds or sumps) and the outlet point (WWTP or CPS), so any leak in the pipes will be identified. All of the pipelines are along transportation corridors, so once a leak has been identified on a monitor or visually, pumps can be immediately shut down and repairs can be made. The excess cost of these secondary containment options make them cost-prohibitive for protection against minor leaks and may not provide adequate protection in the case of shear force, the event most likely to occur, albeit unlikely.

7.2.3 Individual Pipeline Designs

The design of each of the six pipes was evaluated on an individual basis as described in this section.

7.2.3.1 Category 3 and 4 Construction and Overburden Water Conveyance (Pipeline 1)

Process water will be collected from the overburden and waste rock stockpile construction water ponds, as described in Sections 6.4 and 4.2, respectively, and pumped to the CPS. The conveyance and pump system considered the Critical Year for flows from the various sources:

- The Critical Year for the Category 3 and 4 stockpile construction areas is Year 5, and the Category 3 Lean Ore stockpile construction area is Year 10.
- The Overburden Storage and Laydown Area does not change in size or dimension in Years 1 through 15.

The pumps and pipeline were designed for Years 5 and 10 to compare the overall system requirements and determine optimum pumping rates. The results are listed in Table 7.2.3.1-A.

Average annual flows in this pipeline range from 35 gallons per minute (gpm, Year 20) to 149 gpm (Year 1), as shown on Figure 7.2-A. Pipe sizes will range from 2 to 8 inches in diameter and 110 to 6,825 feet in length. Eight pumps will be required for this system in Year 5, and seven pumps will be required in Year 10: calculated pump sizes range from 0.5 to 25 horsepower (hp). The piping system will generally be placed along the southeast side of the haul road from the northeast Category 3 stockpile to Dunka Road. This branch will join the main pipeline at Dunka Road, conveying flows west along the north side of Dunka Road, and combining with flows from process water pond PW-1. The pipeline will convey these combined flows to the CPS pond for storage until routed to the Tailings Basin in the Treated Water Pipeline (Section 8). This pipeline also includes capacity for the haul road runoff ponds PW-2 and PW-4, which have Critical Years of 1 and 5, respectively.

7.2.3.2 Eastern Pit Water Conveyance (Pipeline 2)

The main line of Pipeline 2 extends from the WWTF east along the north side of the Dunka Road. Three major branches extend north to collect water from the East, Central and West Pits. Section 3.2 of this report describes Mine Pit dewatering, which included the sump, pump, and pipe sizes within the East and Central Pits through Year 20 and the west half of the West Pit for Years 5 through 20. Pump and pipe information from the pit rims to the WWTF are also discussed in Section 3.2. This pipeline also includes capacity for haul road runoff ponds PW-2 and PW-4, which have Critical Years of 1 and 5, respectively.

As described in Section 3.3, East and Central Pit filling begins in Year 12, which requires the majority of the runoff and groundwater from the East and Central Pits and periodically may require some additional water from the CPS. Pit dewatering may still be necessary on a seasonal basis to maintain the desired water level within 5 feet of the rock elevation during pit filling. The pipe and pump configuration on the Year 15 and 20 Mine Plan (Figure 7.1-D and 7.1-E) shows continual pumping from these pits for this reason.

7.2.3.3 Category 3 and 4 Stockpiles, Lean Ore Surge Pile, and Rail Transfer Hopper Drainage Conveyance (Pipeline 3)

This pipeline will collect and convey process water from the Category 3, Category 3 Lean Ore, and Category 4 waste rock stockpile sumps, the two Lean Ore Surge Pile sumps, the two haul road ponds, and the Rail Transfer Hopper process water pond for conveyance west to the WWTF.

The pipe sizes will range from 3 to 8 inches in diameter and the individual pipe lengths will range from 305 to 6,825 feet. Calculated pump sizes range from 3 to 40 horsepower. The pipeline will generally be placed along the southeast side of the haul road from the northeast Category 3 stockpile to join with the pipeline from the other stockpiles and placed along the north side of Dunka Road, and will pick up flows from process water ponds PW-2, PW-3, and PW-4. The pipeline will then convey these combined flows to the Stage 1 pond at the WWTF for storage prior to treatment and conveyance to the CPS.

Eleven pumps will be needed for this system during the life of the mine, one from each pond and sump and two intermediate pumps where flows combine. The Critical Year and preliminary pump and pipe sizes are summarized in Table 7.2.3.3-A. Average annual flows in Pipeline 3 range from 137 gpm in Year 1 to 250 gpm in Year 15.

7.2.3.4 Western Pit Water Conveyance (Pipeline 4)

As described in Section 3.3, East and Central Pit filling begins in Year 12, which requires the majority of the runoff and groundwater from the East and Central Pits and periodically may require some additional water from the CPS. Pit dewatering may still be necessary on a seasonal basis to maintain the desired water level within 5 feet the rock elevation during pit filling. The pipe and pump configuration on the Year 15 and 20 Mine Plan (Figure 7.1-D and 7.1-E) remains in the pits for this reason.

As discussed in Section 3.3, water quality estimates from RS31 show that the West Pit runoff and groundwater will need to be pumped to the WWTF for treatment prior to being pumped to the East Pit to aid in pit filling. Nevertheless, the pump and pipe network from the West Pit will be adjusted to pump pit dewatering east to combine with any East and Central Pit dewatering that is required. A pump and pipe network will still be maintained between the west cell of the West Pit and the WWTF to dewater the western half of the West Pit and in case of CPS pond overflow, as described in Section 8.1. Pumping out of the west cell of the West Pit after Year 12 will consist of three pumps, one in or adjacent to the pit sump, one intermediate pump to reduce pump head and pressures, and the last pump at the top of the pit rim to convey the process water to the WWTF. Pump and pipe information from the pit rims to the WWTF are also discussed in Section 3.2. Average annual flows in Pipeline 4 range from 62 gpm in Year 1 to 335 gpm in Year 20.

7.2.3.5 Category 1/2 Process Water Conveyance (Pipeline 5)

The Category 1/2 Stockpile process water will be routed from the south side of the stockpile around the west side of the West Pit, following the most direct route to the WWTF. The Critical Year for the Category 1/2 stockpile process water, or the year with the most process water during peak flows, is Year 5. The average annual flows, as shown in Figure 7.2-A, range from 206 gpm in Years 15 and 20 to 113 gpm in Year 1.

Table 7.2.3.5-A shows the preliminary specifications for pipes and pumps needed for Year 5 based on estimated flows developed in Section 4. Seven pumps will be needed in Years 1 through 20, ranging in size for the Critical Year from 3 to 25 horsepower, with pipe lengths between 545 and 3,180 feet.

7.2.3.6 Category 1/2 Construction and Overburden Water Conveyance (Pipeline 6)

The Category 1/2 construction and overburden footprint areas are only shown on the Mine Year 1 and 5 plans because the entire footprint is covered by the stockpile by Year 10. The pump and pipe system includes 3-4 pumps each mine year, and was therefore designed for both Mine Years, as listed in Table 7.2.3.6-A. The stockpile sumps will be the first area constructed, followed by the diking and a sedimentation basin within the diking to aid in process water collection and storage. The process water pipes for Category 1/2 construction and overburden water will run parallel and adjacent to the Category 1/2 stockpile process water pipeline. These systems will be kept separate because the construction and overburden water only needs treatment for suspended solids and will be sent directly to the CPS without any additional treatment.

The pipe sizes for this system range between 2 and 8 inches in diameter, with pipe lengths between 165 and 3,910 feet. Pumps for each system range between 2 and 15 horsepower with static head between 5 and 55 feet. Three to five pumps are required for this system for years 1 and 5, respectively. This system maintains alignment and junctions similar to Pipeline 5, which carries Category 1/2 process water. It may be beneficial to reduce this system to fewer pumps and pipes segments; this will be evaluated in final design during pump and pipe optimization (see Section 3.2).

Average annual flows within Pipeline 6 range from 130 gpm in Year 1 to 105 gpm in Year 5.

8.0 Central Pumping Station and Treated Water Pipeline to Tailings Basin

This section describes collection and conveyance of water from the Central Pumping Station (CPS) to the Tailings Basin. As described throughout this document, process water from the Mine Site will be routed to the CPS. Process water may require treatment for removal of metals or other substances prior to routing to the CPS or it may be routed directly to the CPS. All of the water from the CPS will be pumped through the Treated Water Pipeline (Pipeline) to the Tailings Basin, with the exception of any water needed during East and Central Pit filling operations, as described in Section 3.3. The flows in this system are expected to be continuous year-round, with lower flows during the winter months and during periods with low precipitation.

8.1 Central Pumping Station

The CPS is the collection point for process water from the Mine Site, located between the West Pit and the Dunka Road near the WWTF as shown in Figures 1.1-A through 1.1-E. The CPS consists of a pumping station structure and storage pond, as shown in Figure 8.1-A. A schematic drawing of the WWTF and CPS is shown in Figure 8.1-B.

Process water from the following sources will be routed directly to the CPS through various pipelines:

- Surface runoff from unreclaimed (active) portions of the overburden portion of the Category 1/2 Stockpile and the Overburden Storage and Laydown Area (Section 6)
- Surface runoff from cleared areas within the Mine Site (e.g. stockpile construction areas prior to placement of stockpile foundation, liner and waste rock) (Section 4.2)

Process water from the following sources that have been treated at the WWTF will also be routed to the CPS:

- Surface runoff from unreclaimed (active) portions of all waste rock stockpiles (Section 4.1),
- Liner drainage and leakage from all waste rock stockpiles (Section 4.1),
- Pit dewatering (Section 3.2),
- Surface runoff and liner drainage from the Lean Ore Surge Pile (Section 5.1),

- Surface runoff from the Rail Transfer Hopper (Section 5.2), and
- Surface runoff from haul roads (Section 5.3).

All water to be pumped by the CPS will be collected in an unlined one-half-acre storage pond. This water will include treated process water from the WWTF and water from the Mine Site that meets water quality limits without treatment. The storage pond will be approximately 12 feet deep with approximately seven feet of active storage (1.2 million gallons), two feet of dead storage at the bottom of the pond and three feet of freeboard. The active storage volume is sized to minimize pump cycling. The upper pond slopes will be vegetated to limit erosion.

The storage pond will be constructed with a high density polyethylene (HDPE) overflow pipe to the West Pit. This pipe will be installed at the maximum permissible water elevation in the storage pond and will be routed along the ground surface to the West Pit approximately 500 feet north of the CPS. The overflow pipe will allow for emergency storage in the pit in the event of an extended power/mechanical failure at the CPS, during maintenance or in the event of extreme storm events that exceed CPS pumping capacity. The location of the CPS was selected so that flow in the overflow pipe would be gravity-fed to the West Pit, thus providing emergency overflow protection without requiring action by personnel or availability of electric power.

The design discharge for the CPS is based on the rate of inflow of treated water from the WWTF and process water (i.e., overburden runoff) from the Mine Site. The inflow to the CPS will vary based on the season, storm events, climatic cycle, and the progression of mining. These flows are described in more detail in various areas of this report, and range from approximately 220 gpm (winter, Year 1) to 3,600 gpm (spring snowmelt, Year 10). As described in Section 3.3, there will also be periods after Year 10 when all available water from the site may be used for filling the East and Central Pits, and very little or no flow will be pumped to the Tailings Basin by the CPS. Based on the process water "high estimate" of approximately 3,600 gpm, the CPS will be designed to discharge a maximum of 4,000 gpm.

The preliminary design of the CPS calls for two vertical turbine pumps that will discharge to the Tailings Basin through the Treated Water Pipeline (Pipeline), a nominal 16-inch diameter steel pipeline, as described in Section 8.2. Figure 8.1-C shows the layout of the CPS. Under normal operating conditions, only one pump will be operating at variable speeds to discharge between 1,200 and 3,000 gpm to the Tailings Basin. The two pumps will alternate duty automatically after each pumping cycle. While the second pump provides redundancy and a maximum firm capacity of 3,000 gpm, the parallel operation of the two pumps also allows for the maximum pumping capacity of 4,000 gpm.

The maximum operating conditions for each pump are experienced when the discharge to the Tailings Basin is 4,000 gpm (i.e., 2,000 gpm per pump). Under these conditions, the total dynamic head (TDH) for each pump is approximately 575 feet. The static head changes over the life of the project as the elevation of the Pipeline discharge location changes. The static head through Year 10 is estimated at 150 feet, after which the static head will increase. Because the required flows decline after Year 10, Year 10 has been designated the Critical Year and was the basis for the CPS design. The friction loss in the Pipeline to the Tailings Basin accounts for approximately 425 feet of the TDH under maximum design conditions. The motor size required for maximum operating conditions is estimated at 450 horsepower (HP). Further analysis will be conducted during detailed design to determine if the use of existing 900 HP motors currently onsite would be more cost-effective than the purchase of new smaller motors.

The two pumps will be installed in a cast-in-place concrete pump station/intake structure that is connected to the storage pond. The pump station will be a trench-well type, below-grade structure, approximately 20 feet wide by 30 feet long by 15 feet deep. The motors, discharge piping and appurtenances, and control panels will be mounted on the at-grade top slab. The installation of the electrical control equipment will be further evaluated during detailed design. A fiberglass or steel enclosure for the control panels and motors may be required to provide weather protection for electrical equipment. Winter operation, in particular, will be further evaluated to determine if freeze protection would be required to allow pumping during the winter months. These considerations could be accommodated by housing the facilities in the WWTF.

The pump station inlet configuration will include a manually-cleaned bar screen, manually-operated inlet sluice gate, and associated equipment to keep the inlet free of ice during the winter. The bar screen will keep debris from entering the pump station and plugging the pumps. While large debris is not expected to collect on the bar screen, routine manual raking to clear woody debris may be required. The sluice gate will allow the station to be closed off from the pond and dewatered for maintenance or cleaning procedures. Dewatering of the station will be accomplished by installing a temporary portable trash pump in the station and discharging the water into the storage pond.

The inlet to the pump station from the storage pond will be submerged to minimize the effect of ice formation at the water surface, but additional ice control will be required for protection of the inlet sluice gate and bar screen. During detailed design development, three forms of ice control will be evaluated: 1) heat trace in the head wall of the pump station around the perimeter of the sluice gate, 2) recirculation of water at the inlet, and 3) aeration at the inlet. Each of these methods will be evaluated for their ease of operation, required maintenance, cost of equipment, and cost of operation. The best methods will be selected to achieve the desired ice control effects.

The pump discharge configuration will include air valves, check valves, and shut-off valves for each pump. The two pump discharge pipes will manifold into the single Pipeline downstream of the valves. Equipment for surge protection on the Pipeline will also be installed at the CPS location, as described below.

Level sensing equipment will start and stop the pumps based on the water level in the storage pond. In between the start and stop levels, the speed of the pumps will be varied to match inflow rates as much as possible in order to minimize pump cycling. The minimum discharge rate from the pumps will be approximately 1,200 gpm in order to maintain a minimum velocity of two feet per second in the Pipeline. This minimum velocity is required to keep any solids in the pumped water in suspension.

In addition to pump control, level sensing equipment will also be used to monitor alarm conditions such as pump failures, low water level, and high water level. A low water level condition would occur if the controls that shut down the pumps malfunctioned and caused the pumps to continue operating past the shut-down level. This condition could cause the pumps to run dry, which could result in damage to the equipment.

A high water level condition might occur if the pumps were to fail to operate or fail to keep up with inflow during large storm events. The high water level indicator will assist in control of the pumps feeding the storage pond from the Mine Site. When the high water level alarm indicates that the pumps are not keeping up with the inflow, the dewatering pumps could be temporarily shut off. As an alternative to shutting down the dewatering systems, mine personnel may choose to continue operating the pumps and allow basin inflow to be diverted to the West Pit through the overflow pipe. During detailed design, the amount of time required to evacuate the West Pit operations prior to allowing storage basin overflow will be considered, and the storage pond design may be adjusted as required. The communication system and destination for the alarm signals will also be determined during detailed design.

8.2 Pipeline to Tailings Basin

As described above, the CPS will discharge into the Pipeline for conveyance to Tailings Basin 1E, located about 8 miles west of the Mine Site. The Pipeline route, preliminary design considerations, installation, and operation are discussed in this section.

The alignment selected for the Pipeline is shown on Figure 8.2-A and has a total length of about 39,000 feet. It begins at the CPS at Station 0+00, following the north side of Dunka Road to Station 345+00, and then follows the north side of the rail spur to Station 360+00. At Station 360+00, the Pipeline leaves the rail spur and veers to the northwest towards Tailings Basin 1E. The Pipeline is then routed towards a high point near Station 375+00, from which it drops down into the Tailings Basin. The Pipeline will be designed so that it always discharges into the water in the Tailings Basin to prevent any potential erosion of tailings and the Tailings Basin dam that might occur if the discharge were to occur up on the beach section of the Basin.

The following criteria were used in selecting this route:

- The alignment follows an existing defined route that PolyMet has already obtained rights to as part of mine planning.

- The Pipeline will be next to Dunka Road, which is an existing established corridor with daily Mine traffic. This means that the Pipeline corridor will be under regular observation by many others. In the unlikely event that a leak should develop, it can be quickly identified and repaired. Leaks in municipal systems buried 8 feet deep quickly bubble at the surface in most soil conditions other than gravels and coarse sands. The tight soils along most of the Pipeline route should expose leaks at the surface rather quickly. In addition, flow meters at both ends of the Pipeline will quickly alert staff of any loss of fluid.

- Wetland impacts along this established route are not as great as along the other alignments considered.

- This route avoids some significant high points encountered along the corridor between the Mine Site and the Tailings Basin, which were encountered on one of the other routes considered.

- The alignment never crosses a major road planned for regular mine traffic or a rail line, minimizing the risk of structural failure due to surface loads from heavy mine vehicles or trains.

- The majority of this route is in areas already disturbed by previous activities in the area.

- This route provides easy access for operations, maintenance, and repairs of the Pipeline.

- Preliminary review of this alignment did not identify any major constructability concerns.

8.2.1 Alternative Routes Considered

Two alternative routes were considered during this phase of design, as shown on Figure 8.2.1-A and described in the following sections:

- Minnesota Power 138 kV Power Line Alignment, and

- Railroad Alignment.

8.2.1.1 Minnesota Power 138 kV Power Line Alignment

In reviewing possible routes for the Pipeline, it was initially believed that a route parallel to the existing Minnesota Power 138kV transmission line through the area would provide the most direct and cost-effective route. A preliminary review of the proposed route using topographic information provided by Minnesota Power revealed that the total length of pipeline required would be approximately 37,000 feet, and included an increase in elevation of approximately 190 feet (Elevation of 1,600 feet at the Mine Site versus a final elevation of 1,790 feet approximately 1 mile east of the Area 2 Shops). While this route may have lower estimated installation costs, uncertainties remained relative to the clearing, grubbing, and grading requirements for pipe installation, impacts to wetlands along the corridor (potentially as much as 3 miles of wetland impacts), and related environmental permitting. Given these concerns along with concerns about limited access for long-term monitoring and operation and maintenance, this route was eliminated from further review.

8.2.1.2 Railroad Alignment

Another route considered followed along the north side of the rail line that travels between the Mine Site and the Plant. The potential benefits of this alignment include smooth, gradual grade changes conducive to reliable Pipeline operation and the need for a minimal number of air release manholes. This alignment also follows a defined existing route. It also simplified draining the Pipeline, which was originally considered as part of normal operations of the Pipeline in the winter. The drawbacks of this alignment included:

- Total length of 42,000 feet, making it the longest of the three routes considered;
- Uncertain rights to access the route;
- Infrequent rail traffic for additional observations of the Pipeline;
- Limited access available for operation, maintenance, and repair; and
- Several unnecessary crossings of Dunka Road with the Pipeline.

These drawbacks were considered significant, so this alignment was eliminated from further consideration.

8.2.2 Pipeline Design

Conceptual design of the Pipeline was based on preliminary topographical data from the selected alignment, flow rates from the process water analysis, preliminary estimates of the ultimate height of the Tailings Basins, and other relevant data. The design of the Pipeline takes into account materials, flow rates, winter operations, and associated appurtenances.

8.2.2.1 Pipeline Material

There are many types of materials that will work for this Pipeline, and the final selection will be based on economics at the time of construction. The prices of the materials used in the manufacture of the pipes vary greatly with the price of energy and raw materials. The final decision on the type of material for Pipeline construction will occur closer to construction.

Preliminary calculations were made using steel as the assumed material of the Pipeline. Steel pipe is readily available, relatively easy to work with, and is often one of the most economical choices. Recently the price of steel has risen dramatically, making other choices more economically. Other benefits of steel include strength, long pipe lengths which reduce field joints, its widespread use in long pipeline applications, and the availability of people that know how to work on this type of pipeline, if needed.

The drawbacks of steel include a slightly greater friction factor than other materials considered resulting in higher pumping head and operating pressure, the susceptibility to corrosion, the rigidity resulting in higher water hammer, and the potential to collapse under vacuum conditions that can occur when uncontrolled water hammer is present. All of these issues are known, can be predicted with reasonable certainty, and can be accounted for in final design.

High density polyethylene (HDPE) is another material evaluated for the pipeline. If steel and HDPE prices were equal, HDPE would be the recommended material of construction for the Pipeline. This material has many of the benefits of steel, plus it has a higher durability in this type of climate with frequent freeze and thaw cycles. Another major benefit of HDPE is that scaling that frequently occurs with steel is rare with HDPE, decreasing the potential that the Pipeline would need to be cleaned during the life of the project.

Other materials considered included PVC, DIP, and fiberglass. Any of these materials could perform the required duty with the possible exception of PVC at the end nearest to the CPS where preliminary design pressures exceed PVC pressure ratings. If PVC were to be used, another material would be needed nearest the CPS with a transition to PVC once the system pressures dropped to an acceptable level. As the project nears final design and the construction phase, the price of all major pipe materials will be re-evaluated to determine the most appropriate selection at that time.

8.2.2.1.1 Pipeline Corrosion

If a steel pipeline is installed, corrosion protection will be important. Internal protection of the Pipeline will be accomplished by monitoring the Langelier Index of the water pumped at the CPS and adjusting treatment at the WWTF accordingly to maintain a slightly positive value. The Langelier Index is an approximate indicator of the degree of saturation of calcium carbonate in the water. A slightly positive value should result in the deposition of a thin calcium carbonate scale layer on the pipe wall over time. The water pumped from the CPS will be close to saturated with dissolved oxygen, so maintaining a positive Langelier Index will be important to protecting the Pipeline from internal corrosion. This will also mean regular cleaning of the Pipeline will be necessary to prevent excessive scale build-up, which could affect operations and limit Pipeline capacity. External corrosion will be dealt with either through sacrificial anodes of magnesium at specified intervals or an impressed current system, both of which will be evaluated in further design. If a non-ferrous material, such as HDPE, is selected for the Pipeline, no corrosion protection measures will be necessary.

8.2.2.2 Flow Rates and Pipeline Diameter

The Pipeline will be designed to handle flow rates from 1,000 to 4,000 gpm. The majority of the time, the Pipeline is expected to operate between 1,000 to 2,000 gpm. The design range allows the highest predicted flow of process water to be conveyed to the Tailings Basin plus a factor of safety of 10%. In order to accommodate this range of flow rates, a nominal pipe diameter of 16 inches has been selected. Smaller pipe diameters result in a significant increase in pumping head at the higher flows, and larger pipe diameters result in unacceptably slow velocities at the lower flow rates. Pipeline velocities will vary from 1.8 feet per second (fps) at 1,000 gpm up to 7 fps at 4,000 gpm. These pipe velocities are considered acceptable by current design standards. However, if PVC is considered for pipe material, it may be necessary to increase the diameter to ensure that pumping pressures do not exceed the rated pressure of the pipe material. PVC has thinner walls than other plastic pipes and lower working pressures, which must be accounted for in final design if this material is selected.

Preliminary head loss calculations were performed for a steel pipe using a Hazen-Williams C factor of 125. Documents reviewed indicate that new steel will have C values near 140, but as the pipe ages, the C value will diminish. References showed values ranging from 100 to 140 for use in design. The peak flow rate carried by the pipe will occur in Mine Year 10, and some reduction in the C value is anticipated by this time. If steel is used, pipeline cleaning will be required in order to prevent the C value from dropping to 100. Using this value, the dynamic head for the system will vary from 33 feet at 1,000 gpm to 425 feet at 4,000 gpm. Static head in the system will be constant through Mine Year 10 at 150 feet. After Mine Year 10, the static head will rise slowly along with the elevation of the Tailings Basin, but the process water flows will decrease as the East and Central Pits are filling, as described in Section 3.3. The maximum total design head (static plus dynamic) occurs in Mine Year 10 when a maximum flow rate of approximately 3,600 gpm is needed resulting in a total design head of 575 feet.

In addition to normal operating pressures, water hammer must also be evaluated, especially for long pipelines constructed of rigid material. For a steel pipeline, there is potential for a maximum water hammer surge of 380 pounds per square inch (psi) above operating pressures if a power failure were to occur or a valve were to be operated too quickly when the Pipeline is flowing at 4,000 gpm. Water hammer values will be considerably lower for more flexible materials such as HDPE or PVC. In order to prevent regular occurrences of water hammer, valve operation will be controlled. Other permanent water hammer devices will also be installed at the CPS and along the Pipeline route where needed to account for the possibility of a power failure. Devices could include air/vacuum valves, check valves to prevent flow reversal, and surge tanks.

The thickness of the pipe wall will be selected during final design once the material is selected. If a steel pipeline is installed, the wall thickness will likely be either 0.375 inches or 0.5 inches depending on specific pressure characteristics at a given point along the Pipeline, water hammer potential, and corrosion potential.

8.2.2.3 Winter Operations

Another major design consideration of the Pipeline is winter operations. The Pipeline will need to operate in the winter when flows are significantly less than in the summer. Low flows and low Pipeline velocities raise the risk of freezing. Five alternatives were identified to protect the Pipeline from damage due to freezing:

- Bury the pipe below the regional frost line (approximately 8 feet);
- Heat trace the Pipeline if installed above-grade;
- Drain the Pipeline after each operational cycle of the CPS;
- Stop use of the Pipeline during the winter; and/or
- Cover the Pipeline.

Shallow bedrock and heavily wooded areas along the Pipeline route make burial below grade cost-prohibitive. Heat tracing the Pipeline would also be costly, not only because of the actual heat trace materials and insulation, but also because power is not readily available along the entire alignment. Refraining from pumping throughout the winter is not a feasible alternative because groundwater leakage into the Mine Pits and stockpile drainage is expected to continue throughout the winter months, albeit at a lower rate than in summer, that would severely limit mining operations during some years. Draining the Pipeline after each operational cycle is not feasible due to the total volume of water in the pipe and the rolling nature of the Pipeline alignment which would require numerous pump-out points. Therefore, the most feasible alternative is to cover the Pipeline to prevent winter freezing.

The Pipeline will be covered with 8 feet of material consisting of overburden material from the Mine Site or Pipeline construction or LTVSMC tailings available at the Plant Site. This will prevent freezing during winter operations and protect the Pipeline from damage.

8.2.2.4 Bedding and Cover for Pipeline Construction

In the area where the Pipeline will be constructed, the land will be stripped of debris and vegetation, and a bedding material will be placed on the ground where the Pipeline will be laid. The preliminary design of the bed is approximately six inches thick. Once the Pipeline is placed on this bed, an eight-foot high berm will be constructed over the Pipeline, as mentioned in Section 8.2.2.3, to protect it from freezing during winter operations and from damage year-round. Side slopes of this berm will be approximately 1.5 to 1 (1.5H:1V) resulting in a footprint that will be approximately 26 feet in width. This width will vary as the topography changes along the Pipeline alignment. Figures 8.2.2.4-A through 8.2.2.4-D show typical cross sections of the Pipeline alignment along different topographic circumstances.

The material used for bedding and fill for the berm will consist of overburden material from the Pipeline construction or from the Mine Site, and/or LTVSMC tailings available at the Plant Site. LTVSMC tailings would be easy to use in construction for foundation bedding and cover, but would not facilitate vegetation growth on the resultant berm. If tailings are used as fill for the berm, the tailings would be covered by a minimum of two feet of overburden material and seeded to stabilize the soil against erosion.

Fugitive dust during construction may be of concern if tailings are used as a construction material for the Pipeline. A Fugitive Emissions Control Plan has been outlined for the Plant Site and the Mine Site and would also apply to construction of the Pipeline. Dust would need to be controlled during construction by application of water and/or MPCA-approved commercial dust suppressants.

8.2.2.5 Pipeline Appurtenances

Pipeline appurtenances will include air/vacuum valves at all high points along the route, valves near the CPS and at various intervals along the Pipeline, pig launching and retrieval stations at various points to facilitate cleaning if steel pipe is used, drains at strategically located low spots to facilitate draining, and water hammer control equipment at the CPS. In addition, the flow rate in the Pipeline will be monitored at each end using flow meters to ensure that leaks are detected as soon as possible, although leaks are significantly less likely in buried pipes as compared to aboveground installations based on Barr’s experience in pipeline design and operation.

8.2.3 Pipeline Operation

Pipeline operations that are important considerations at this stage in preliminary design include filling and draining techniques, cleaning of the Pipeline, and valve control.

8.2.3.1 Filling and Draining Techniques

The Pipeline will need to be carefully filled after construction or after being drained. Slow filling is needed to ensure that a controlled increase in system pressure occurs, that air is allowed to evacuate the system at high points, and that water hammer is prevented. As water reaches the top of a local rise, it will have a tendency to rush down the other side potentially trapping air, and setting up the potential for undesirable surges and water hammer. Filling the line at a restricted rate will allow air to evacuate the system in a controlled manor and minimize the potential for surges.

Draining of the Pipeline is generally undesirable and efforts should be taken to minimize the need to drain and re-fill the Pipeline. However, in the unlikely event that draining is required, drains will be installed at selected low points along the alignment. These drains will be designed so that the water could be pumped into tanker trucks to prevent discharging the water to the environment. These tanker trucks would transport the water to the Tailing Basin.

It should be noted that there are very few circumstances that would require the Pipeline to be drained or partially drained. The most likely scenario would involve the repair of a leak that could not be repaired with a simple sleeve. While leaks can occur, they would be extremely rare in a covered pipeline such as the one planned.

8.2.3.2 Cleaning the Pipeline

If steel is used, the Pipeline may need to be cleaned to prevent scale build-up. The WWTF will create water with a Langelier Index that is slightly positive once mixed with the untreated water at the CPS. This will result is a slight scale build-up in the Pipeline, which is preferable as opposed to corrosion of the interior of the Pipe, which would result from a negative Langelier Index. Cleaning will be accomplished by pigging, as is common in the industry. A pig is a device that moves through the inside of a pipeline for cleaning, dimensioning, or inspecting. Pigging involves launching a pig through the Pipeline to remove accumulated solids and debris from the walls of the Pipeline. Cleaning pigs are often equipped with blades or brushes to do the cleaning.

Pig launching and retrieval stations will be installed at appropriate points along the Pipeline. The frequency of cleaning will need to be determined once the system is in operation and more is known about the characteristics of the water being pumped. Descalants or other chemicals may be required to disturb any scale buildup or corrosion sites and remove scale, water, microbes, and corrosion products, depending on the Pipeline material and the chemical characteristics of the water being pumped. Any material removed from the Pipeline during cleaning will be disposed of at the Tailing Basin or off-site, depending on the nature of the material removed.

Cleaning the Pipeline is only a concern if steel is used. Other materials considered, such as HDPE, are either resistant to scale buildup or have linings that are resistant to scale buildup.

8.2.3.3 Valve Control

Preliminary calculations for a steel pipeline that is 38,000 feet long (note that the last 1,000 feet of pipe will be hose directing flow to the water in the Tailings Basin and are not included in this calculation), 16 inch outer diameter, with a wall thickness of 0.375 inches result in a critical period of just under 20 seconds. *This value must be recalculated during final design, and will change with wall thickness, diameter, length and material.* Any valve closed in this time or less will initiate a pressure surge equivalent to instantaneous flow stoppage which could damage the Pipeline and Pipeline appurtenances. Due to this potential for water hammer, all valve operation will occur over long preset periods in the range of approximately 40 seconds to one minute. Depending on the specific valves used, closure of the last 5 to 25% of the valve range may result in the greatest reduction in flow rate, so closure rates may not be linear.

9.0 Maintenance and Monitoring

All Mine Site water management infrastructure needs to be evaluated on a regular basis to maintain adequate capacity and functionality. A monitoring program will be developed for the Mine Site and will be refined through permitting, as necessary. All process water ditches, dikes, sumps and ponds should to be evaluated at least once every two months during non-frozen conditions, as required by the NPDES/SDS General Storm Water Permit for Industrial Activity, and after every major storm event equal to 2 inches or more within 24 hours. These inspections should be performed by a capable individual evaluating the systems for erosion or sediment build-up or structural damage. Annual evaluations of the depth of sediment buildup should be performed at the process water ponds and sumps, as a proactive measure for protection of the required design volume.

All pumps and pipelines should be visually inspected quarterly, and after every major storm event equal to 2 inches or more. Pumps and pipelines should be inspected annually or as required by the manufacturer to evaluate seals, pressure, and proper function. Pumps and pipelines should also be inspected prior to spring thaw to verify that no apparent damage has occurred over the winter. In addition to regular inspections, pump maintenance will be important for extending the life of the numerous pumps needed in this project.

Pump maintenance requirements will vary depending on the type of pump used and the amount of use the pumps experience. In general, submersible pumps installed in the mine dewatering and process water pond systems will have minimal routine maintenance needs. They should be checked at least quarterly for leaking seals and wear ring condition. The motors and cables should also be inspected. The pumps must be removed from service to perform these inspections. For planning purposes, submersible pumps may need to be rebuilt every seven to ten years. The expected service life for submersible pumps is fifteen to twenty years.

The vertical turbine pumps at the CPS will also need routine inspection of seals or packing condition and wear rings. The seal inspection may be possible without removing the motor or pump, depending on the type of seal. An inspection of the motor and electrical control components should also be performed annually. For planning purposes, the pumps may need to be rebuilt every seven to ten years. If operated under optimal conditions, well maintained and periodically rebuilt, the pumps may last for many years.

10.0 References

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Tables

Table 3.1.1-A Snowmelt Data for USGS Gage1 on Partridge River above Colby Lake

Water Year	Number of Days of Spring Runoff[2]	Runoff (inches)
1978-1979	33	7.7
1979-1980	39	1.8
1980-1981	NA[3]	
1981-1982	22	2.9
1982-1983	24	2.2
1983-1984	58	5.3
1984-1985	NA[3]	
1985-1986	29	3.0
1986-1987	58	1.6
1987-1988	35	1.6
Average	37.3	3.3
Final Values[4]	30.3	3.2

[1] USGS Gaging Station #040154750
[2] Defined by air temperatures greater than 34 degrees Fahrenheit and shown by the first peak in gage flows.
[3] Not applicable, the first peak is continuous and not clearly defined for these years.
[4] Water years 1983-1984 and 1986-1987 included several peak rainfall events that significantly influenced the runoff, so they were not included in the final values.

Table 3.1.1-B Average Annual and Peak Annual Inflow Rates in Pits

		Mine Year (all flows in gpm)				
		Year 1	Year 5	Year 10	Year 15	Year 20 (during operations)
West Pit	Average Annual Runoff[1]	39	71	123	162	162
	Annual Snowmelt Rate[2]	134	244	423	560	560
	Annual Groundwater Inflow	23	87	169	325	811
	Total Average Annual Flow (annual runoff + groundwater inflow)	62	157	292	487	973
	Total Annual Peak Flow (annual snowmelt + groundwater inflow)	157	331	592	885	1371
Central Pit	Average Annual Runoff[1]	0	0	0	32	32
	Annual Snowmelt Rate[2]	0	0	0	109	109
	Annual Groundwater Inflow	0	0	0	74	9
	Total Average Annual Flow (annual runoff + groundwater inflow)	0	0	0	105	41
	Total Annual Peak Flow (annual snowmelt + groundwater inflow)	0	0	0	183	119
East Pit	Average Annual Runoff[1]	31	57	69	69	69
	Annual Snowmelt Rate[2]	106	196	238	238	238
	Annual Groundwater Inflow	176	823	875	766	-108
	Total Average Annual Flow (annual runoff + groundwater inflow)	207	880	944	834	-39
	Total Annual Peak Flow (annual snowmelt + groundwater inflow)	282	1019	1114	1003	130

[1] Average annual runoff is over 365 days
[2] Annual snowmelt rates are over an average 30-day period

Table 3.1.1-C Rainfall for Given Recurrence Intervals of 24-Hour Storms (Huff and Angel, 1992)

Storm Event	Rainfall (inches)
2-Year	2.31
5-Year	2.88
10-Year	3.36
25-Year	4.08
50-Year	4.64
100-Year	5.20

Table 3.2-A Preliminary Pit Sump Specifications

Sump		Year	Total Volume[1] (acre-feet)	Sump Volume[2] (acre-feet)	Sump Area[2] (acres)	Area Available[3] (acres)
West Pit	WP-W	1	7.2	4.7	0.3	55.0
		5	7.3	4.7	0.3	1.2
		10	12.3	8.0	0.5	1.1
		15	14.4	9.4	0.6	1.1
		20	14.5	9.1	0.6	0.9
	WP-E	5	6.0	3.9	0.3	42.0
		10	10.9	7.0	0.5	1.4
		15	16.8	10.4	0.7	24.5
		20	18.7	10.7	0.7	3.1
Central	CP	15	4.0	2.4	0.2	0.5
		20	2.2	1.3	0.1	0.5
East Pit	EP-W	1	3.4	2.0	0.1	0.3
		5	3.0	2.0	0.1	0.2
		10	3.0	2.0	0.1	0.2
		15	2.4	1.3	0.1	21.9
		20	11.9	5.6	0.4	26.3
	EP-E	1	13.1	6.4	0.4	14.3
		5	13.0	6.4	0.4	0.9
		10	9.2	6.4	0.4	0.9
		15	6.1	3.9	0.3	0.2
		20	5.8	3.9	0.3	0.2

WP: West Pit, W: west cell, E: east cell, CP: Central Pit, EP: East Pit

[1] Total volume includes 1.28 inches of runoff (40 percent of the snowmelt volume) from all pit surfaces and the average groundwater inflow occurring during the 3 days required to remove the water.

[2] Required sump area and volume are based on an average depth of 15 feet; this represents the minimum size sump needed to hold the total volume minus the pumped volume. Pumps were sized to remove the water within 3 days from the start of the snowmelt event.

[3] Area available is the total footprint of the lowest level of the pit. This lowest level is may have to be the sump during spring snowmelt if there is not sufficient room for a sump and mining operations.

Table 3.2-B Preliminary Pump and Pipe Specifications for Pit Dewatering

Description[1]	Pump Number[1]	Peak Flow[2] (gpm)	Pipe Length (feet)	Static Head (feet)	Pump Horsepower
End of Year 1					
EP (East Pit)-W (West sump)	11	348	670	310	60
EP-W out	12	348	2,775	25	25
EP-E (East sump)	14	251	110	60	10
EP-E out	15	251	1,700	40	7.5
EP-Combined	13	598	3,960	55	30
EP-Dunka	63	651	4,425	110	50
PW-4	65	757	4,465	85	40
To WWTF	67	757	475	20	10
WP-W	1	560	150	90	30
To WWTF	2	560	865	5	2
End of Year 5					
EP-W	11	193	670	310	40
EP-W out	12	193	2,775	25	5
EP-E	14	1,413	635	375	275
EP-E out	26	1,413	1,380	20	20
EP-Combined	13	1,607	3,960	55	60
EP-Dunka	63	1,658	4,090	110	125
WP (West Pit)-Combined	34	2,132	425	5	8
PW-4	65	2,241	4,465	95	150
To WWTF	67	2,241	475	20	25
WP-E (East sump)	59	473	205	105	30
WP-E out	33	473	4,750	105	40
WP-W	22	588	635	360	125
WP-W2	73	588	660	315	100
To WWTF	2	588	865	5	3
End of Year 10					
EP-W	30	318	240	145	25.0
EP-W2	70	318	705	485	100.0
EP-W out	31	318	2,640	25	10.0
EP-E	14	1,510	705	505	400.0
EP-E2	71	1,510	310	165	150.0
EP-E out	37	1,510	1,175	10	15.0
EP-Combined	13	1,828	3,960	55	60.0
EP-Dunka	63	1,879	4,090	110	125.0
WP-Combined	34	2,772	425	5	10.0
PW-4	65	2,855	4,465	95	200.0
To WWTF	67	2,855	475	20	40.0
WP-E	32	893	700	175	100.0
WP-E2 (intermediate)	60	893	1,325	0	5.0

Table 3.2-B Continued

Description[1]	Pump Number[1]	Peak Flow[2] (gpm)	Pipe Length (feet)	Static Head (feet)	Pump Horsepower
End of Year 10 (continued)					
WP-E3 (intermediate)	61	893	400	280	150.0
WP-E out	33	893	4,750	105	60.0
WP-W	22	969	635	360	200
WP-W2	73	969	1,005	310	200
To WWTF	23	969	530	5	5
End of Year 15					
EP-W	30	224	240	145	20
EP-W2	70	224	705	485	60
EP-W out	31	224	2,640	25	5
EP-E	14	1,492	705	505	400
EP-E2	71	1,492	310	165	150
EP-E out	37	1,492	1,175	10	15
EP-Combined	13	1,717	3,960	55	75
EP-Dunka	63	1,768	4,090	110	125
WP-Combined	34	3,708	425	5	15
PW-4	65	3,782	4,465	95	225
To WWTF	67	3,782	475	20	50
WP-E	41	1,428	165	105	100
WP-B (Both)	54	1,428	1,645	335	250
WP-E3 (intermediate)	42	1,428	380	175	150
WP-E out	43	1,428	310	10	10
WP-CP (Central Pit Combined)	46	1,939	3,820	95	125
CP	44	511	600	350	100
CP2	72	511	295	270	75
CP out	45	511	1,345	10	5
WP-W (West cell)	22	1,136	1425	510	300
WP-W2	73	1,136	450	155	100
To WWTF	39	1,136	530	5	5
End of Year 20 (on next page)[3]					

Table 3.2-B Continued

Description[1]	Pump Number[1]	Peak Flow[2] (gpm)	Pipe Length (feet)	Static Head (feet)	Pump Horsepower
End of Year 20[3]					
WP-E	50	1,811	555	90	100
WP-B	51	1,811	905	610	600
WP-E3 (intermediate)	52	1,811	1,705	185	200
WP-E out	43	1,811	310	10	15
WP-CP	46	1,820	3,820	95	100
EP-E	14	0	705	505	250
EP-E2	71	0	310	165	75
EP-E out	37	0	5,135	65	10
WP-Combined	34	1,868	425	5	5
PW-4	64	1,942	4,465	95	100
To WWTF	67	1,942	475	20	20
CP	44	9	600	350	2
CP2	72	9	295	270	2
CP out	45	9	1,345	10	2
WP-W	22	1,240	1015	375	259
WP-W2	73	1,240	830	550	350
To WWTF	39	1,240	530	5	5

gpm: gallons per minute, E: East, P: Pit, S: Sump, W: West, WWTF: wastewater treatment facility, B: both

[1] Pump descriptions and numbers correspond to the pumps shown on Figures 1.1-A through 1.1-E and 7.1-A through 7.1-E.

[2] Peak flows as presented in Table 3.1.1-B from groundwater, precipitation, and surface runoff within the pits.

[3] Figure 7.1-E shows the pump and pipeline systems for the East and Central Pits still active for Year 20; however, these systems will only be used on an intermittent basis during pit filling. It is estimated that all pit runoff and groundwater will be used in pit filling in Year 20, so there will be little to no flow to the WWTF.

Table 3.3-A Water Balance for East and Central Pit Filling

Mine Year	Combined East and Central Pit Inflows[1] (gpm)	Annual Flow Required to Fill East and Central Pits[2] (gpm)	Additional Water Needed from CPS (gpm)	Excess Pit Water Diverted to CPS (gpm)
Year 12	960	1001	41	0
Year 13	953	432	0	521
Year 14	946	328	0	618
Year 15	940	1427	487	0
Year 16	781	1274	493	0
Year 17	622	1122	500	0
Year 18	415	913	498	0
Year 19	209	1024	816	0
Year 20	2	976	973	0

[1] Combined pit water includes direct precipitation, runoff, and groundwater inflows from the East and Central Pits.

[2] Annual flow required to fill pits is the volume required to keep the water surface within 5 feet from the backfilled rock surface and varies with the rock volume placed in the pits.

Table 4.1-A Comparison of Stockpile Parameters

Study		AMAX Piles	Dunka Piles	LTVSMC Piles	Cluff Piles	Mine Site
Reference		Eger and Lapakko (1985)	Eger, Antonson, Udoh (1990)	Eger, Melchert, Wagner (1999)	Nichol, Smith, Beckie (2005)	RS18, RS49, RS73
Location		Babbitt, MN	Babbitt, MN	Babbitt, MN	Saskatchewan, Can.	Babbitt, MN
Rock Source		Duluth Complex	Duluth Complex	Duluth Complex	Peter River Gneiss	Duluth Complex
Pile Size	*Height (ft)*	13	4	Full Scale	16.4	80-240
	Area (ac)	0.08-0.11	0.023	9-54 capped	0.016	54-565
	Volume (ac-ft)	0.32-0.67	0.092	Unknown	0.26	4,320-141,250
Liner	*Type*	30 mil Hypalon liner	60 mil HDPE liner	None	40 mil geomembrane over cement catchment system	Varies – See RS23T, RS49
Cover Material	*Thickness (ft)*	0, 0.6, 1, 1.9	2	2	None	2-3
	Type	None, topsoil with vegetation, glacial till with veg., sandy till on coarse sand with veg.	Glacial till, 20 mil PVC with pit run sand, bentonite mixed with glacial till	Screened soil, compacted soil barrier, 30 mil membrane		ET cover, membrane cover, combination
Precipitation (inches)		28.5 (average annual)	17.9-19.2 (July-Nov.)	19 (May-Nov.)	11.9 (average annual, 1981-1997 at site)	28.2 (average annual)
Evapotranspiration		14 in/yr	7.8-15.8 in (calculated)	Not given	Not given	Not available
Outflow collected from bottom (infiltration drainage) (% given is % of precip)		41% natural wshed, 44-58% no cover, 30% topsoil and sandy till over sand both with veg, 45% glacial till	Bottom: 19% PVC, 21% bentonite, 58% glacial till Barrier[2]: 0.3-12%	Not given	Outflow (bottom of pile) was 57% of precip; surface runoff was not allowed	Not available
Infiltration		4.1 in/yr (calculated)	3.2-11.2 in (total yield)	Reduction of 40% (native soil) to over 90% (membrane)	34-136% of precip[1] (average 63%)	Not available
Other Pertinent Information		-Individual annual outflow ranged from 28-66%. -No surface runoff.	-All barriers were capped with 12-inches of glacial till with sod. -No data provided for uncovered stockpiles -Only lasted 5 months	-Average seep flows before cover ranged from 3-840 L/min -Only tops were covered, not side slopes	-Several artificial storm events -Median residence time: 4.4 yrs -Initial response time of hydrograph was 2-12 hrs for large storm events. -See note 3 for additional work.	-Engineered, compacted covers

[1]Infiltration for the Cluff test piles was 136% of precipitation due to several artificial storm events (sprinklers) that were not quantified.

[2]Barrier flow for the Dunka Piles represents flow collected over the barrier (cover) layer but under the cap of glacial till with sod.

[3]Marcoline, Smith and Beckie (2006) reported 2-5 ft/yr as the upper limit for average pore velocities and preferential pore velocities as high as 6.5-13 ft/day for Cluff piles.

Table 4.1-B Annual Stockpile Process Water Volumes and Flow Rates[1]

	Mine Plan Year[2]	Annual Volume, Average Flow, and Nature of Area						
		High Estimate[1]		Low Estimate[1]		Uncovered	Covered	Total
		acre-ft	avg gpm	acre-ft	avg gpm	acres	acres	acres
Category 1/2 Stockpile	Year 1	94	59	72	44	69.3	0.3	69.6
	Year 5	315	195	78	48	259.2	69.1	328.3
	Year 10	194	120	62	38	237.0	195.0	432.0
	Year 15	332	206	148	92	0	437.0	437.0
	Year 20	332	206	148	92	0	437.0	437.0
Category 3 Stockpile	Year 1	8	5	6	4	5.9	0	5.9
	Year 5	32	20	24	15	21.8	3.8	25.6
	Year 10	57	35	41	25	36.6	10.3	46.9
	Year 15	80	50	56	34	46.9	25.1	72.0
	Year 20	37	23	16	10	0	72.0	72.0
Category 3 Lean Ore Stockpile	Year 1	48	30	37	23	35.4	0	35.4
	Year 5	81	50	59	37	54.5	9.3	63.8
	Year 10	101	62	54	34	67.6	30.1	97.7
	Year 15	129	80	42	26	79.1	77.7	156.8
	Year 20	84	52	36	22	0	156.8	156.8
Category 4 Stockpile	Year 1	6	4	5	3	4.5	0	4.5
	Year 5	49	30	37	23	35.6	4.4	40.0
	Year 10	57	35	42	26	39.8	23.5	63.3
	Year 15	52	32	38	23	35.7	27.6	63.3
	Year 20	7	5	2	1	0	63.3	63.3
Category 4 Lean Ore Stockpile	Year 1	74	46	56	35	54.5	0	54.5
	Year 5	74	46	56	35	54.5	0	54.5
	Year 10	74	46	56	35	54.5	0	54.5
	Year 15	74	46	56	35	54.5	0	54.5
	Year 20	74	46	56	35	54.5	0	54.5

acre-ft: acre-feet. avg gpm: average gallons per minute.

[1]High and low process water flow estimates are based on active stockpiles and covered stockpiles with grasses and forbs, as described in Sections 4.1.2.2 and 4.1.3.2 and on Tables 4.1.2.2-A (uncovered), 4.1.3-A and 4.1.3-B (covered). These estimates assume all process water is conveyed to the sump, with no loss from liner leakage.

[2]Although Table 4.1.2.2-A provides an estimate on the number of years to produce liner yield, this table conservatively assumes yields will occur immediately for calculation of process water.

Table 4.1-C Uncovered Stockpile Storm Event Yields: Surface Runoff plus Liner Drainage

Storm Event[1]	Precipitation (inches)	Runoff (inches)[2]	Infiltration (inches)	Drainage on Liner (inches)[3]	Total Yield (inches)[4]	% Yield
Typical	0.9	0.23	0.67	0.0	0.23	25%
10-year	3.36	2.23	1.13	0.07	2.30	68%
25-year	4.08	2.90	1.18	0.08	2.98	73%
50-year	4.64	3.43	1.21	0.09	3.52	76%
100-year	5.20	3.96	1.24	0.10	4.06	78%
500-year	6.30	5.03	1.27	0.11	5.13	82%

[1] Storm events analyzed are all based on a 24-hour event with the exception of the typical 0.9 inch storm.

[2] Runoff was calculated using an SCS curve number of 89 for gravel roads with a type C soil.

[3] Drainage on the liner was calculated using infiltration as the precipitation with an SCS curve number of 76 for gravel roads with a type A soil.

[4] Total yield equals runoff plus drainage on liner, which represents the total stockpile yield for the storm event.

Table 4.1.2.2-A Uncovered Stockpile Annual Yield Percentages: Surface Runoff plus Liner Drainage

Stockpile Height	Number of Years Uncovered to Produce Yield[1]	Uncovered Yield Percentage					
		Summer[2]		Winter[2]		Snowmelt[3]	
		Low	High	Low	High	Low	High
40’	3	44%	48%	0%	10%	90%	110%
80’	6	44%	48%	0%	10%	90%	110%
120’	9	0%	30%	0%	10%	0%	110%
160’	12	0%	30%	0%	10%	0%	110%
200’	15	0%	5%	0%	5%	0%	88%

[1] The number of years to produce yield is based on pore velocity, residence time, and retention capacity of the material, as described in Section 4.1.2.2. Table 4.1-B conservatively assumes flows occur immediately.

[2] Summer and winter yields are percentages of annual precipitation: summer values plus winter values equal annual yield.

[3] Snowmelt yields are percentages of average annual snowmelt runoff, which is accounted for as part of the summer yield percentages.

Table 4.1.3-A Evapotranspirative Cover Stockpile Yield Percentages

Yield	Evapotranspirative Cover Yield Percentage					
	Summer[1]		Winter[1]		Snowmelt[2]	
	Low	High	Low	High	Low	High
Tops and Benches: Slopes Approximately Equal to 1000H:1V						
Liner Yield	16%	26%	0%	10%	0%	26%
Side Slopes: Slopes 2.5H:1V or 3H:1V						
Liner Yield	13%	21%	0%	8%	0%	26%

[1] Summer and winter yields are percentages of annual precipitation: summer values plus winter values equal annual yield.

[2] Snowmelt yields are percentages of snowmelt runoff, which is accounted for as part of the summer yield percentage.

Table 4.1.3-B Membrane Cover Stockpile Yield Percentages

Yield	Membrane Cover Yield Percentage					
	Summer[1]		Winter[1]		Snowmelt[2]	
	Low	High	Low	High	Low	High
Liner Yield	1%	4%	0%	1%	0%	4%

[1] Summer and winter yields are percentages of annual precipitation: summer values plus winter values equal annual yield.

[2] Snowmelt yields are percentages of snowmelt runoff, which is accounted for as part of the summer yield percentage.

Table 4.1.5-A Preliminary Waste Rock Stockpile Sump Specifications

Stockpile[1]	Sump Number	Area (acres)	Average Depth (feet)	Storm Event[2] (inches)
Category 1/2	S-1	0.1	6	0.9
Category 1/2	S-2	0.2	6	0.9
Category 1/2	S-3	0.1	6	0.9
Category 1/2	S-4	0.1	6	0.9
Category 1/2	S-5	0.1	6	0.9
Category 4	S-8	1.3	6	3.36
Category 3 Lean Ore	S-9	1.3	6	3.36
Category 3 Lean Ore	S-10	1.5	6	3.36
Category 3	S-11	1.5	6	3.36

[1]This table only lists sumps for waste rock stockpiles. The Lean Ore Surge Pile Sumps (S-6 and S-7) are listed in Table 5.4-A.

[2]Sumps for the Category 1/2 stockpile were designed for the yield (surface runoff plus liner drainage) from a typical storm event of 0.9 inches, with overflow directed to the West Pit in lined ditches. Sumps for the other stockpiles were sized for the 10-year, 24-hour storm event with overflow either directed back onto the stockpile foundation liner or to an overflow pond up to the yield from the 100-year, 24-hour storm event.

Table 4.1.5-B Preliminary Stockpile Overflow Pond Specifications

Pond	Stockpile Sump Overflow	Pond Size (Acres)	Depth (feet)	Liner Type[1]
Pond PW-5	Category 3 Lean Ore and Category 4	3.0	6	Category 1/2
Pond PW-6	Category 3	1.2	6	Category 1/2

[1]Liner types are described in Section 4.1.5.

Table 4.1.5.1-A Comparison of Preliminary Sump Requirements for the Category 1/2 Stockpile[1]

Sump	0.9-inch Storm Event		10-year Storm Event[2]		100-year Storm Event[2]	
	Area (acre)	Volume (acre-feet)	Area (acre)	Volume (acre-feet)	Area (acre)	Volume (acre-feet)
S-1	0.11	0.7	0.63	3.8	4.33	26.0
S-2	0.15	0.9	0.73	4.4	5.05	30.3
S-3	0.11	0.7	0.55	3.3	3.83	23.0
S-4	0.08	0.5	0.41	2.5	2.82	16.9
S-5	0.04	0.2	0.28	1.7	1.89	11.4

[1]Based on the critical year, the year with the most runoff, for each individual sump.

[2]The 10-year and the 100-year events are both 24-hour storms.

Table 4.2-A Stockpile Construction Area Surface Runoff

		Annual Volume acre-ft	Annual Rate avg gpm
Stockpile Category 1/2	Year 1	136	85
	Year 5	73	45
	Year 10	0	0
Stockpile Category 3	Year 1	19	11
	Year 5	21	13
	Year 10	24	15
Stockpile Category 3 Lean Ore	Year 1	27	17
	Year 5	32	20
	Year 10	56	34
Stockpile Category 4	Year 1	33	21
	Year 5	22	14
	Year 10	0	0

acre-ft: acre-feet. avg gpm: average gallons per minute.

Table 5.4-A Preliminary Ore Handling Area Pond and Sump Specifications

Pond/Sump	Drainage Area	Critical Year[1]	Watershed Area (Acres)	Pond Size (Acres)	Depth (feet)	Liner Type[2]
Pond PW-2	Haul Roads	5	26	0.7-1.4	6-12	Category 1/2
Pond PW-3	Rail Transfer Hopper	NA	10	0.4-0.7	6-12	Category 3/4
Pond PW-4	Haul Roads	5	54	1.5-2.9	6-12	Category 1/2
Sump S-6	Lean Ore Surge Pile	NA	31	1.5	6	Category 3/4
Sump S-7	Lean Ore Surge Pile	NA	23	1.1	6	Category 3/4

[1]The Rail Transfer Hopper and sump watershed areas do not change from Mine Plan Year 1 through Year 20.

[2]Liner types are described in Section 4.1.5.

Table 6.4-A Preliminary Overburden Area Pond Specifications

Pond	Drainage Area	Critical Year	Watershed Area (Acres)	Pond Size (Acres)	Depth (feet)	Liner Type[1]
Pond PW-1	Overburden Storage and Laydown Area	5	92	1.7-3.3	6-12	None
Pond PW-7	Overburden Portion of the Category 1/2 Stockpile and Construction Area	5	173	3.1-6.2	6-12	None

[1]No liners are needed for the overburden ponds.

Table 7.2.3.1-A Preliminary Pump and Pipe Specifications for Category 3/4 Construction Area Surface Runoff and Overburden Surface Runoff (Pipeline 1)

Description[1]	Pump Number[2]	Critical Year[3]	Peak Flow[4] (gpm)	Pipe Length (feet)	Static Head (feet)	Pump Horsepower[5]
Year 5						
S-11.5 (Cat 3)	81	X	51	6,825	110	10
PW-2 (Roads)	64	X	53	260	15	0.5
S-10.5 (Cat 3LO)	29		119	2,020	55	5
S-8.5 (Cat 4)	80	X	190	2,240	85	15
Combined (S-11, PW-2 and S-10)	62	X	293	4,275	105	25
PW-4 (Roads)	65	X	402	3,390	70	25
PW-1 (SW ob)	10	X	588	1,110	15	7.5
To CPS	69	X	588	110	5	2
Year 10						
S-11.5 (Cat 3)	74		51	6,825	110	5
PW-2 (Roads)	64		53	260	15	0.5
S-10.5 (Cat 3LO)	82	X	119	5,820	150	15
Combined (S-11, PW-2 and S-10)	62		222	4,275	105	15
PW-4 (Roads)	65		331	3,390	70	20
PW-1 (SW ob)	10		517	1,110	15	7.5
To CPS	69		517	110	5	1.5

gpm: gallons per minute, S: Sump, Cat: Category, PW: Process Water pond, SE: Southeast, ob: overburden, LO: Lean Ore, SW: southwest, CPS: Central Pumping Station, NA: not applicable.

[1] Pump descriptions correspond to locations shown on Figures 1.1-A through 1.1-E and 7.1-A through 7.1-E.

[2] Pump numbers correspond to the pumps shown on Figures 7.1-A through 7.1-E.

[3] Critical year is based on the year with the most process water volume. The critical year is Year 5 or 10 for all pumps in this system.

[4] Peak flow includes peak annual flows from a snowmelt event.

[5] Pump horsepower is the maximum horsepower necessary during the Critical Year when all pumps in the system are cycling.

Table 7.2.3.3-A Preliminary Pump and Pipe Specifications for Category 3/4 Process Water and Rail Transfer Hopper Surface Runoff (Pipeline 3)

Description[1]	Pump Number[2]	Critical Year[3]	Peak Flow[4] (gpm)	Pipe Length (feet)	Static Head (feet)	Pump Horsepower[5]
S-11 (Cat 3)	17	15	112	6,825	110	10
S-10 (Cat 3LO)	29	15	107	2,020	55	7.5
S-9 (Cat 3LO)	24	15	201	1,560	20	3
S-8 (Cat 4)	25	10	291	2,085	75	15
Combined (S-11, S-10, S-9 and S-8)	57	10	403	570	20	5
S-7 (Cat 4LO)	7	15	454	1,690	50	15
S-6 (Cat 4LO)	8	15	524	2,355	65	20
PW-3 (Rail Transfer Hopper)	9	15	544	3,390	65	25
To WWTF	68	15	544	305	20	7.5

gpm: gallons per minute, S: Sump, Cat: Category, LO: Lean Ore, NA: not applicable.

[1] Pump descriptions correspond to locations shown on Figures 1.1-A through 1.1-E and 7.1-A through 7.1-E.

[2] Pump numbers correspond to the pumps shown on Figures 7.1-A through 7.1-E.

[3] Critical year is based on the year with the most process water volume.

[4] Peak flow includes peak annual flows from a snowmelt event.

[5] Pump horsepower is the maximum horsepower necessary during the Critical Year when all pumps in the system are cycling.

Table 7.2.3.5-A Preliminary Pump and Pipe Specifications for Category 1/2 Stockpile Process Water (Pipeline 5)

Description[1]	Pump Number[2]	Critical Year[3]	Peak Flow[4] (gpm)	Pipe Length (feet)	Static Head (feet)	Pump Horsepower[5]
S-5	19	10	81	2,410	35	3
S-4	21		204	2,370	30	5
S-3	27		375	1,550	15	10
S-2	28		598	1,050	15	10
S-1	35		191	2,765	60	7.5
Combined (S-1 through S-5)	36		790	3,180	40	25
To WWTF	55		790	545	5	3

gpm: gallons per minute, S: Sump, NA: not applicable.

[1] Pump descriptions correspond to locations shown on Figures 1.1-A through 1.1-E and 7.1-A through 7.1-E.

[2] Pump numbers correspond to the pumps shown on Figures 7.1-A through 7.1-E.

[3] Critical year is based on the year with the most process water volume.

[4] Peak flow includes peak annual flows from a snowmelt event.

[5] Pump horsepower is the maximum horsepower necessary during the Critical Year when all pumps in the system are cycling.

Table 7.2.3.6-A Preliminary Pump and Pipe Specifications for Category 1/2 Construction Area Surface Runoff (Pipeline 6)

Description[1]	Pump Number[2]	Peak Flow[3] (gpm)	Pipe Length (feet)	Static Head (feet)	Pump Horsepower
Year 1					
S-5	19	36	3,400	55	2.5
Construction Area	89	169	3,910	35	10
Pond PW-7	81	449	3,180	40	15
(Intermediate)	83	280	165	10	2
To CPS	84	449	545	5	2
Year 5					
Pond PW-7	81	362	165	10	3
(Intermediate)	83	362	3,180	40	15
To CPS	84	362	545	5	2

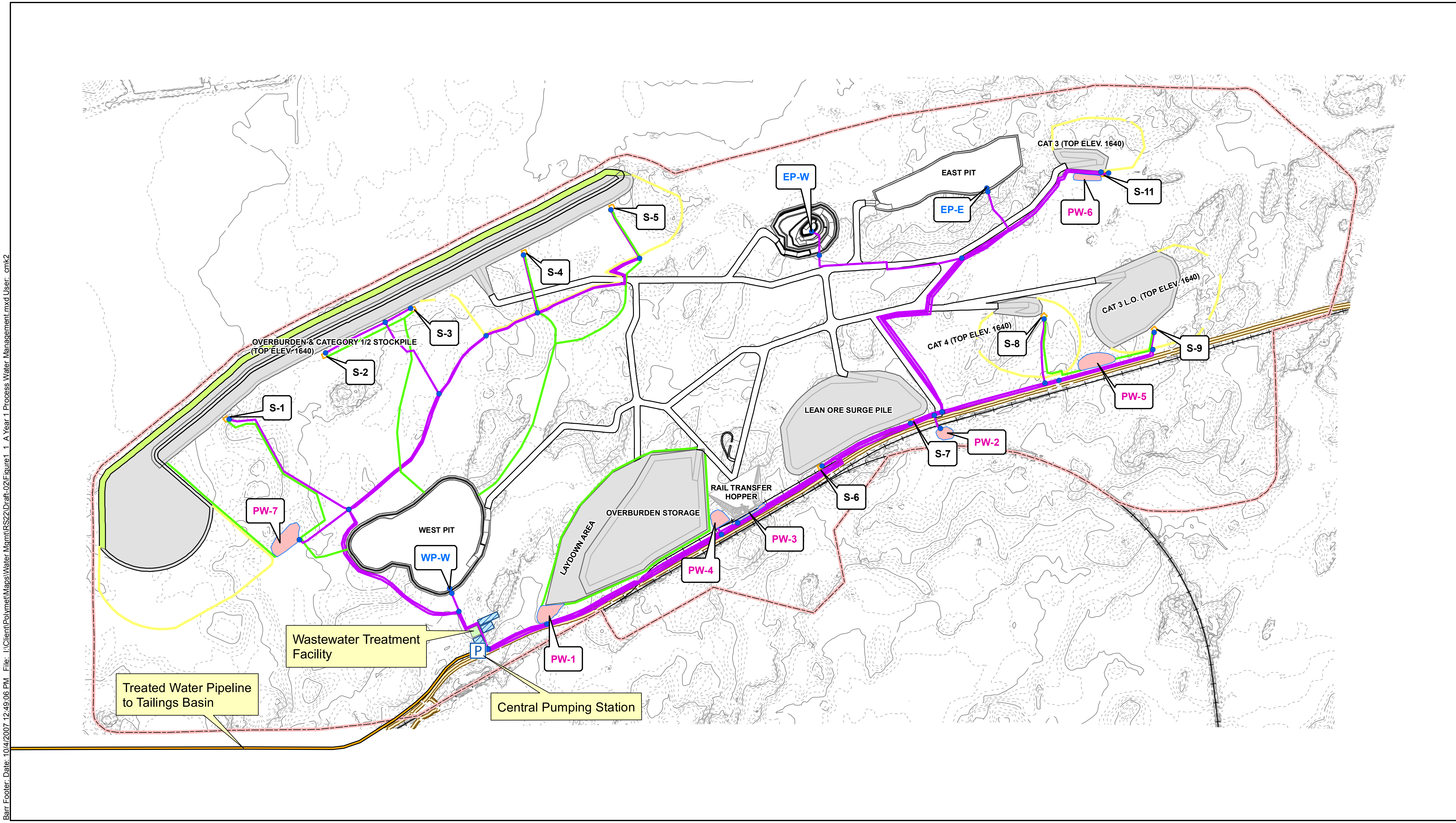
gpm: gallons per minute, S: Sump, CPS: Central Pumping Station.

[1] Pump descriptions correspond to locations shown on Figures 1.1-A through 1.1-B and 7.1-A through 7.1-B.

[2] Pump numbers correspond to the pumps shown on Figures 7.1-A through 7.1-B.

[3] Peak flow includes peak annual flows from a snowmelt event.

Figures



Year 1 Process Water
- Ditches
- Pipes
- Pit Rim and Stockpile Prep Dikes
- Year 1 Pumps
- Process Water Sedimentation Ponds

Year 1 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Category 1/2 Overburden
- Stockpile Sumps
- Haul Roads

Year 1 Mine Plan (8-22-06)
- Index Contours
- Other

Year 1 Stockpile Covers-Cat12
- Covered - Year 1
- Not Yet Covered (Active)

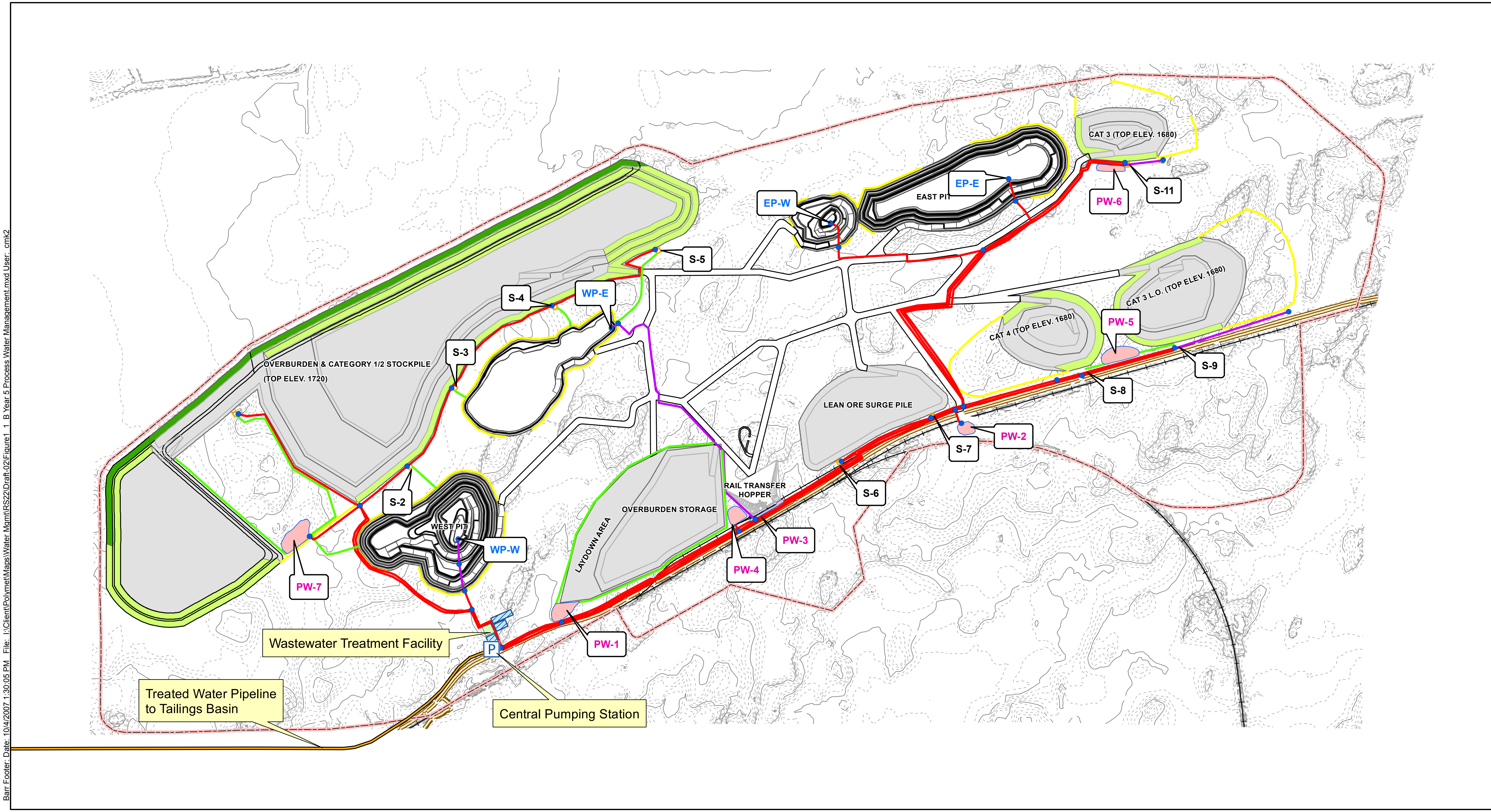
- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
Pink = Process Water Ponds
Blue = Mine Pit Sumps
Black = Stockpile Sumps

Note: Pipeline alignments have been exaggerated for illustration purposes.

0 750 1,500 3,000
Feet

RS22 Figure 1.1-A
YEAR 1 PROCESS WATER MANAGEMENT
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Year 5 Process Water
- Ditches
- Pipes
- Pipes - New Year 5
- Pit Rim and Stockpile Prep Dikes
- Year 5 Pumps
- Sedimentation Ponds

Year 5 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Category 1/2 Overburden
- Stockpile Sumps
- Haul Roads

Year 5 Mine Plan (8-22-06)
- Index Contours
- Other

Year 5 Stockpile Covers
- Covered - Year 5
- Not Yet Covered (Active)

- Mine Site
- Railroad
- Dunka Road

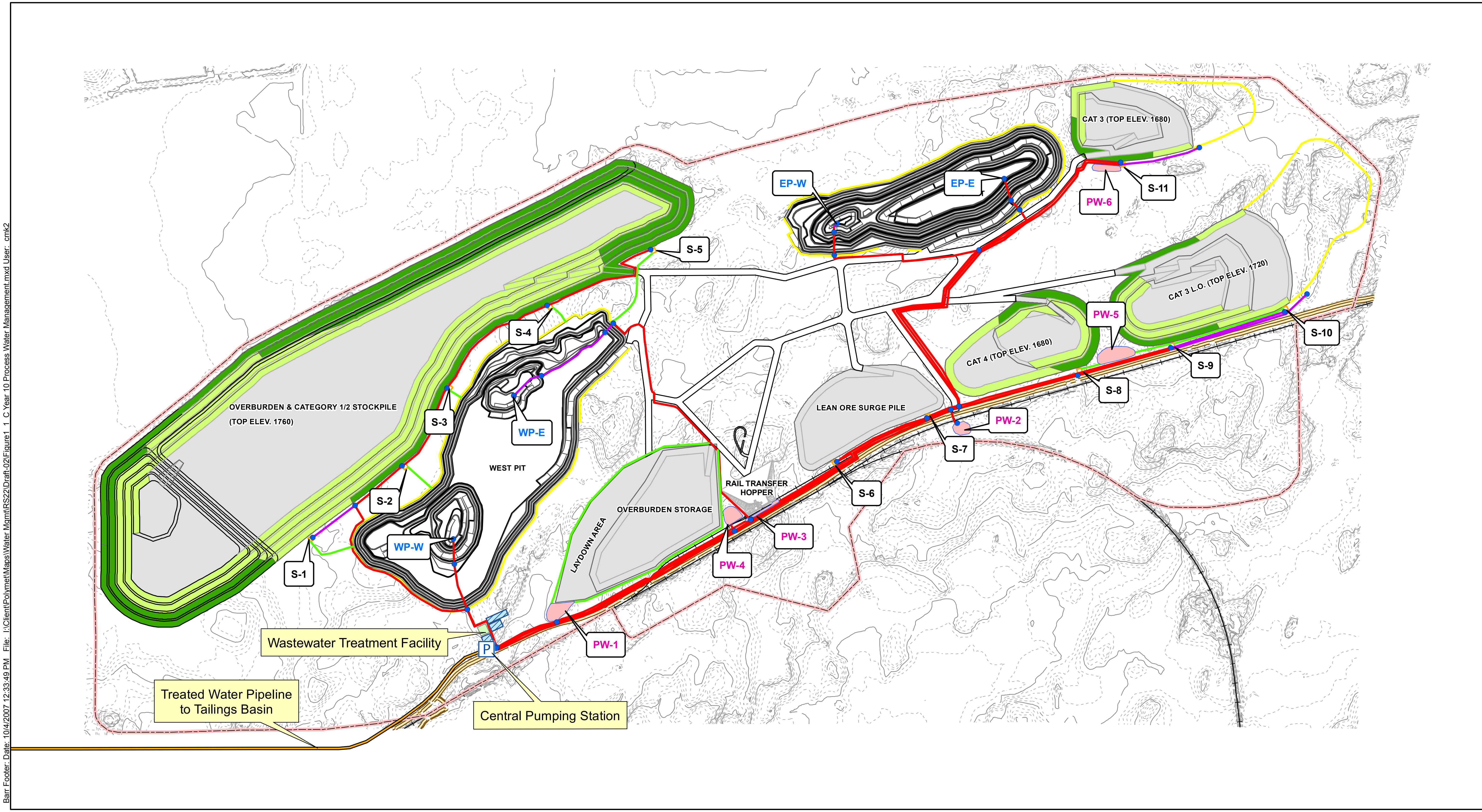
Callouts depict the following:
Pink = Process Water Ponds
Blue = Mine Pit Sumps
Black = Stockpile Sumps

Note: Pipeline alignments have been exaggerated for illustration purposes.

N

0 750 1,500 3,000
Feet

RS22 Figure 1.1-B
YEAR 5 PROCESS WATER MANAGEMENT
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Year 10 Process Water
- Ditches
- Pipes
- Pipes - New Year 10
- Pit Rim and Stockpile Prep Dike
- Year 10 Pumps
- Process Water Sedimentation Pond

Year 10 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Stockpile Sumps
- Category 1/2 Overburden
- Haul Roads

Year 10 Mine Plan (8-22-06)
- Index Contours
- Other

Year 10 Stockpile Covers
- Covered - Year 10
- Covered Previous Years
- Not Yet Covered (Active)

- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
Pink = Process Water Ponds
Blue = Mine Pit Sumps
Black = Stockpile Sumps

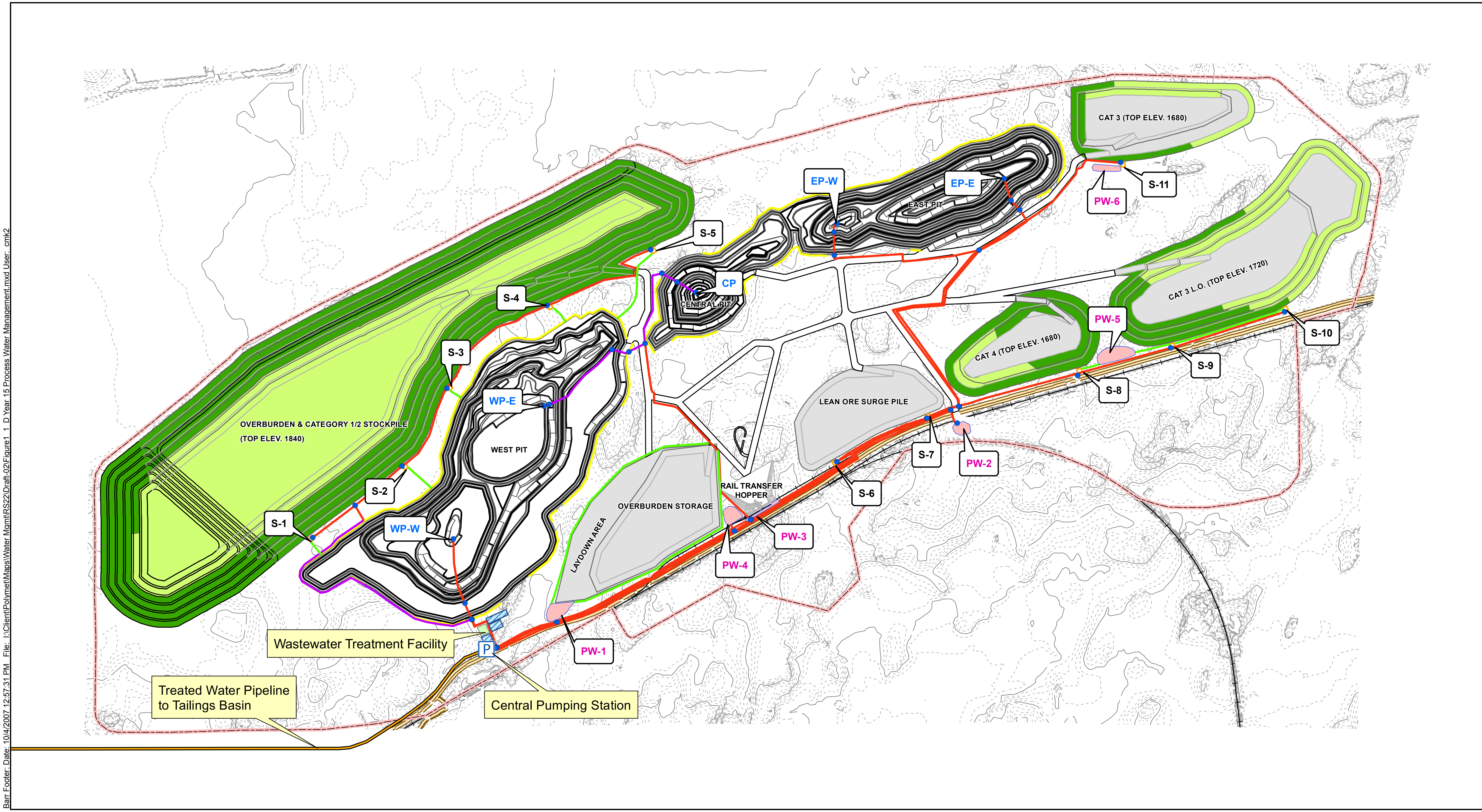
Note: Pipeline alignments have been exaggerated for illustration purposes.

N

0 750 1,500 3,000
Feet

RS22 Figure 1.1-C
YEAR 10 PROCESS WATER MANAGEMENT
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

Barr Footer: Date: 10/4/2007 12:33:49 PM File: I:\Client\Polymet\Maps\Water Mgmt\RS22\Draft-02\Figure1_1_C Year 10 Process Water Management.mxd User: cmk2



Year 15 Process Water
- Ditches
- Pipes
- Pipes - New Year 15
- Pit Rim Dikes
- Year 15 Pumps
- Process Water Sedimentation Pond

Year 15 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Stockpile Sumps
- Category 1/2 Overburden
- Haul Roads

15 Year Mine Plan (8-22-06)
- Index Contours
- Other

Year 15 Stockpile Covers
- Covered - Year 15
- Covered Previous Years
- Not Yet Covered (Active)

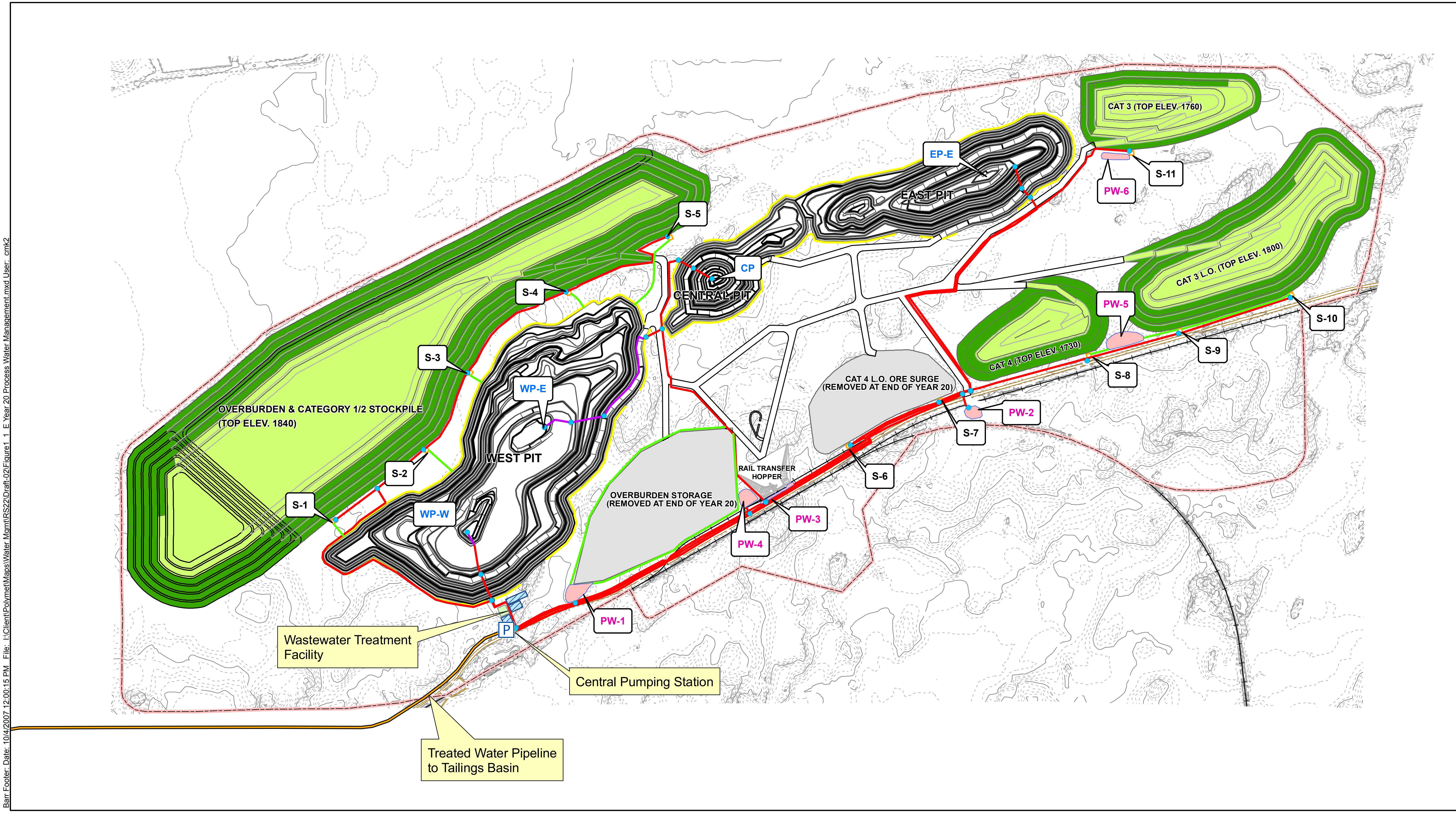
- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
Pink = Process Water Ponds
Blue = Mine Pit Sumps
Black = Stockpile Sumps

Note: Pipeline alignments have been exaggerated for illustration purposes.

0 750 1,500 3,000
Feet

RS22 Figure 1.1-D
YEAR 15 PROCESS WATER MANAGEMENT
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Year 20 Process Water
- Ditches
- Pipes
- Pipes - New Year 20
- Pit Rim Dikes
- Year 20 Pumps
- Process Water Sedimentation Pond

Year 20 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Category 1/2 Overburden
- Stockpile Sumps
- Haul Roads

20 Year Mine Plan (8-22-06)
- Index Contours
- Other

Year 20 Stockpile Covers
- Covered - Year 20
- Covered Previous Years
- Not Yet Covered (Active)

- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
Pink = Process Water Ponds
Blue = Mine Pit Sumps
Black = Stockpile Sumps

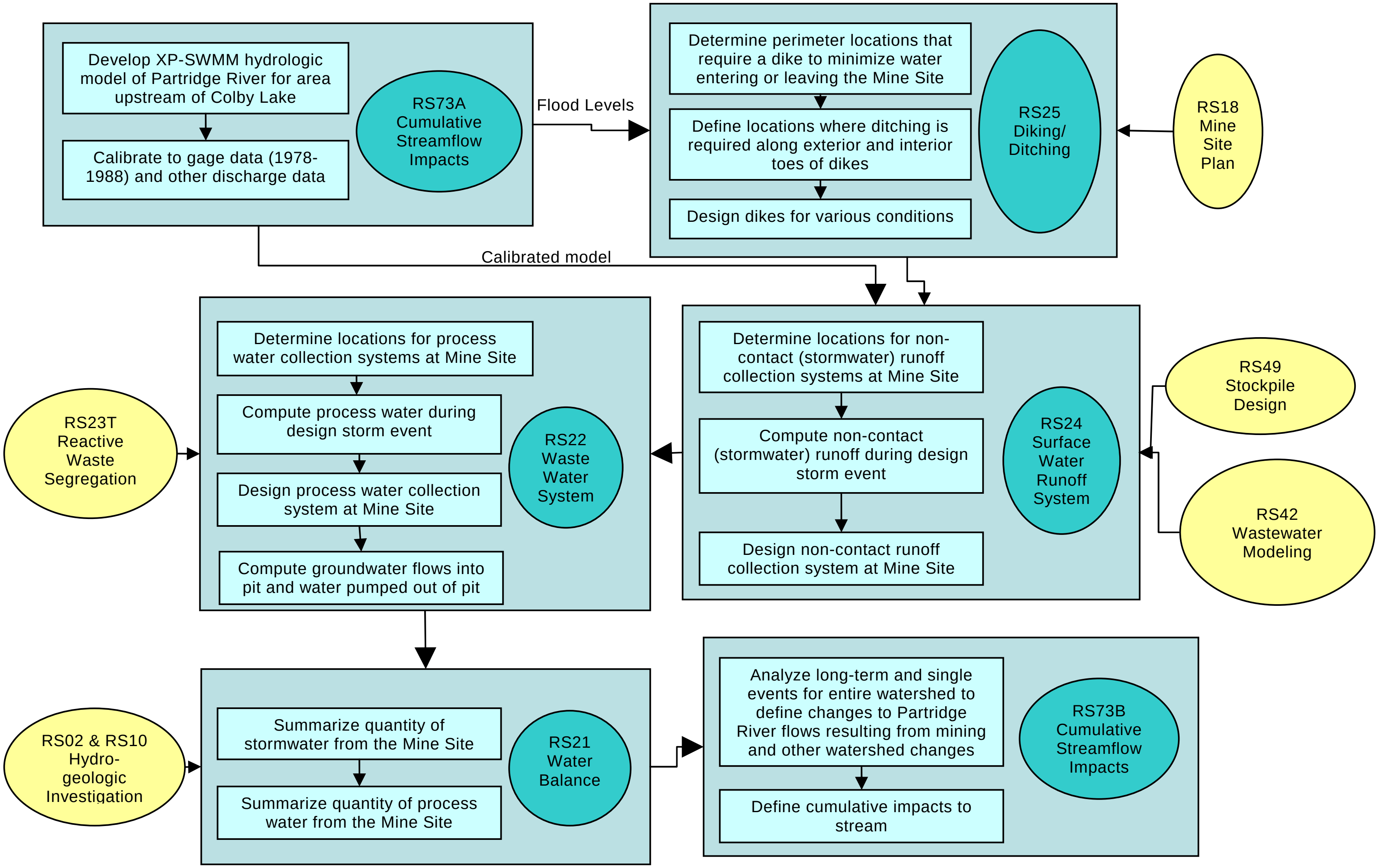
Note: Pipeline alignments have been exaggerated for illustration purposes.

N

0 750 1,500 3,000
Feet

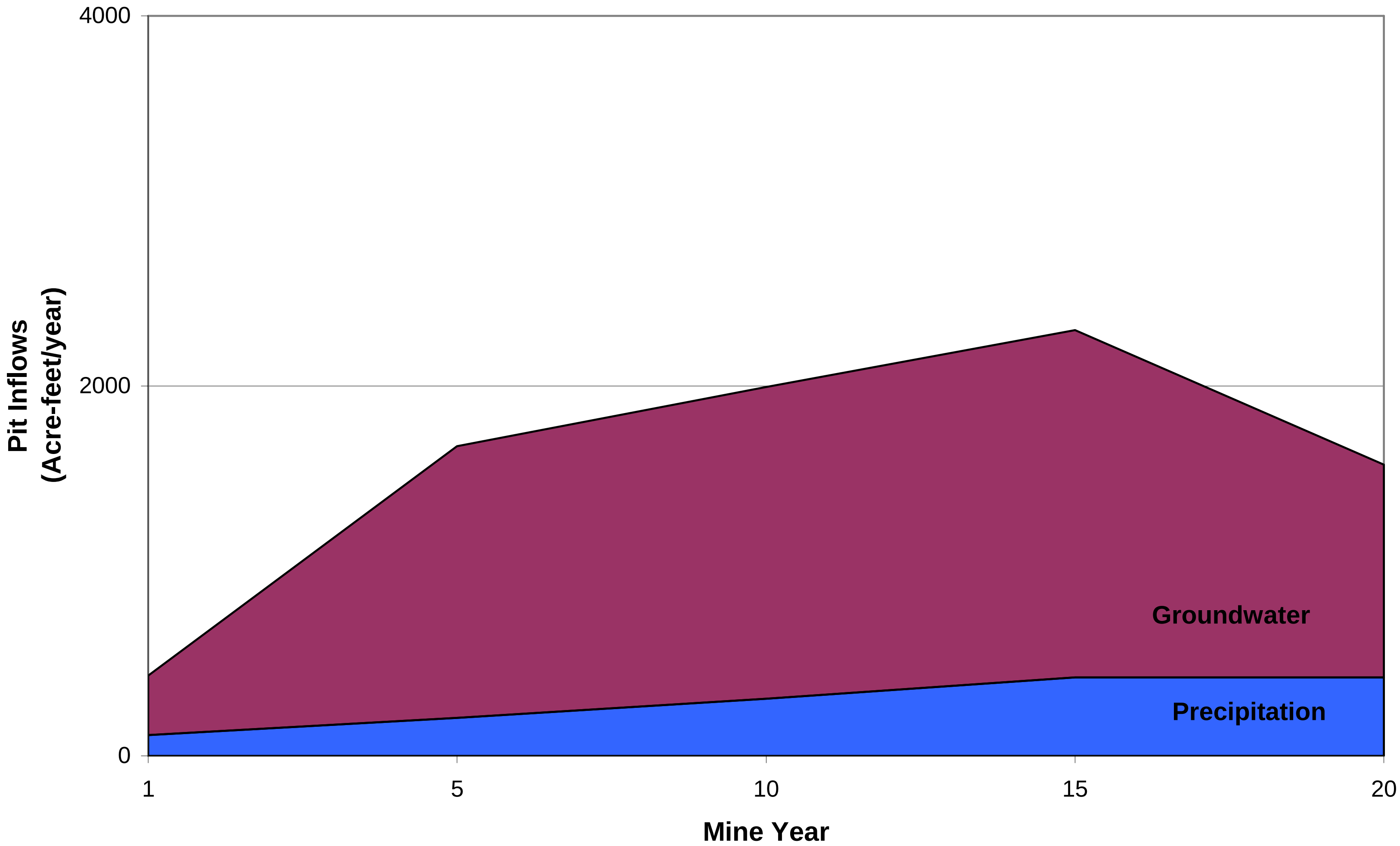
RS22 Figure 1.1-E
YEAR 20 PROCESS WATER MANAGEMENT
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

Figure 1.2-A Mine Site Water Management Studies



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 3.1.1-A Mine Pit Inflows



RS22

Figure 3.2-A   Pit Dewatering Typical Cross Section

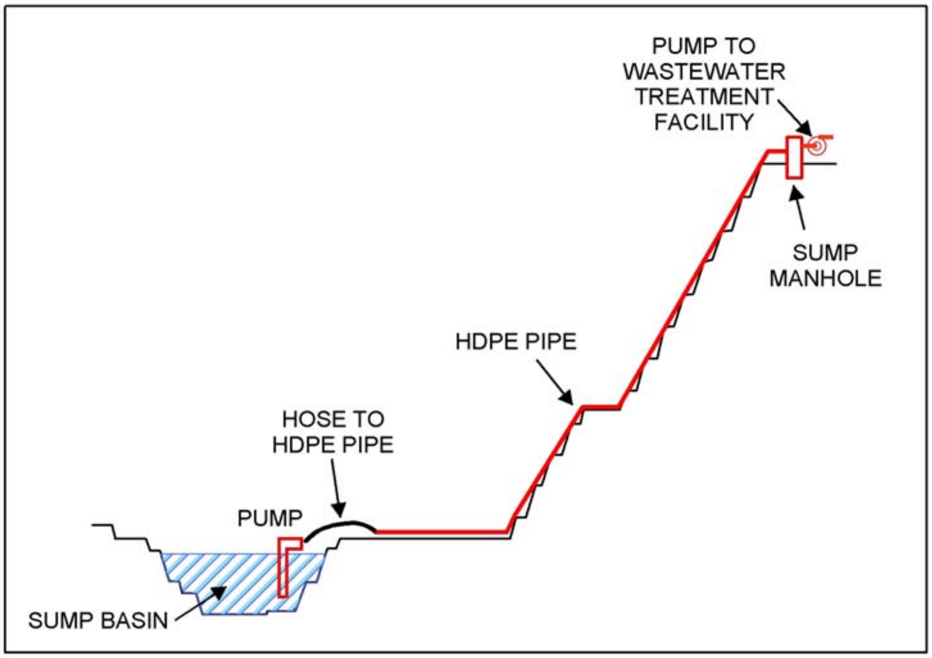


Figure 3.3-A   East And Central Pit Filling

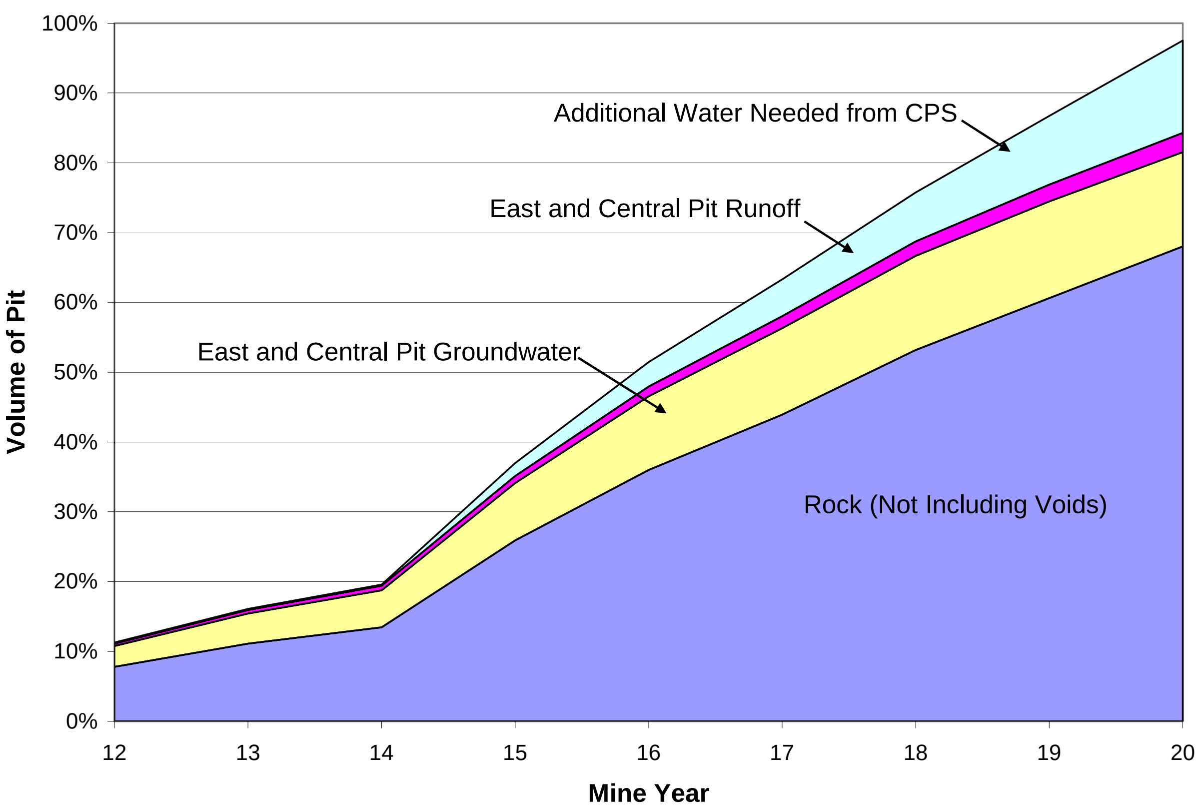


Figure 4.1-A   Active Stockpile Water Balance

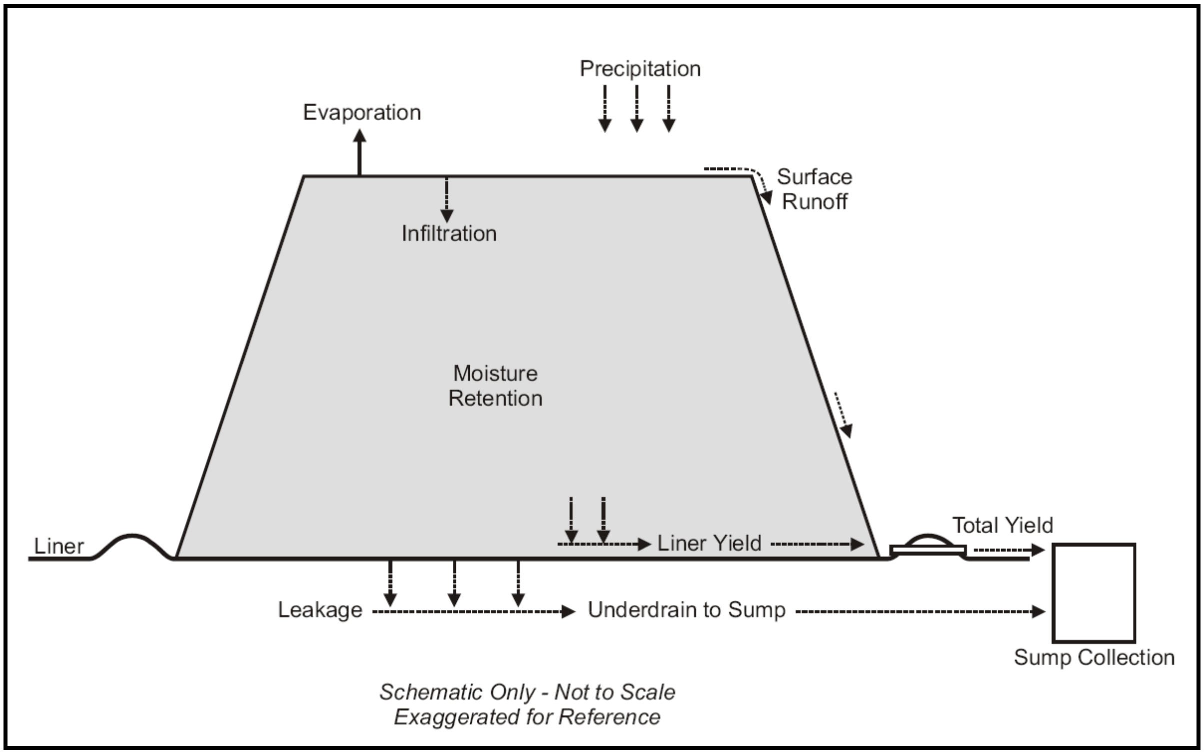
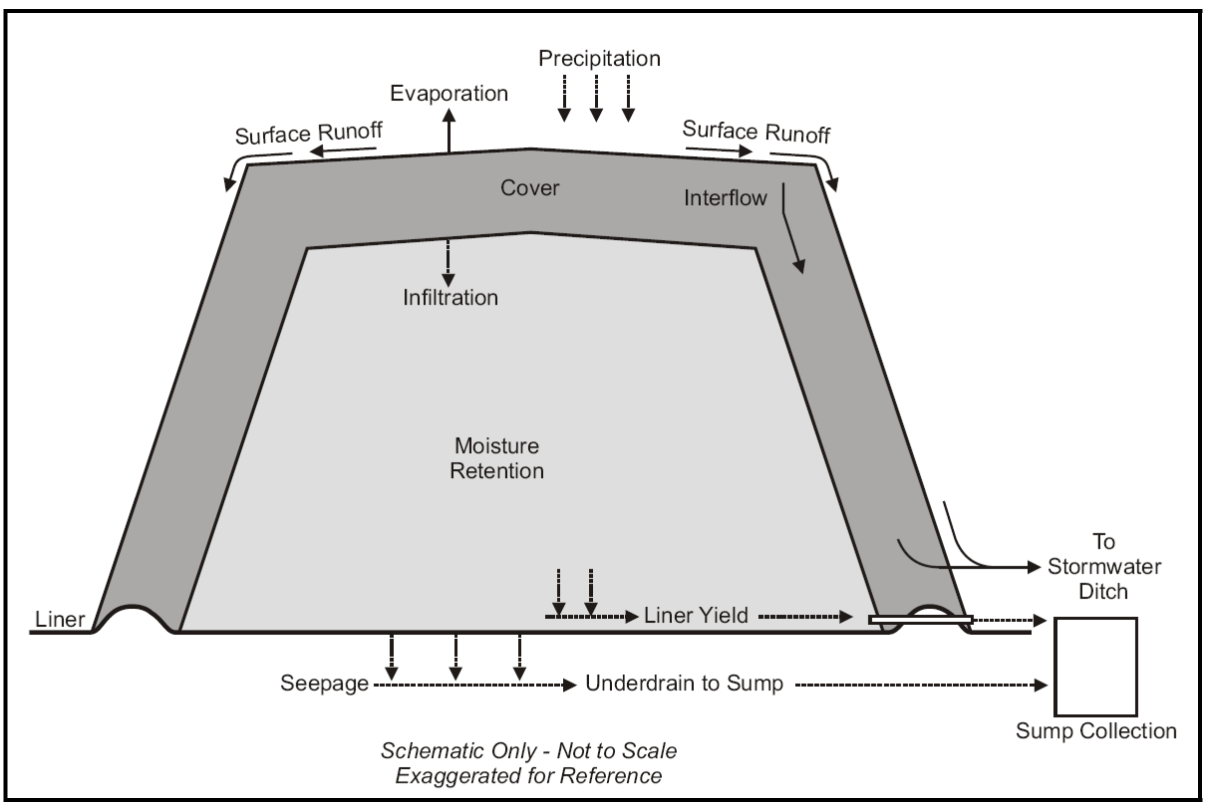
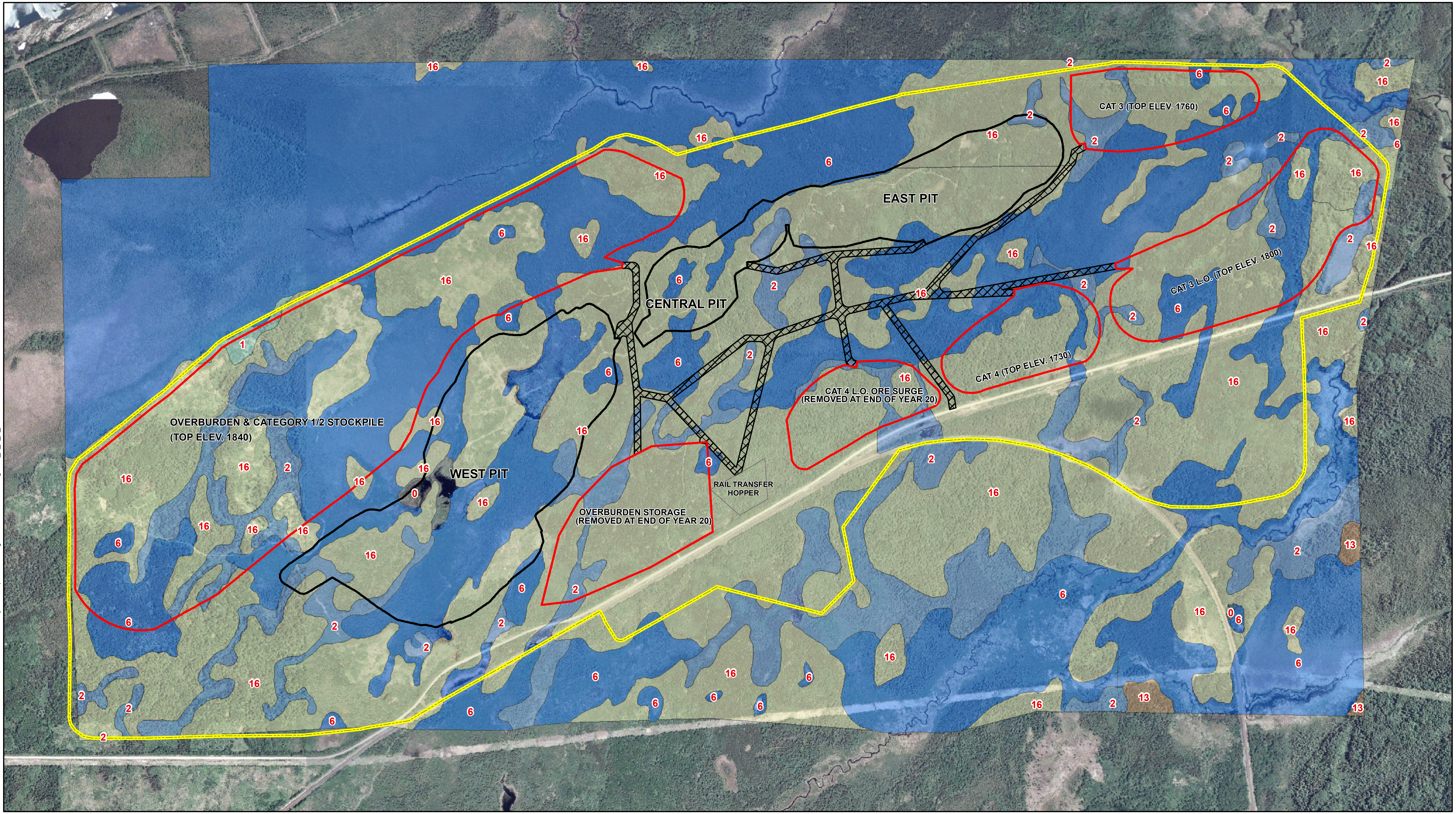


Figure 4.1-B   Reclaimed Stockpile Water Balance





Project Area
Year 20 Pit Footprints
Year 20 Stockpile Footprints
Year 20 Haul Roads

USFS and Grigal Soils used for EAW

Ecological Landtype (Hydrologic Soil Group)

1-Lowland Loamy Moist (Type D)
2-Lowland Loamy Wet (Type D)
6-Lowland Organic Acid to Neutral (Type D)
13-Upland Deep Loamy Dry Course (Type B)
16-Upland Shallow Loamy Dry (Type B)

See Appendix C for a description of each soil and landtype.

Figure 4.1.3.3-A
U.S. FOREST SERVICE SOILS MAP
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

**Figure 4.1.4.2-A	Typical Foundation Underdrain (from RS49)**

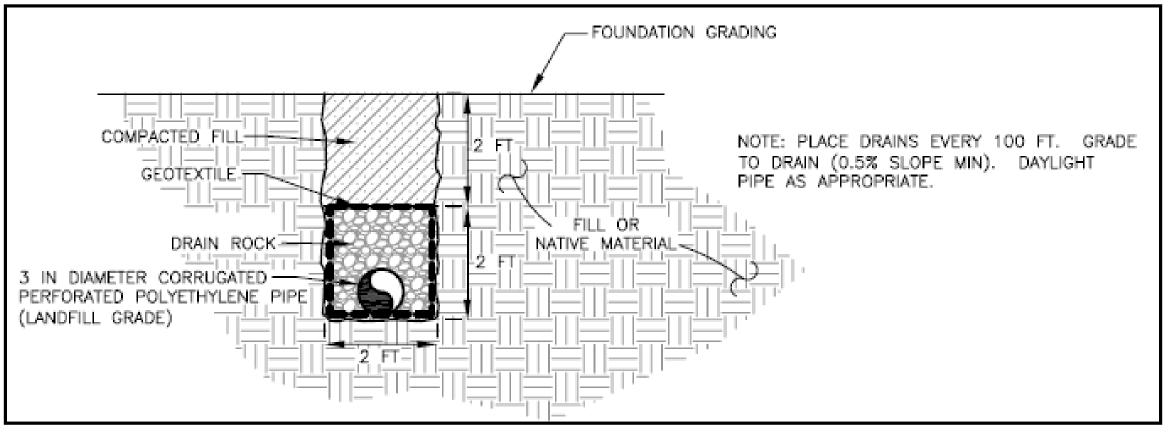


**Figure 4.1.4.2-B	Plan View of Underdrain/Sump Interaction**

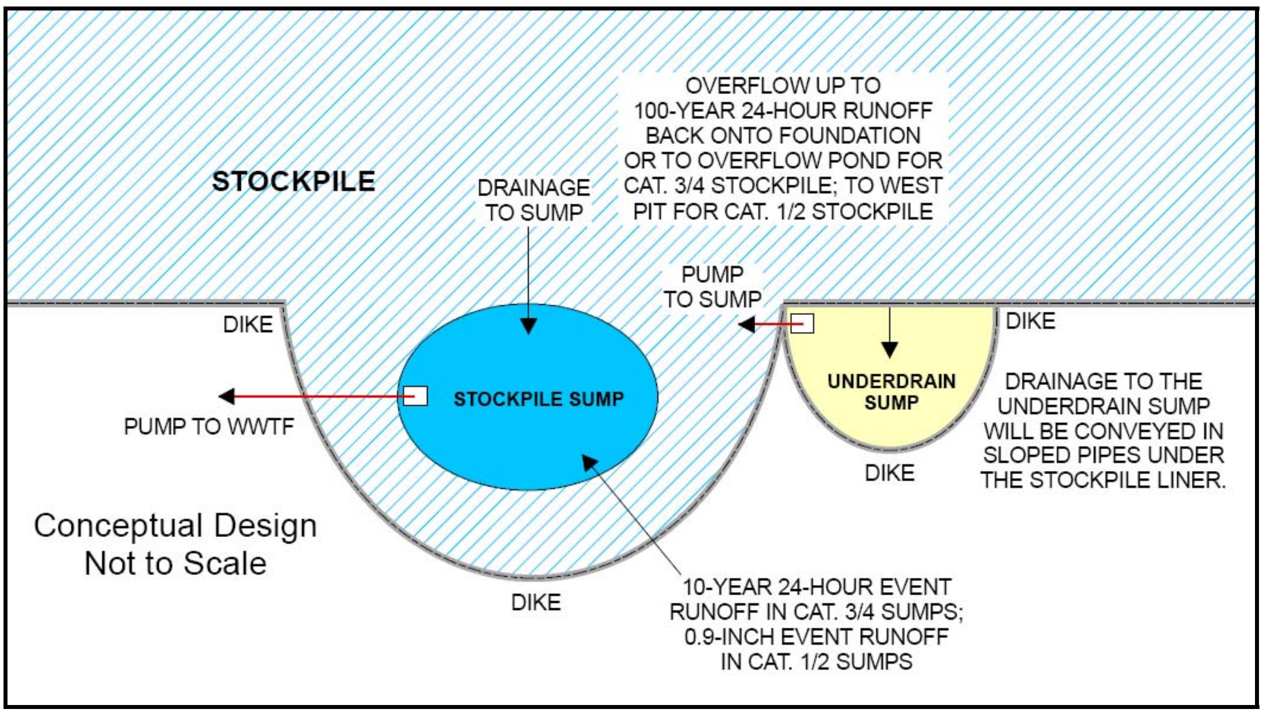


Figure 4.1.5.1-A Conceptual Category 1/2 Sump Layout

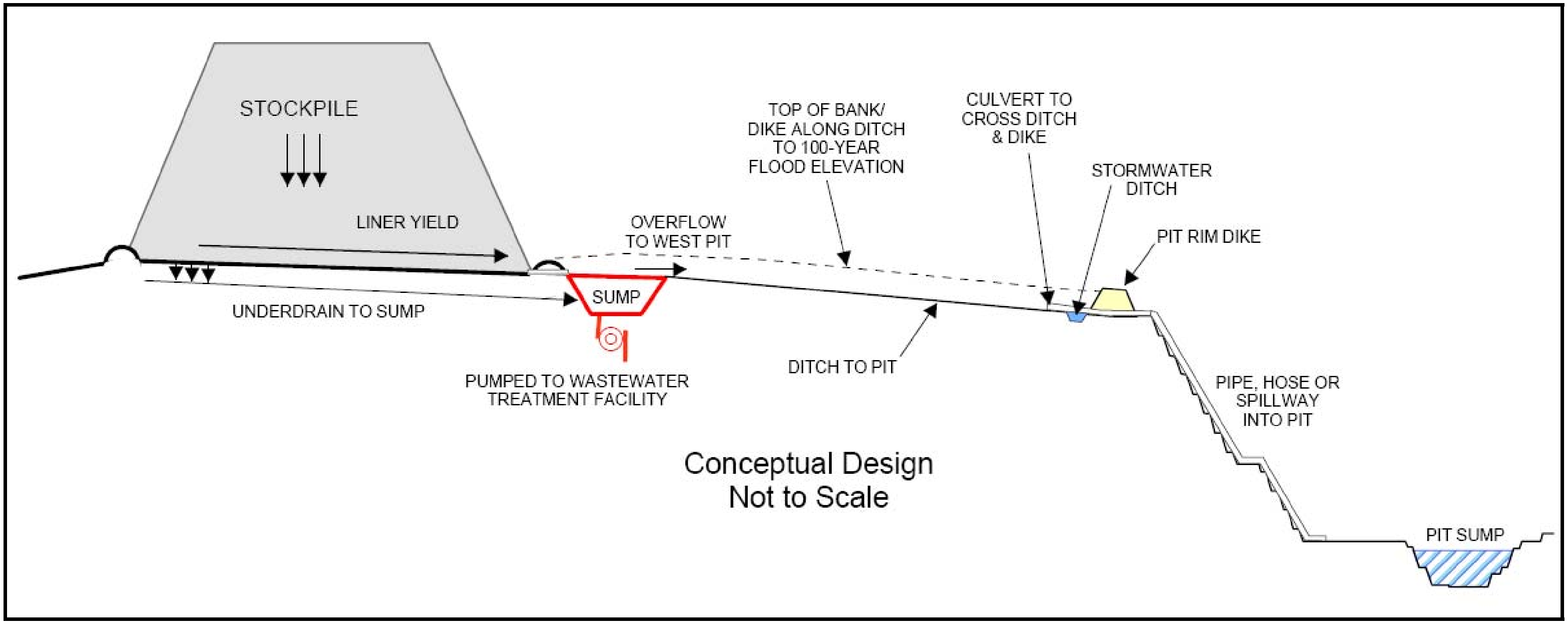


Figure 4.1.5.1-B Category 1/2 Material Drainage Sump Liner (Drawing 12 of RS49)

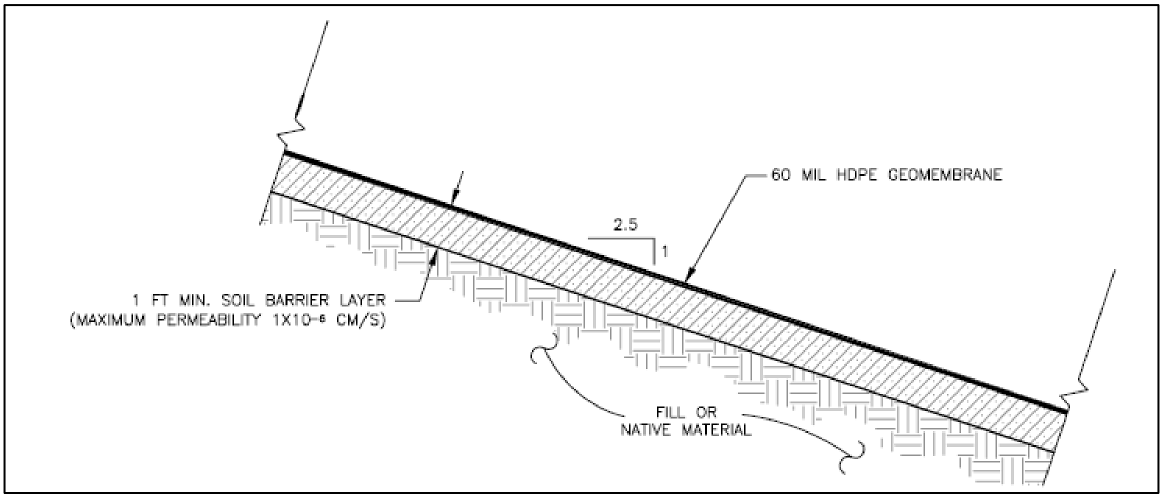


Figure 4.1.5.2-A Generalized Stockpile Liner Configuration (Figure 2 from RS23T)

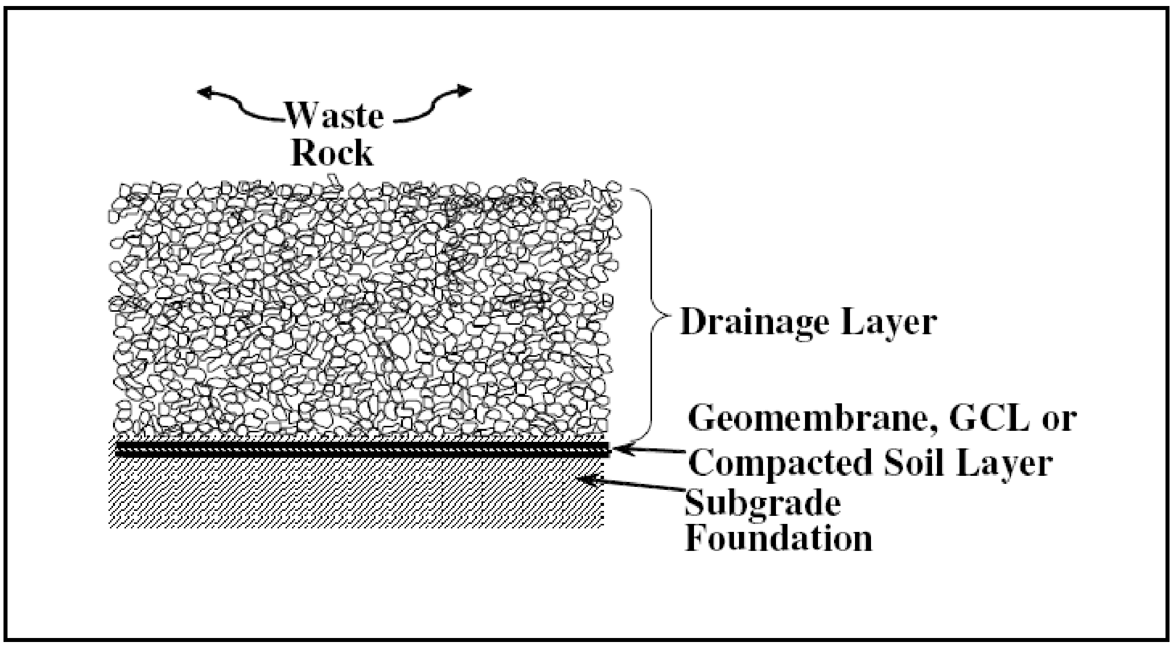


Figure 4.1.5.2-B **Conceptual Category 3 & 4 Sump Layout**

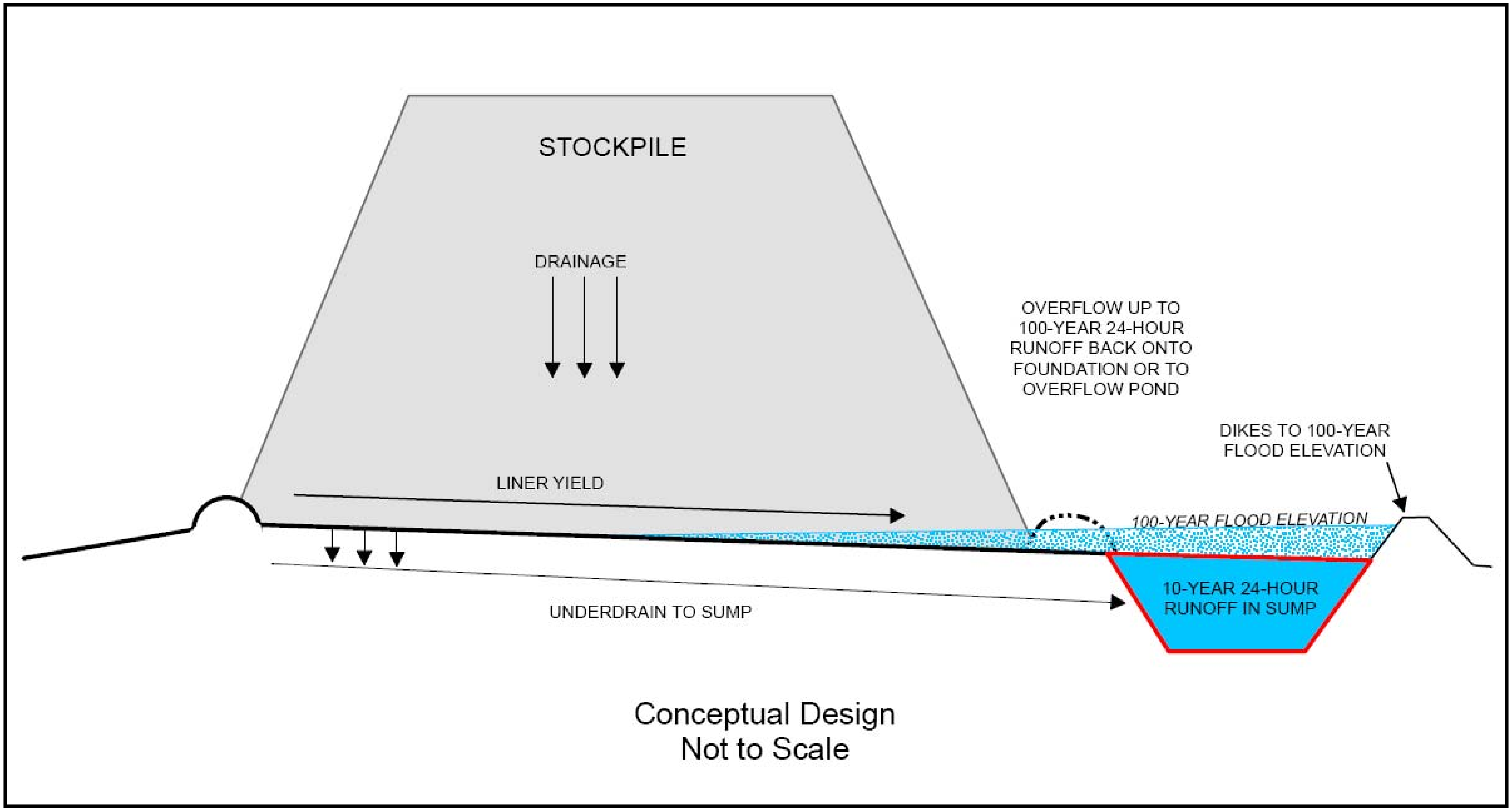


Figure 4.1.5.2-C Category 3/4 Material Drainage Sump Liner (Drawing 12 of RS49)

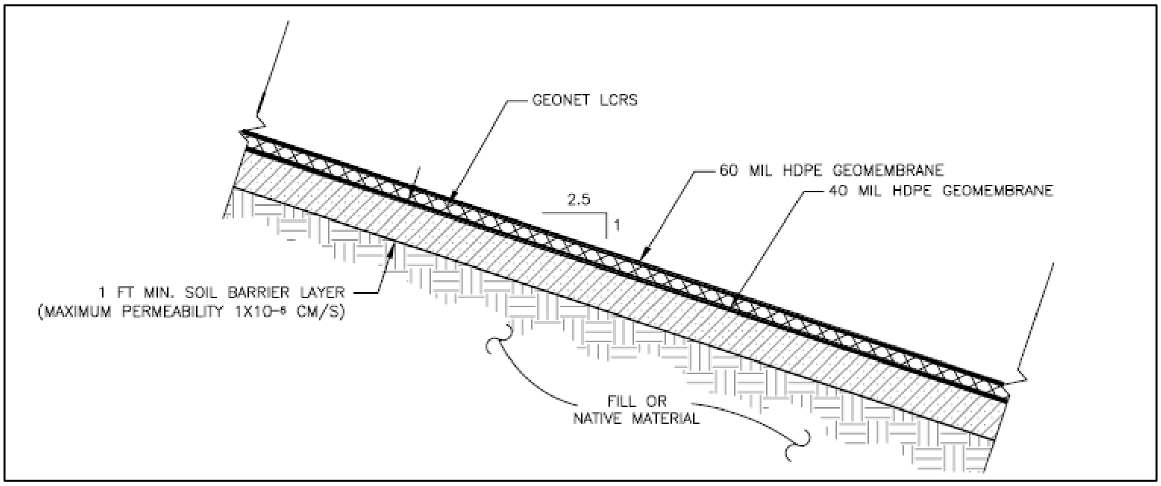


Figure 5.0-A Ore Handling Area Yield Volumes

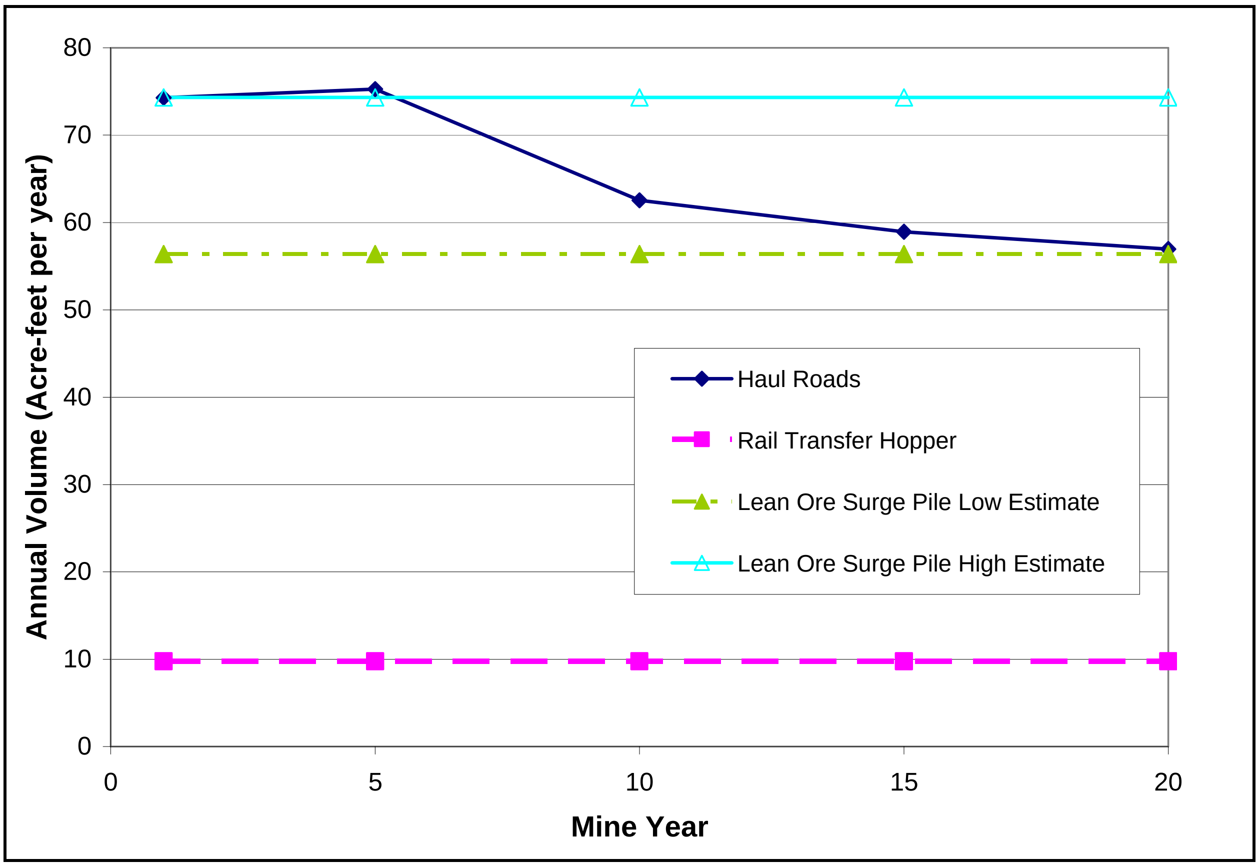


Figure 5.2-A Rail Transfer Hopper

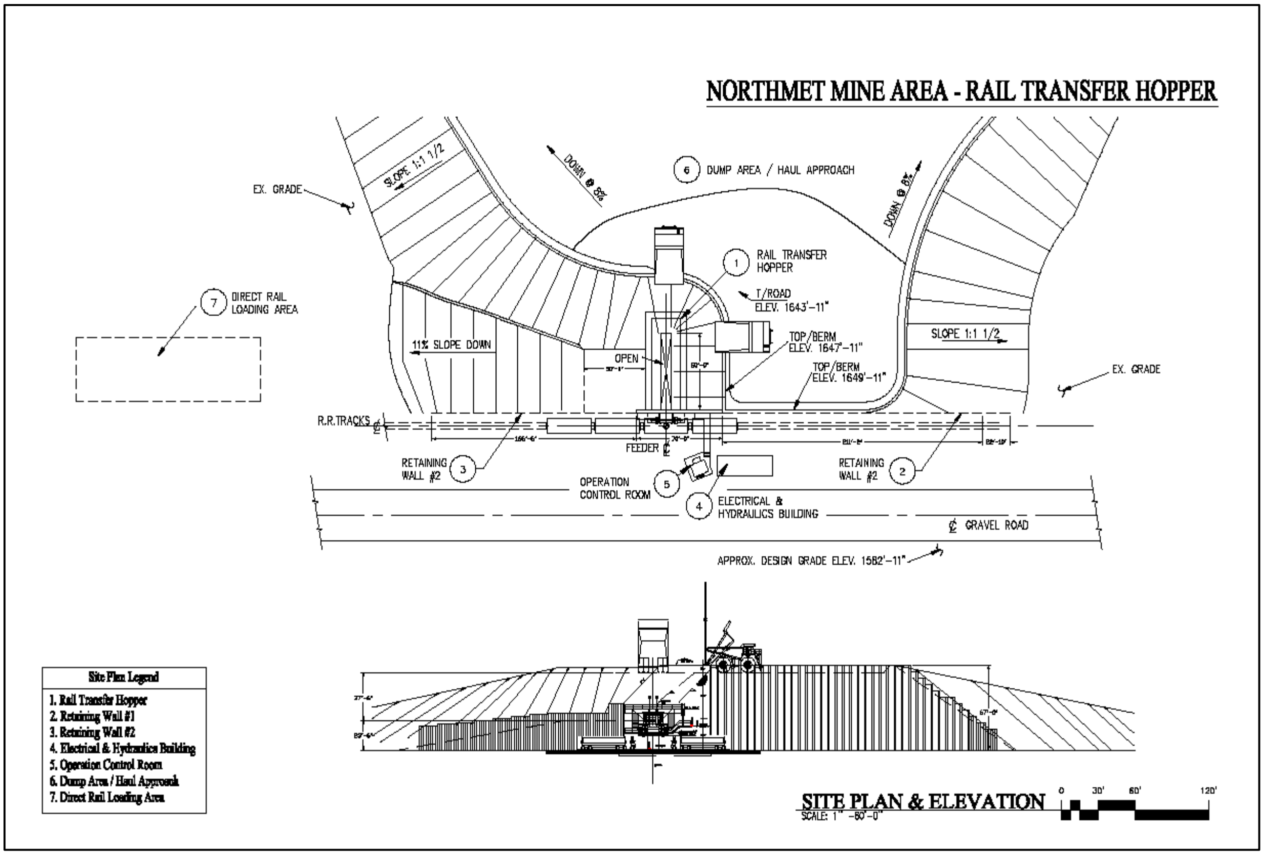


Figure 6.1-A Overburden Storage Annual Process Water Surface Runoff Volume

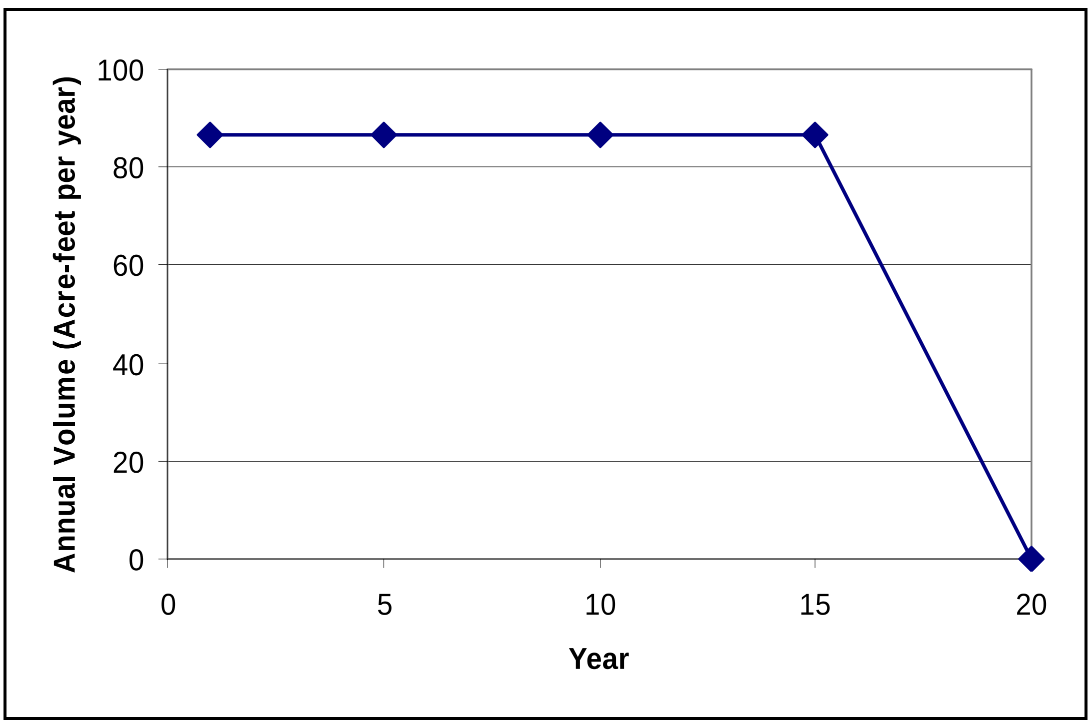
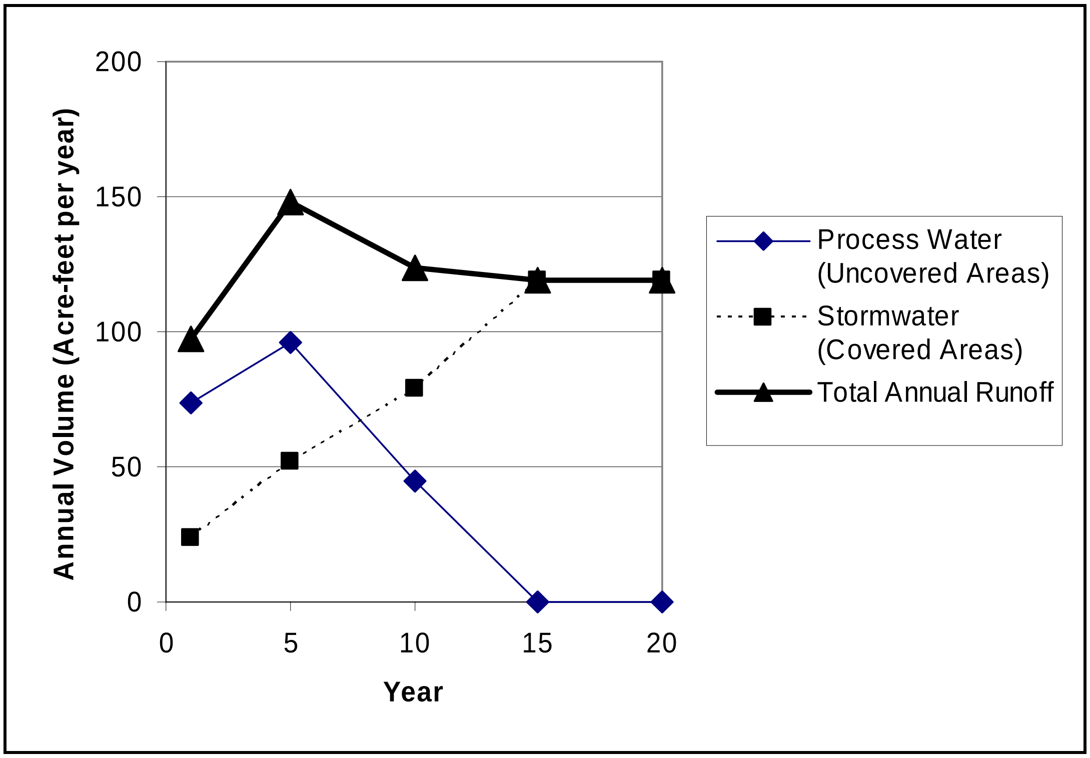
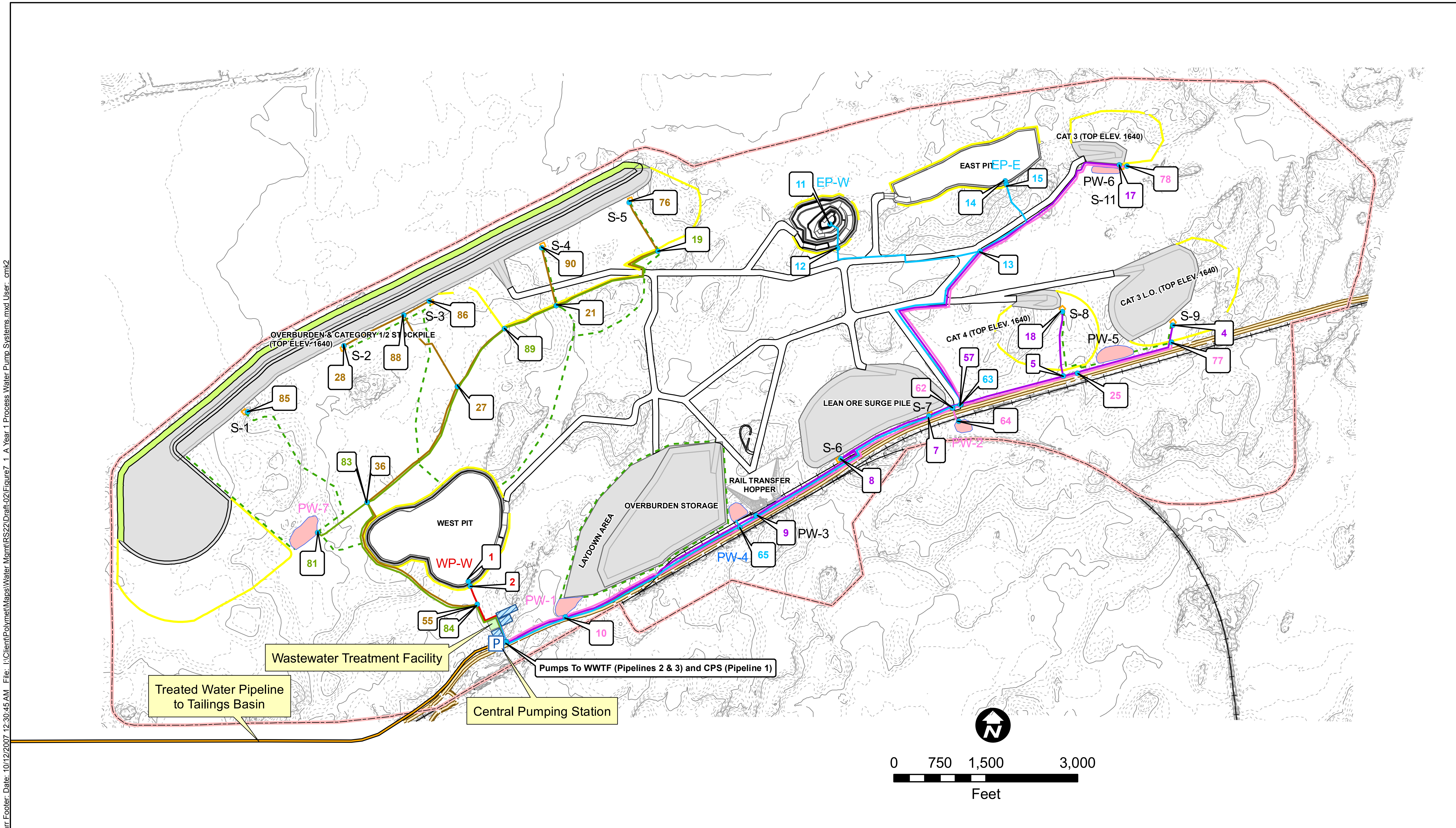


Figure 6.2-A Overburden Portion of the Category 1/2 Stockpile Annual Runoff





Year 1 Process Water
- Ditches
- Pit Rim and Stockpile Prep Dikes
- Year 1 Pumps
- Process Water Sedimentation Pond

- Pipeline 1
- Pipeline 2
- Pipeline 3
- Pipeline 4
- Pipeline 5
- Pipeline 6

Year 1 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Category 1/2 Overburden
- Stockpile Sumps
- Haul Roads

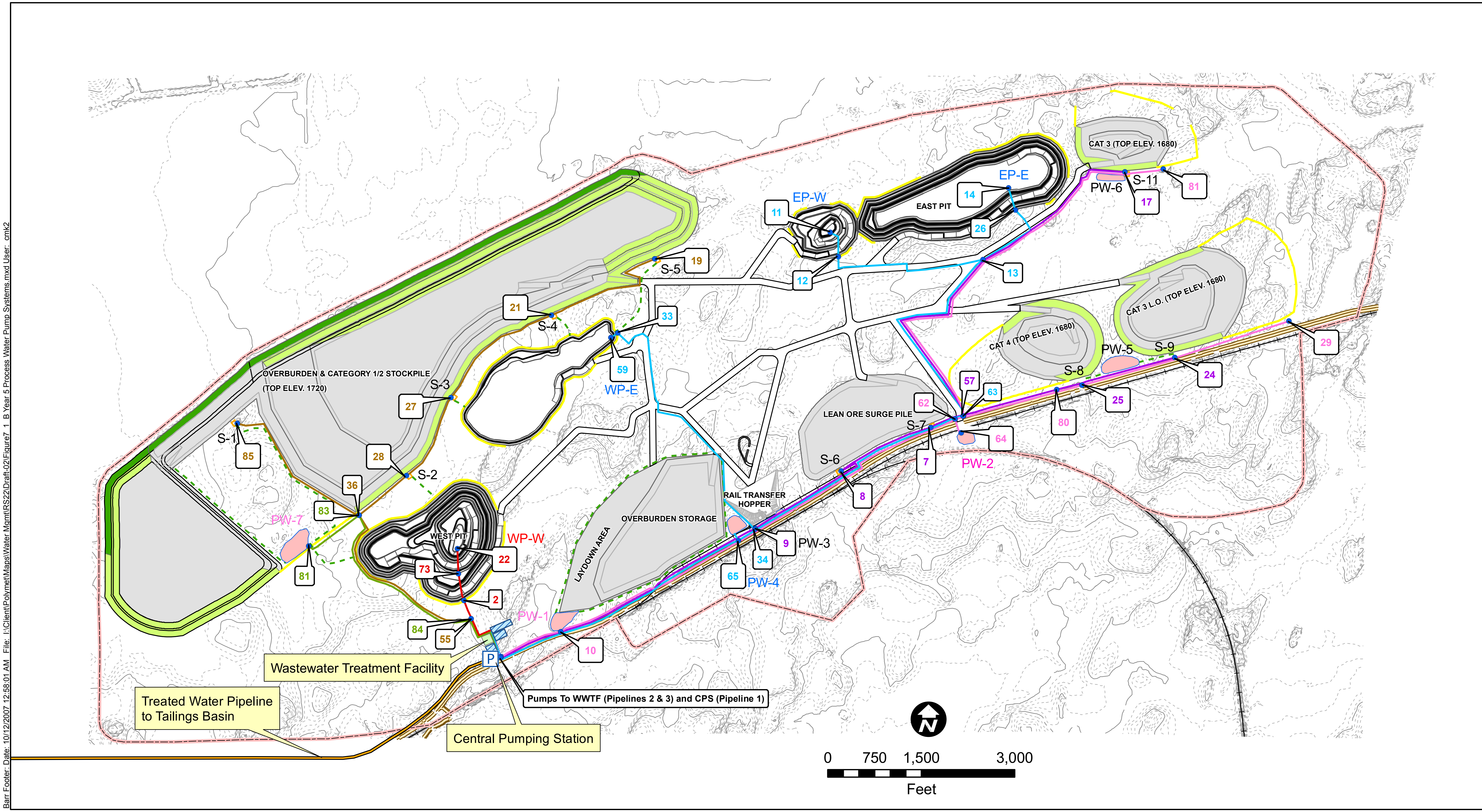
Year 1 Mine Plan (8-22-06)
- Index Contours
- Other

Note: Pipeline alignments have been exaggerated for illustration purposes.

- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
- 10 Pipeline 1 Pumps (Table 7.2.1-A)
- 65 Pipeline 2 Pumps (Table 3.2-B)
- 8 Pipeline 3 Pumps (Table 7.2.3-A)
- 2 Pipeline 4 Pumps (Table 3.2-B)
- 55 Pipeline 5 Pumps (Table 7.2.5-A)
- 75 Pipeline 6 Pumps (Table 7.2.6-A)

RS22 Figure 7.1-A
YEAR 1 PROCESS WATER PUMP SYSTEM
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Year 5 Process Water
- Ditches
- Pit Rim and Stockpile Prep Dikes
- Year 5 Pumps
- Sedimentation Ponds

- Pipeline 1
- Pipeline 2
- Pipeline 3
- Pipeline 4
- Pipeline 5
- Pipeline 6

Year 5 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Category 1/2 Overburden
- Stockpile Sumps
- Haul Roads

Year 5 Mine Plan (8-22-06)
- Index Contours
- Other

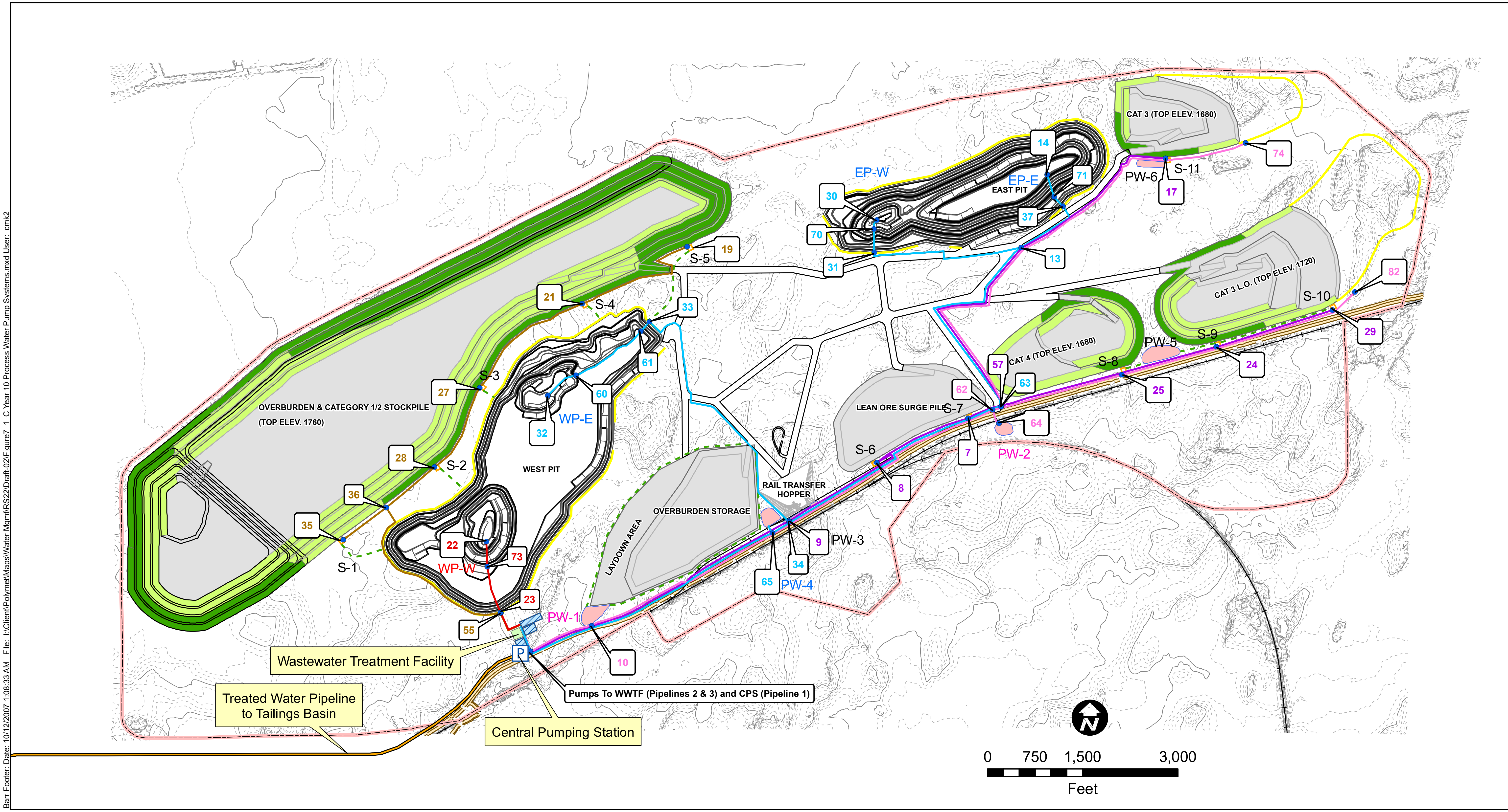
Year 5 Stockpile Covers-Cat12
- Covered - Year 5
- Covered Previous Years
- Not Yet Covered (Active)

- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
- 62 Pipeline 1 Pumps (Table 7.2.1-A)
- 65 Pipeline 2 Pumps (Table 3.2-B)
- 57 Pipeline 3 Pumps (Table 7.2.3-A)
- 22 Pipeline 4 Pumps (Table 3.2-B)
- 19 Pipeline 5 Pumps (Table 7.2.5-A)
- 84 Pipeline 6 Pumps (Table 7.2.6-A)

Note: Pipeline alignments have been exaggerated for illustration purposes.

RS22 Figure 7.1-B
YEAR 5 PROCESS WATER PUMP SYSTEM
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Year 10 Process Water
- Ditches
- Pit Rim and Stockpile Prep Dike
- Year 10 Pumps
- Process Water Sedimentation Pond

- Pipeline 1
- Pipeline 2
- Pipeline 3
- Pipeline 4
- Pipeline 5

Year 10 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Stockpile Sumps
- Category 1/2 Overburden
- Haul Roads

Year 10 Mine Plan (8-22-06)
- Index Contours
- Other

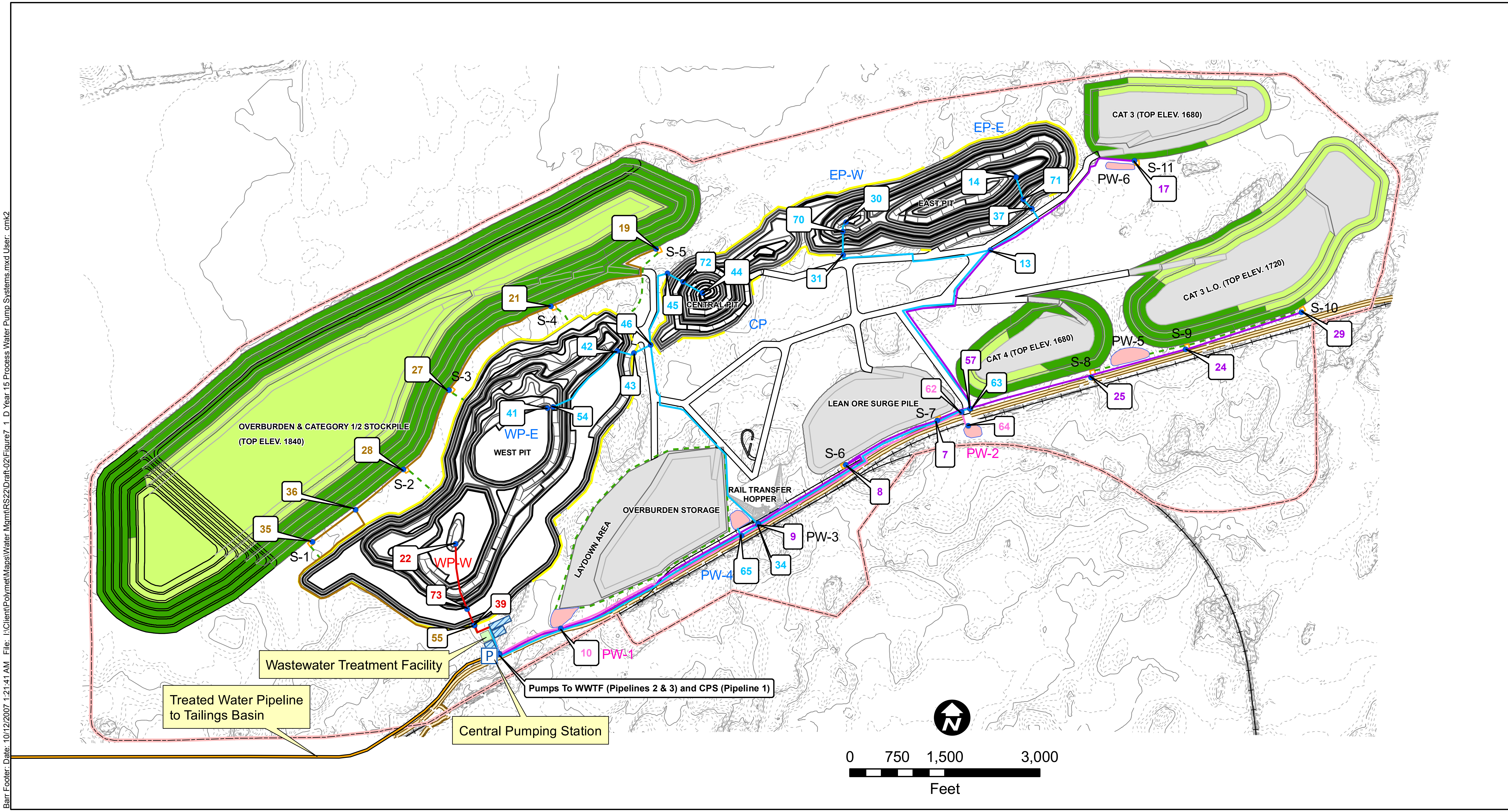
Year 10 Stockpile Covers-Cat12
- Covered - Year 10
- Covered Previous Years
- Not Yet Covered (Active)

- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
- 62 Pipeline 1 Pumps (Table 7.2.1-A)
- 65 Pipeline 2 Pumps (Table 3.2-B)
- 57 Pipeline 3 Pumps (Table 7.2.3-A)
- 22 Pipeline 4 Pumps (Table 3.2-B)
- 19 Pipeline 5 Pumps (Table 7.2.5-A)

Note: Pipeline alignments have been exaggerated for illustration purposes.

RS22 Figure 7.1-C
YEAR 10 PROCESS WATER PUMP SYSTEMS
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Barr Footer: Date: 10/12/2007 1:21:41 AM File: I:\Client\Polymet\Maps\Water Mgmt\RS22\Draft-02\Figure7_1_D Year 15 Process Water Pump Systems.mxd User: cmk2

Year 15 Process Water
- Ditches
- Pit Rim Dikes
- Year 15 Pumps
- Process Water Sedimentation Pond

- Pipeline 1
- Pipeline 2
- Pipeline 3
- Pipeline 4
- Pipeline 5

Year 15 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Stockpile Sumps
- Category 1/2 Overburden
- Haul Roads

15 Year Mine Plan (8-22-06)
- Index Contours
- Other

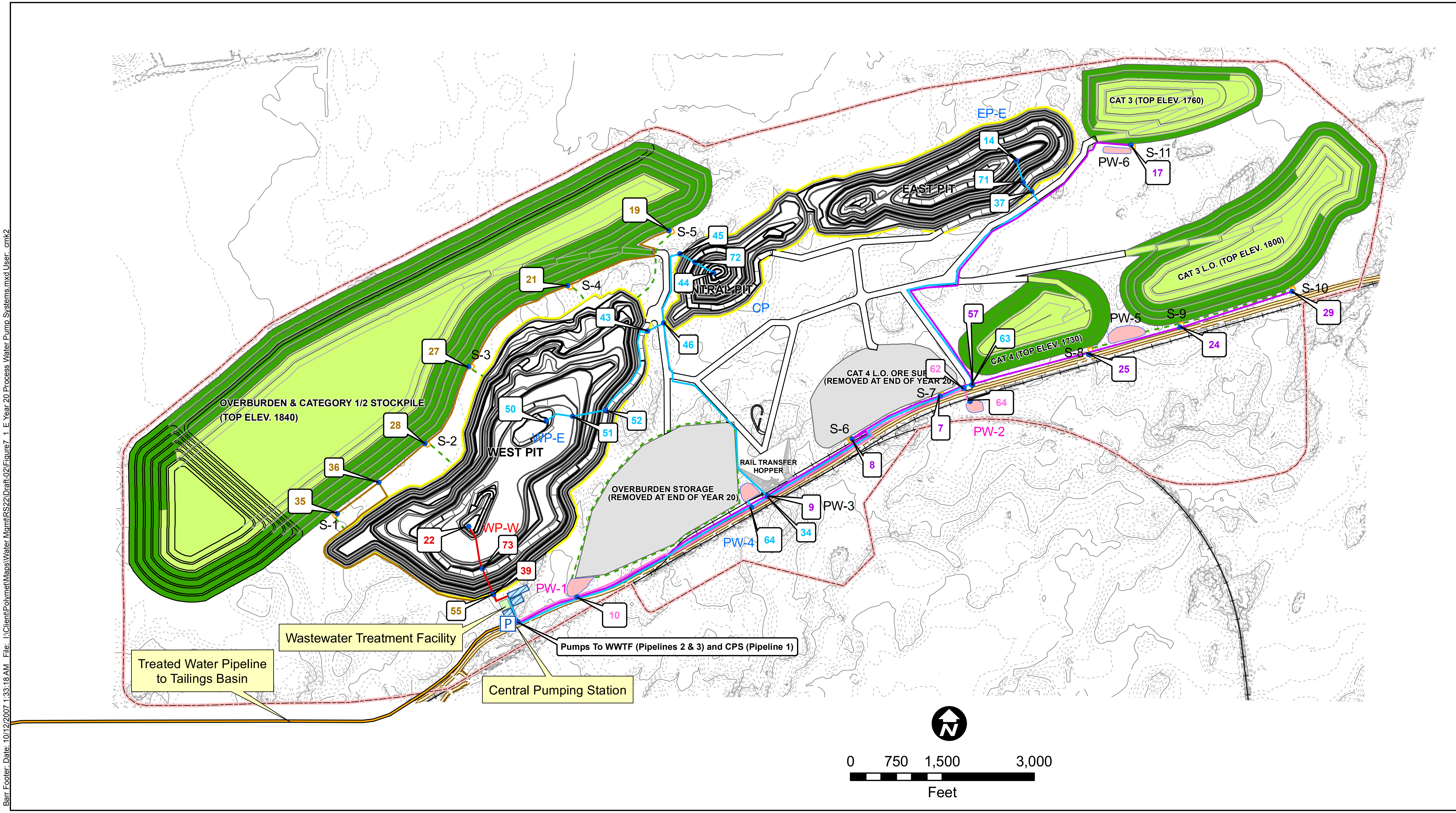
Year 15 Stockpile Covers-Cat12
- Covered - Year 15
- Covered Previous Years
- Not Yet Covered (Active)

- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
- 62 Pipeline 1 Pumps (Table 7.2.1-A)
- 65 Pipeline 2 Pumps (Table 3.2-B)
- 57 Pipeline 3 Pumps (Table 7.2.3-A)
- 22 Pipeline 4 Pumps (Table 3.2-B)
- 19 Pipeline 5 Pumps (Table 7.2.5-A)

Note: Pipeline alignments have been exaggerated for illustration purposes.

RS22 Figure 7.1-D
YEAR 15 PROCESS WATER PUMP SYSTEMS
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Year 20 Process Water
- Ditches
- Pit Rim Dikes
- Year 20 Pumps
- Process Water Sedimentation Pond

- Pipeline 1
- Pipeline 2
- Pipeline 3
- Pipeline 4
- Pipeline 5

Year 20 Stockpile Layout (7-9-07)
- Stockpile Contours
- Stockpile Breaklines
- Category 1/2 Overburden
- Stockpile Sumps
- Haul Roads

20 Year Mine Plan (8-22-06)
- Index Contours
- Other

Year 20 Stockpile Covers
- Covered - Year 20
- Covered Previous Years
- Not Yet Covered (Active)

- Mine Site
- Railroad
- Dunka Road

Callouts depict the following:
- 62 Pipeline 1 Pumps (Table 7.2.1-A)
- 65 Pipeline 2 Pumps (Table 3.2-B)
- 57 Pipeline 3 Pumps (Table 7.2.3-A)
- 22 Pipeline 4 Pumps (Table 3.2-B)
- 19 Pipeline 5 Pumps (Table 7.2.5-A)

Note: Pipeline alignments have been exaggerated for illustration purposes.

RS22 Figure 7.1-E
YEAR 20 PROCESS WATER PUMP SYSTEMS
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

Figure 7.1-F Process Water Components

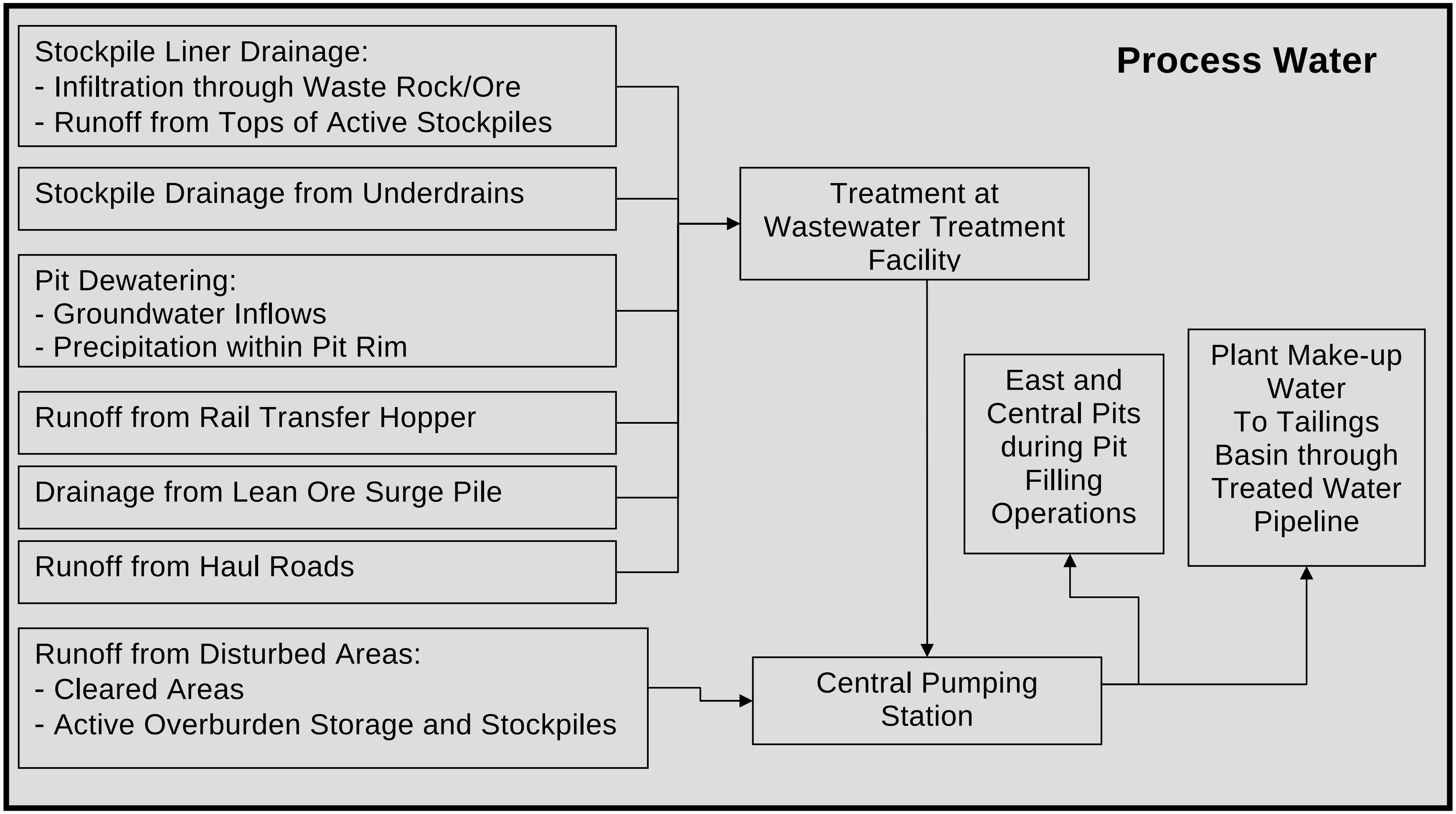
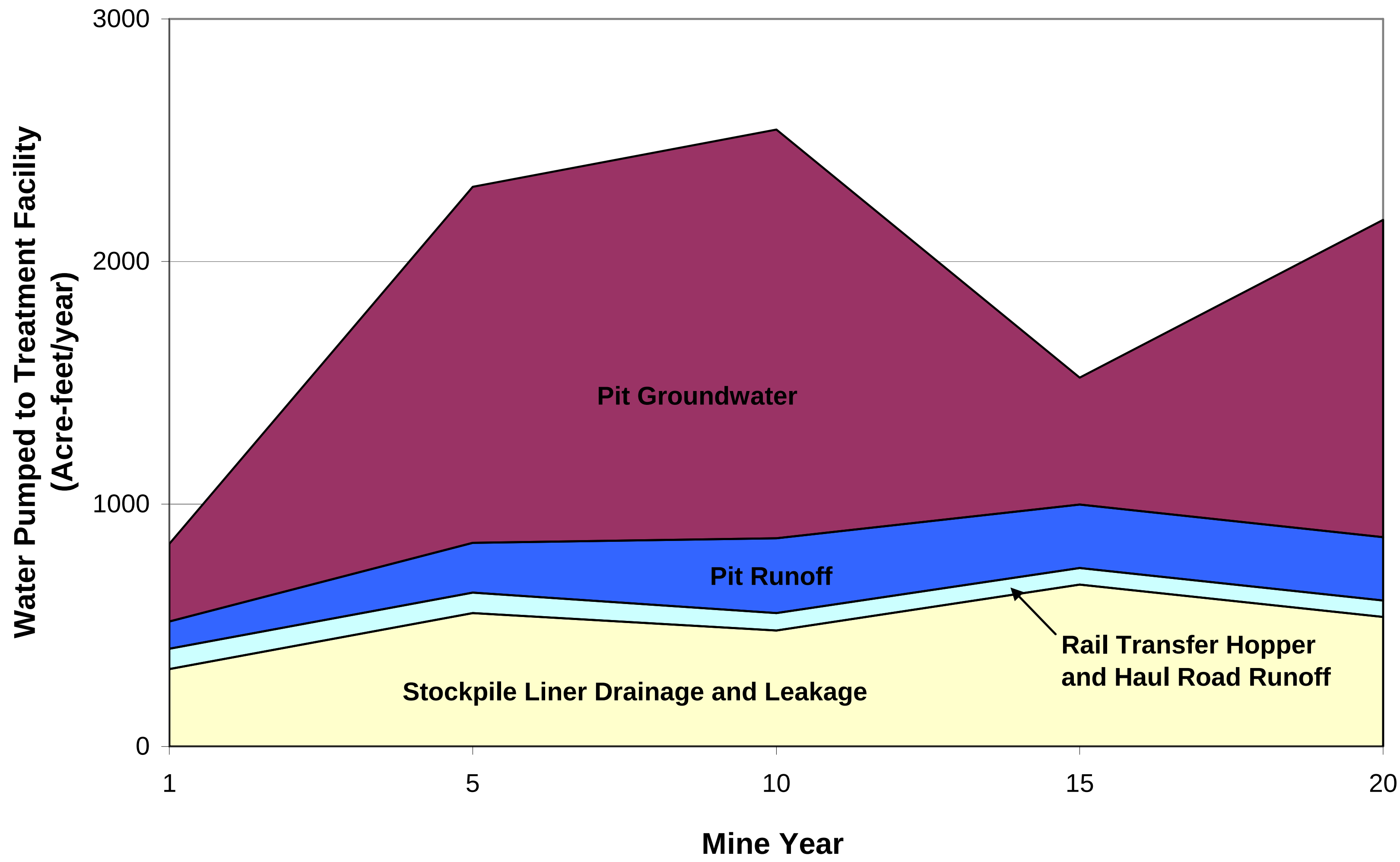


Figure 7.1-G   Water to be Treated



RS22

Figure 7.1-H Cross Section Along Dunka Road

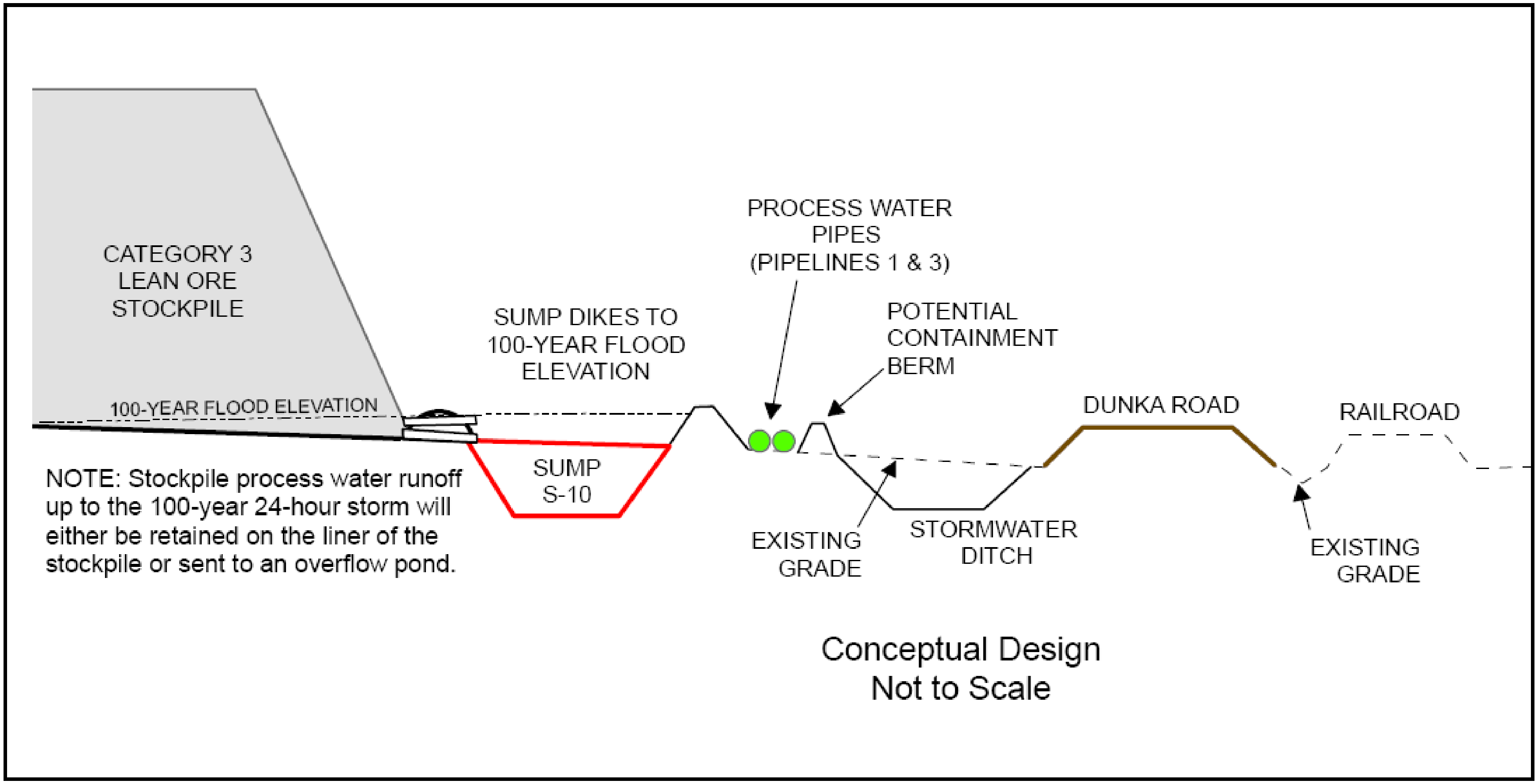


Figure 7.2-A Average Annual Flows for the Six Pipelines within the Mine Site

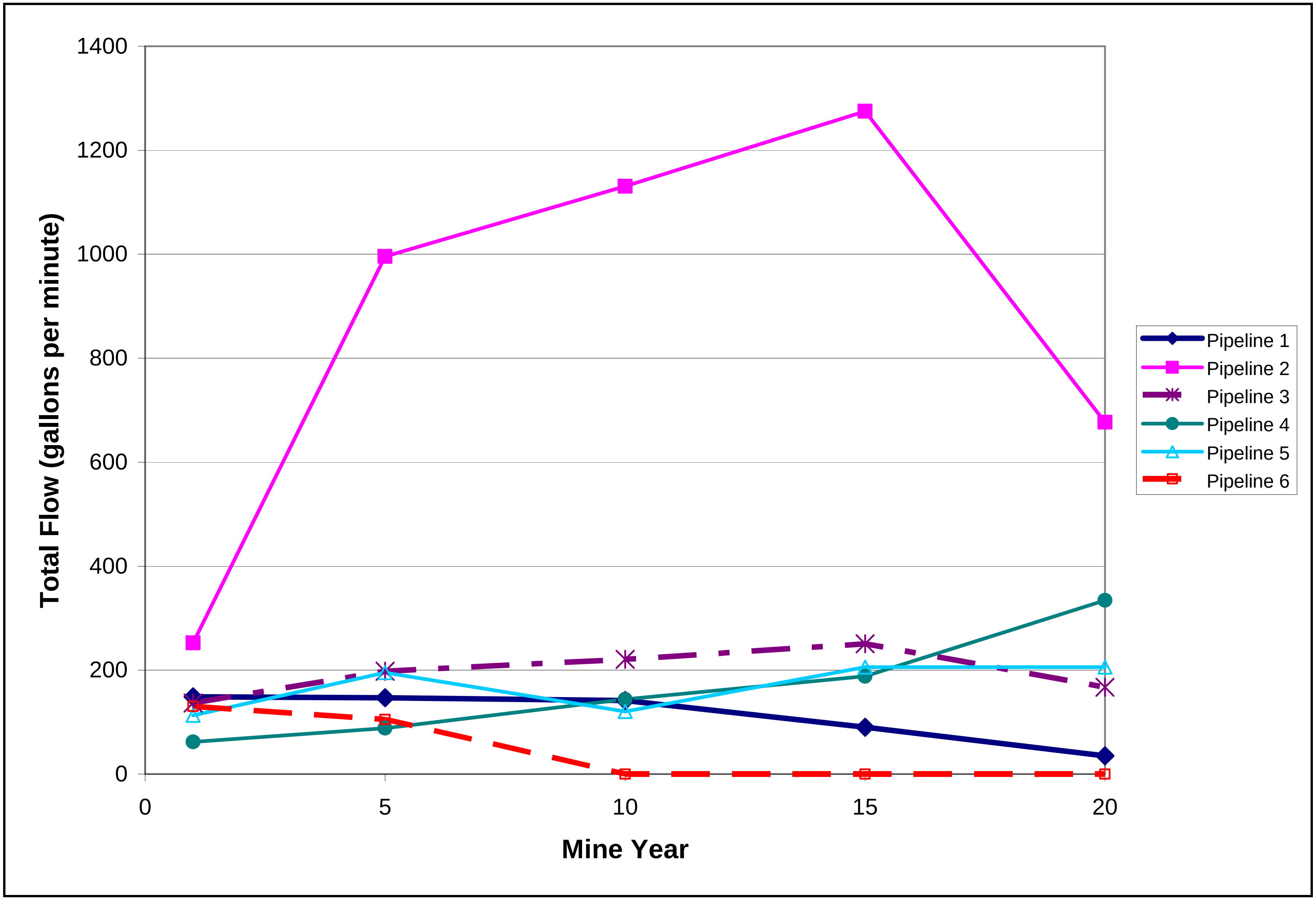
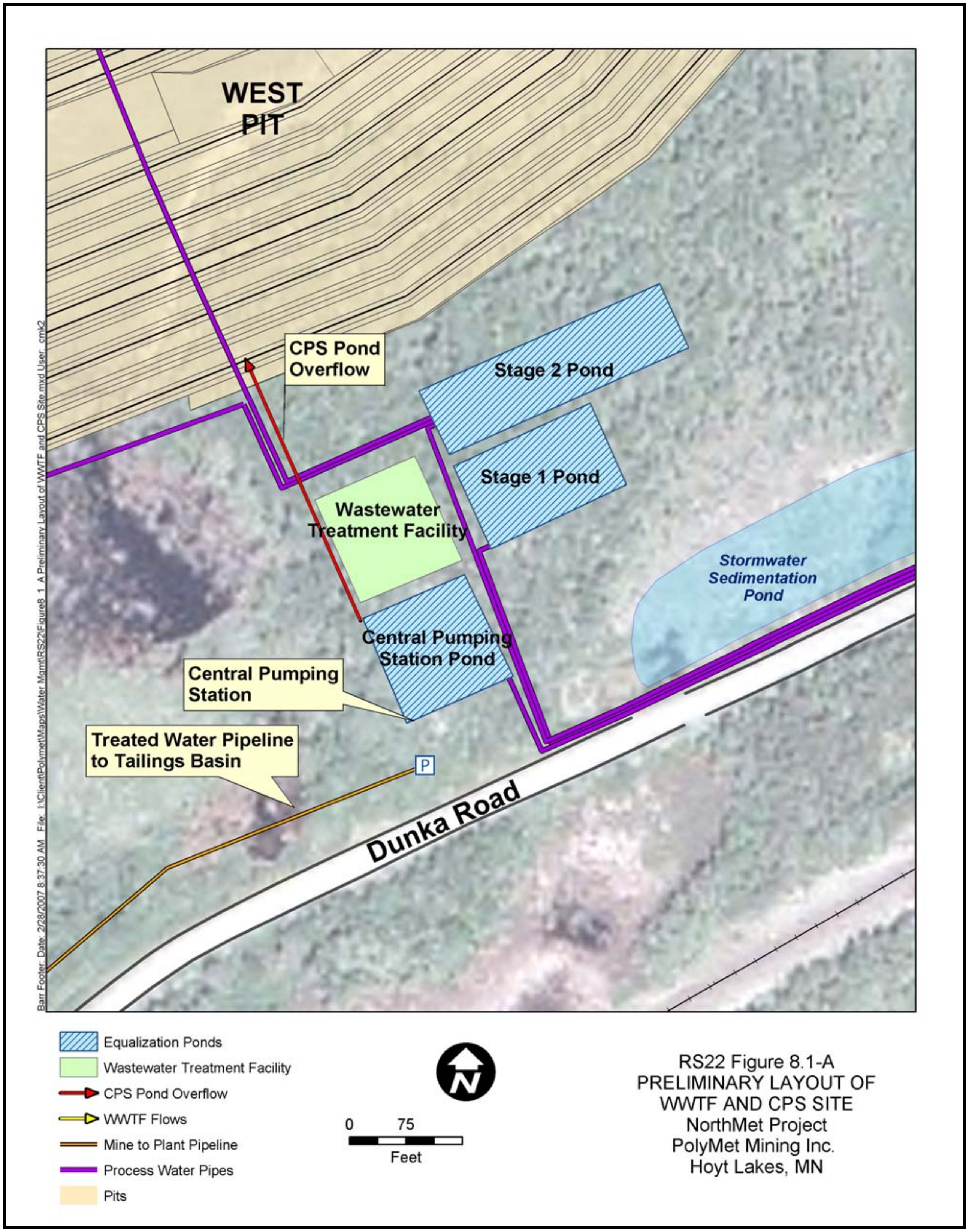


Figure 8.1-A Preliminary Layout of the WWTF and CPS Site



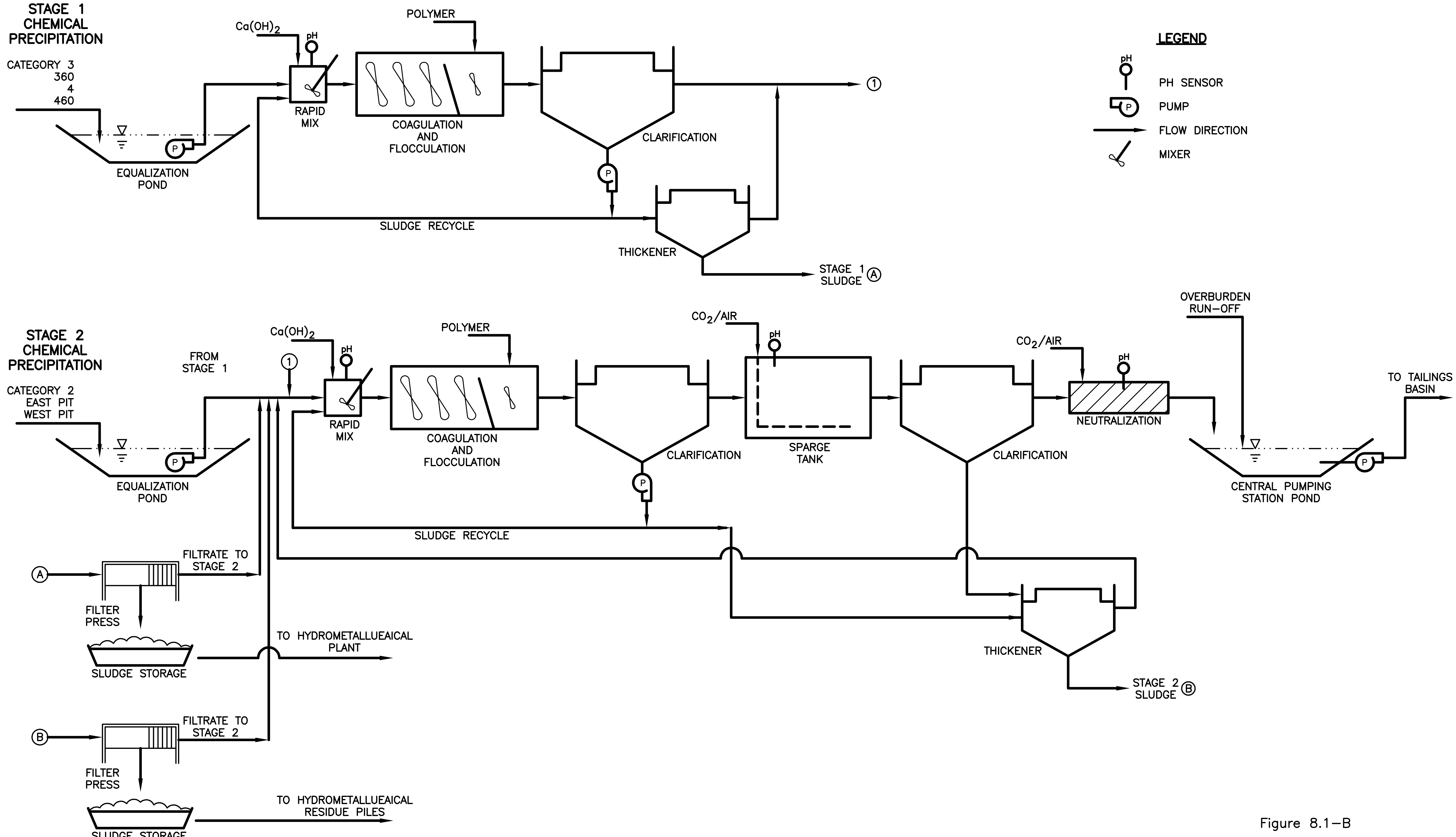
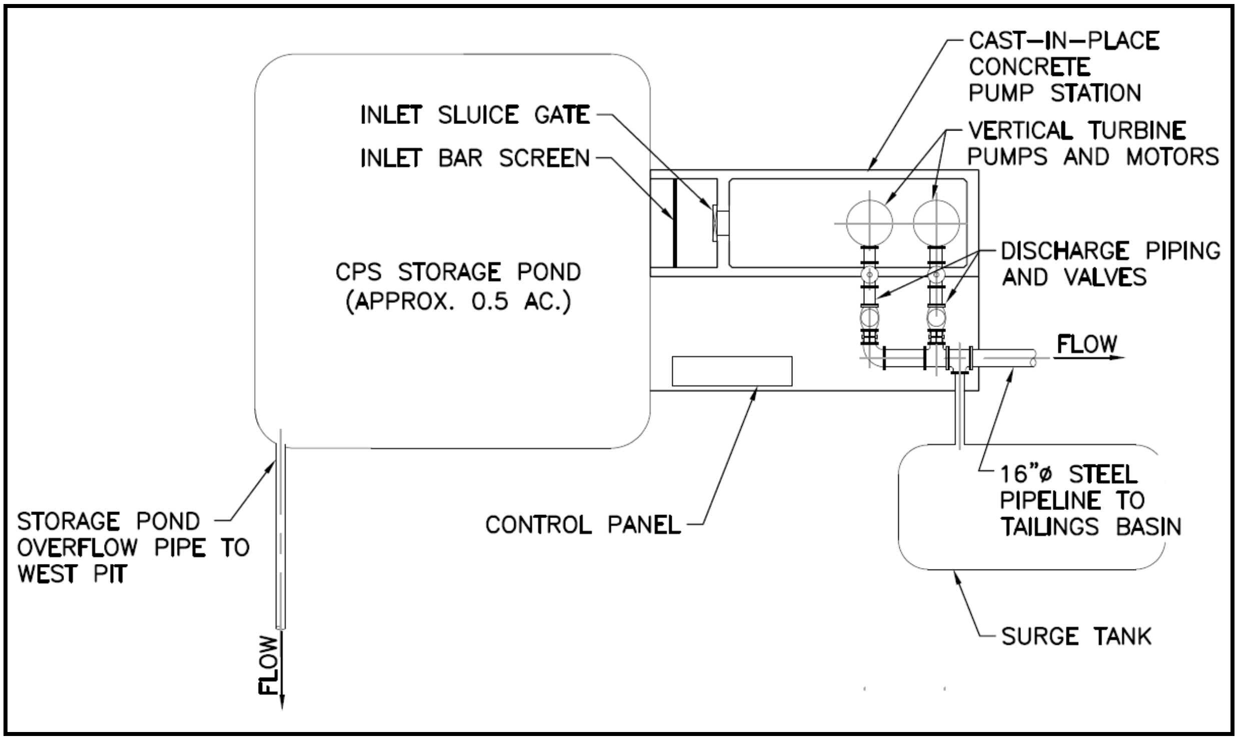
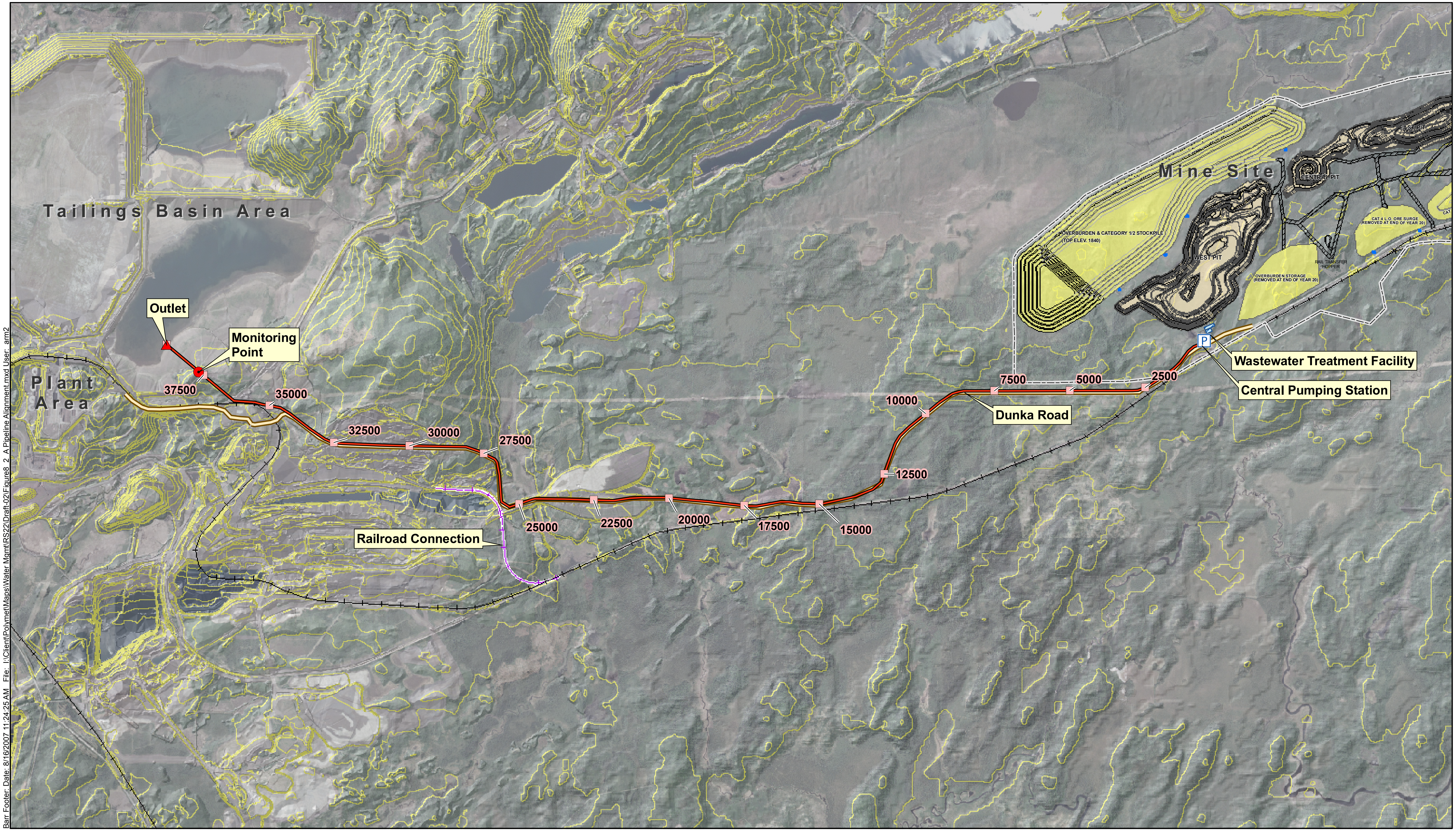


Figure 8.1–B
PROCESS FLOW DIAGRAM
WASTEWATER TREATMENT FACILITY
Mine Site Process Water
Northmet Project

AMP M:\cad\2369862\25393_1 RS.DWG Plot at 0 02/06/2007 14:04:53

Figure 8.1-C   Schematic of the Central Pumping Station

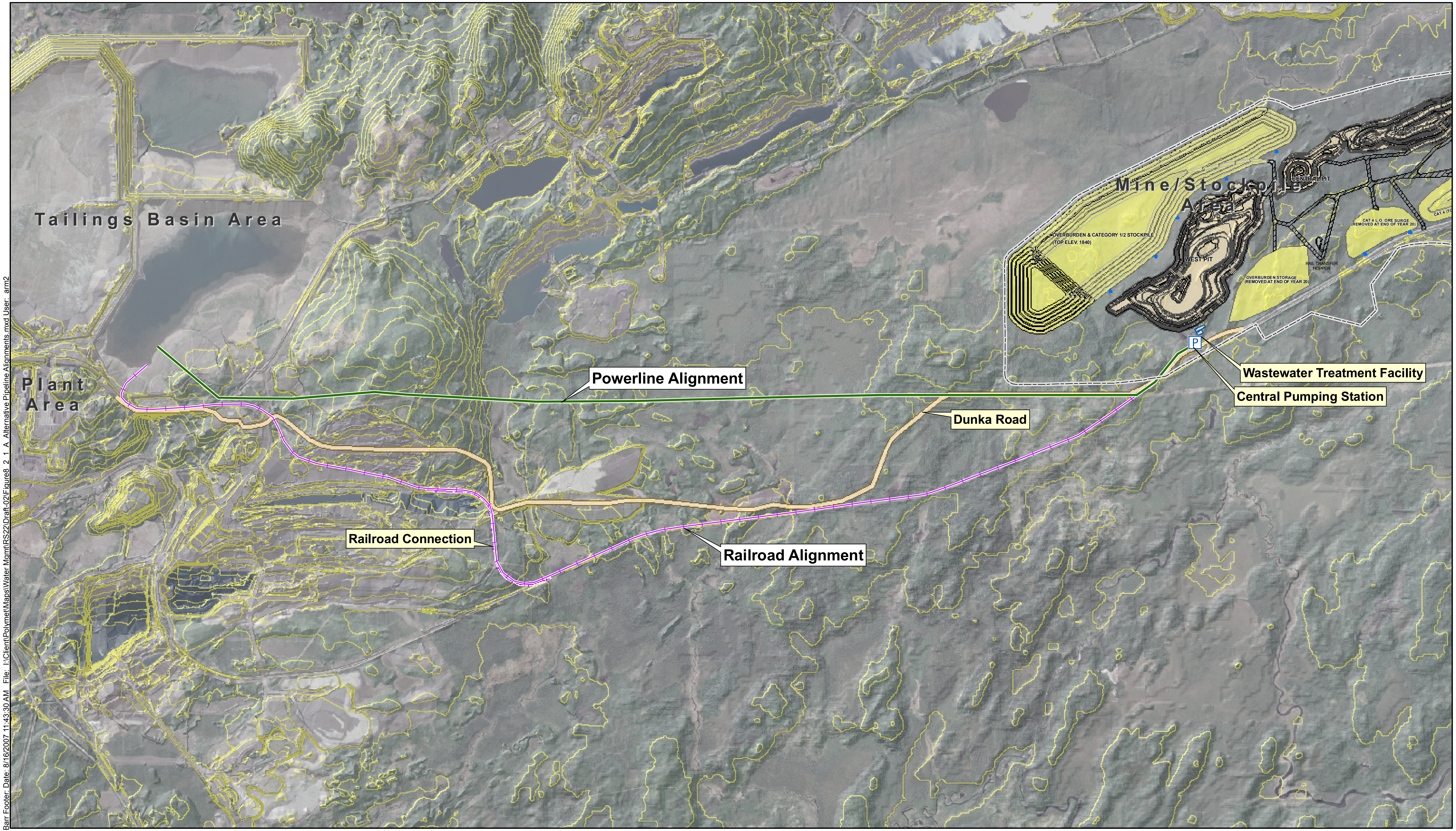




P Central Pumping Station
Monitoring Point
Outlet
Pipeline Stationing (feet)
Pipeline Alignment

0 3,000 6,000
Feet

RS22 Figure 8.2-A
TREATED WATER
PIPELINE ALIGNMENT
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Powerline Alignment
Railroad Alignment
Pits
Stockpiles

0 3,000 6,000
Feet

RS22 Figure 8.2.1-A
ALTERNATIVE PIPELINE ALIGNMENTS
NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

Figure 8.2.2.4-A Typical Road/Pipeline Cross Section – Flat Topography

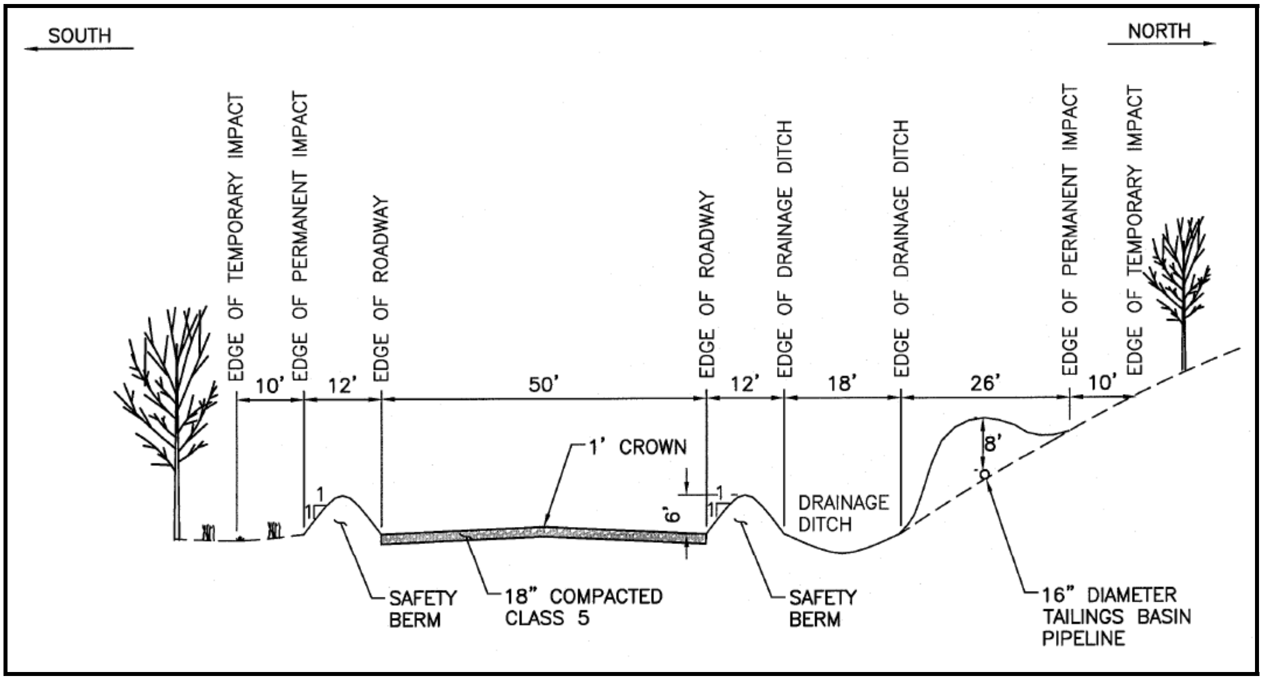


Figure 8.2.2.4-B Typical Road/Pipeline Cross Section – High Ground to the North

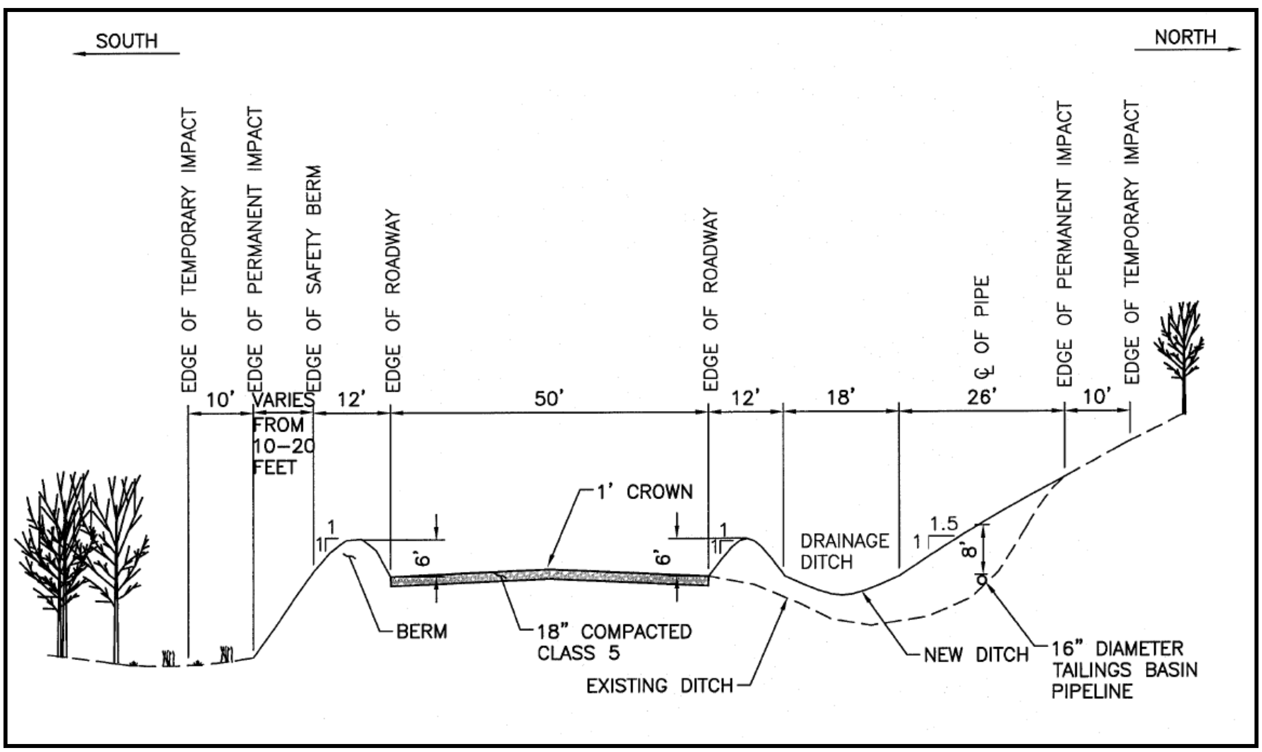
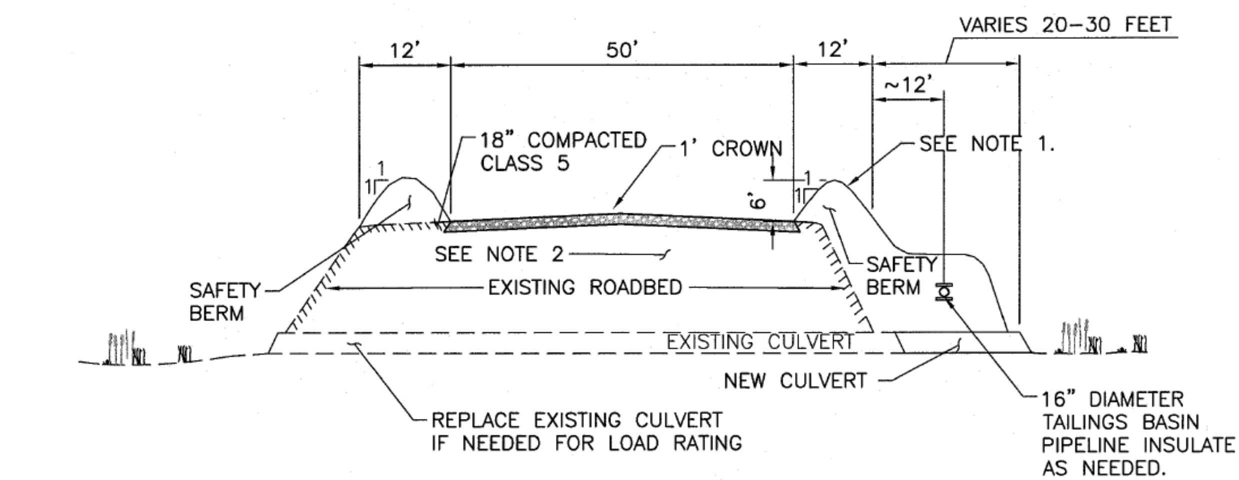


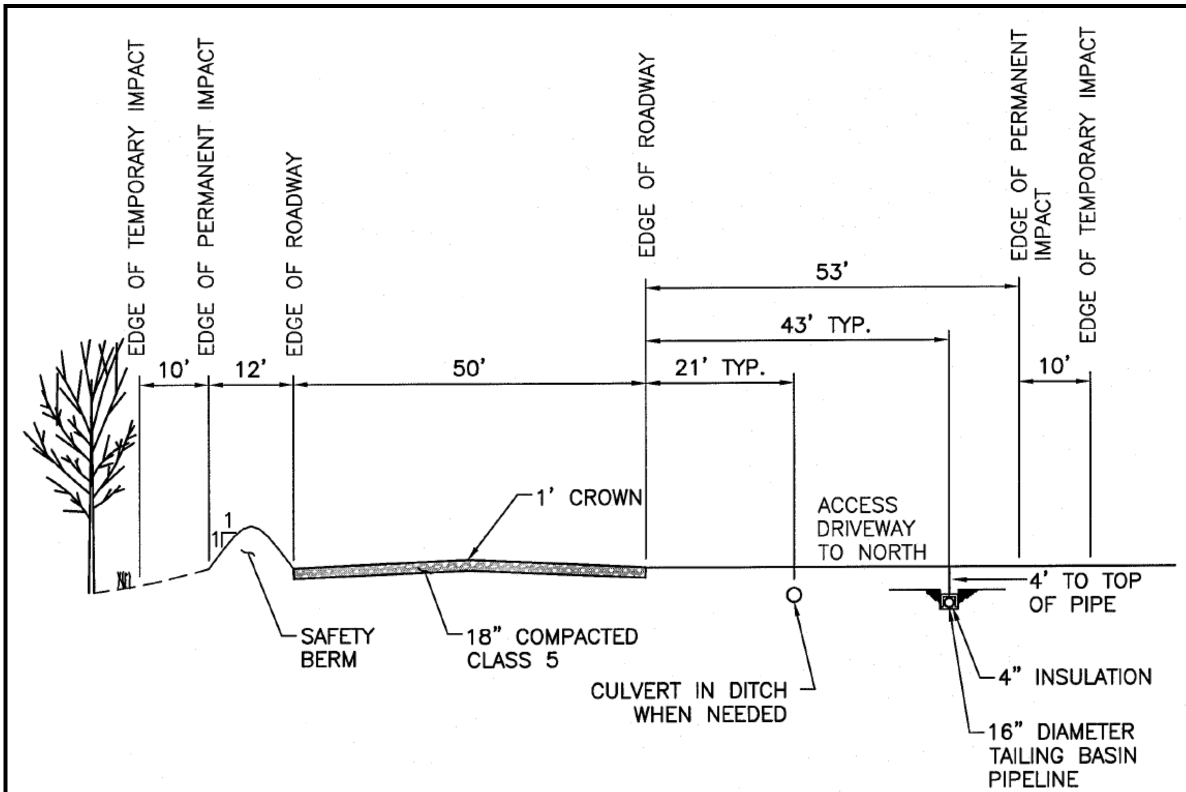
Figure 8.2.2.4-C　Typical Road/Pipeline Cross Section – Low Areas



NOTES:

1. AT LOW POINTS AND PERIODICALLY ALONG THE ROAD ALIGNMENT, DRAINAGE BREAKS WILL BE PLACED IN THE SAFETY BERM TO ALLOW WATER TO DRAIN.
2. IF APPROPRIATE, EXISTING ROAD BED MAY BE EXCAVATED LOWER TO MINIMIZE WIDTH OF IMPACT.

Figure 8.2.2.4-D　Typical Road/Pipeline Cross Section – Road Access Required



Appendix A

RS22 Work Plan

Mine Waste Water Management Systems Work Plan (RS22)

for the PolyMet NorthMet Mine Site

Prepared on behalf of
PolyMet Mining, Inc.

May 2, 2006

Mine Waste Water Management Systems Work Plan (RS22) for the PolyMet NorthMet Mine Site

Table of Contents

Figures.......... 1
1.0 Objective.......... 1
2.0 Evaluation.......... 1
2.1 Conveyance System Alignment.......... 2
2.2 Water Quantity Assessment.......... 2
2.3 Conveyance System Design.......... 4
2.4 Make-up Water Transfer.......... 4
3.0 Investigation Report and Schedule.......... 5

Figures

Figure 1 Mine Site Water Management

1.0 Objective

Reactive waste water from PolyMet's NorthMet mine site near Babbitt, Minnesota will be collected in a system that is separate from non-contact water collection systems, and will be conveyed to a central location for treatment or returned to the plant for makeup water. A conceptual design for the ditches, pipes, and ponds that convey this reactive waste water throughout the mine site will be prepared as part of this study. This Work Plan describes the design process that will be used to evaluate and manage the reactive waste water within the mine site.

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. The collection and routing of non-contact and non-reactive runoff water from the mine site will be evaluated in an earlier study (RS24 Mine Surface Water Runoff Systems). The perimeter diking system around the exterior of the mine site will be evaluated in an earlier study (RS25 Mine Diking/Ditching Effectiveness Study). Data from these three studies (RS22, RS24, and RS25) will be incorporated into the overall Mine Water Balance (RS21) and into the evaluation of Cumulative Streamflow Impacts (RS73). The runoff model used in these studies (XP-SWMM) will be calibrated to the Upper Partridge River gaging record during the analysis of existing flows as part of the Cumulative Streamflow Impacts study (RS73). The results of the Mine Site Water Balance study will be incorporated into the evaluation of Cumulative Streamflow Impacts (RS73), and used to simulate mining-altered runoff from the Polymet mine site. All of the water management studies listed above will be based on the Mine Site Plan (RS17). Runoff yield from stockpiles will be developed 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.

The goal of the reactive waste water system design is to ensure that waste water drainage that does not meet water quality standards is collected for treatment prior to discharge. The collection system must perform so that water that escapes the collection system does not create adverse impacts to surface water or groundwater quality. The design of monitoring systems to ensure that this performance standard is met will be addressed in a separate report.

This system will be designed to accommodate surface water runoff from all potential sources of reactive runoff. Due to uncertainty of the amount and nature of reactive material, this will need to be a conservative design that can convey surface water from all stockpiles. The design will also consider the critical phase for runoff volumes, depending on whether the stockpiles are open, capped, or partially filled.

Additional alternatives/options that may be evaluated in the EIS, which have different reactive drainage, will need to be evaluated in a separate report.

2.0 Evaluation

The waste water management plan will be considered in five components:

1) Conveyance system alignment – locations for the system components will be identified
2) Conveyance system design – type, size, and slopes will be designed to accommodate runoff from the project design storm
3) Pond design – temporary holding ponds will be designed to store water during the large storm events
4) Make-up water transfer – the pump and pipeline system required to transport make-up water to the plant will be evaluated

Additional description of these components is provided in the following paragraphs.

2.1 Conveyance System Alignment

The location of the conveyance system for reactive waste water will be determined based on the final site plan and phasing (RS17), the stockpile collection system locations as determined in the Stockpile Design Report (RS49) and the Reactive Waste Segregation System Report (RS23T), the location of other site facilities and other conveyance systems for non-contact and non-reactive water, and the location of the plant facility. The recommended scenario(s) provided by RS23T will be evaluated (e.g. separate collection systems for segregated waste stockpiles and/or segregated collection systems for a combined stockpile). The segregated waste stockpile systems may require controls to re-direct flows depending on the water quality concentrations from each zone. The combined stockpile system would have a much simpler design with two collection systems: one for reactive and one for non-reactive.

This information will be used to define the conveyance required during various years (e.g. years 1, 5, 10, 15, and 20 of mine operation). The phasing plans will be evaluated to determine the phase that would create the largest flows for each segment, which will be used for the conceptual design. The conceptual designs will be summarized and submitted in a memo to the MnDNR for review prior to completion of the report.

2.2 Water Quantity Assessment

The conveyance systems within the mine site will be designed to accommodate the expected storm flows for reactive water from capped and open stockpiles in various stages and from undisturbed areas within the mine site. Non-reactive water flows will also be considered in the design, if they have a potential for requiring treatment. The flows will be determined using the XP-SWMM computer model[1]. The system will be sized for discharges expected as a result of a single design storm event (e.g., 10-year SCS Type II 24-hour storm event). A frequency of the design storm event will be selected based on available guidance in state rules or other applicable regulations. The selection rationale will be summarized in a memo and discussed with the MnDNR prior to finalizing the analysis. Several storm durations will be analyzed and the storm duration that produces the largest peak flow rates and/or the largest storage requirements (critical event) will be used for the design.

The XP-SWMM model is an unsteady flow model that can be used for design of storm sewer systems. 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 completed for this study.

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 roads and areas that are directly connected to the storm sewer system. Impervious surfaces draining onto

---

[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.

adjacent pervious areas would not be treated as effective impervious areas. This system being analyzed is primarily comprised of impervious areas that are directly connected to the storm sewer system. Watershed "width" in XP-SWMM is used along with velocity and channel length to compute the time of concentration. The width is typically 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. We expect there will be very little infiltration at the Mine Site due to the hard surfaces, although there may be some from the non-contact areas. 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 the model calibration conducted in RS73. 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).
- 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 the model calibration conducted in RS73.
- 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 also be based on the model calibration conducted in RS73.

The routing data that is required by the model will be based on the preliminary design, as described in the section below, and includes: (a) pipe locations, sizes, types, materials, and elevations; (b) channel cross-sections; (c) storage basin elevation, volume, and outflow characteristics; and (d) surface flow characteristics (overland flow upstream of the channels).

Several duration events will be analyzed to determine the storm that produces the peak discharge, which will be considered the critical event. The shape of the synthetic storm events will be obtained from published data; the SCS 24-hour Type II and the Huff's distribution for shorter duration events (e.g. 1-hour and 6-hour). Snowmelt events are typically not critical for storm sewer design but may be critical for design of the sediment pond, therefore a 10-day snowmelt event will also be analyzed.

2.3 Conveyance System Design

A preliminary design will be prepared for a conveyance system within the mine site to accommodate the expected reactive water flows. Flow rates will be determined using information from the Stockpile Design Report (RS49), the Reactive Waste Segregation Report (RS23T), and hydrologic modeling of reactive runoff from storm flows within the mine site. The flows will be determined by modifying the XP-SWMM model that was developed for the Mine Diking/Trenching study (RS25) and modified for the Mine Surface Water Runoff Systems study (RS24).

Rationale for selection of a design frequency for the storm event will be summarized in a memo and discussed with the MnDNR prior to finalizing the hydrologic analysis. The system will be sized for the expected flows during this design event.

Storm durations of 1 hour, 6 hours, 24 hours, and 10 days will be modeled. The storm duration that produces the largest peak flow rates and/or the greatest storage requirement (depending on the design feature being considered) will be used for the design.

Hydrologic impacts of the stockpile design presented in the Stockpile Design Report (RS49) and the Reactive Waste Segregation Report (RS23T) will be incorporated in the collection system design. For example, capping systems on the stockpiles will reduce the amount of precipitation passing through the stockpile but may increase runoff, and liner systems will capture most of the water that leaches through the stockpile. All of the information on stockpile seepage and runoff will be obtained directly from the stockpile design reports listed above.

Liner systems for ditches and ponds transporting or storing reactive wastewater will be evaluated. Part of this evaluation will include the effectiveness of the system to prevent leakage and the operation and maintenance requirements. The Stockpile Design Report (RS49) will provide the effectiveness of preventing leakage from stockpile areas. Information for this evaluation will be obtained from the Reactive Waste Segregation study (RS23), from the Phase II Hydrogeological Investigations (RS2), and from other readily available sources. Plans will minimize the use of ditches by using pipelines wherever practical.

Ponds will be designed for temporary detention of waste water emanating from the conveyance systems prior to treatment. The XP-SWMM model will be used to determine the pond sizes required to temporarily store the stormwater runoff expected for the design storm event. Several storm durations (identified above) will be analyzed and the storm duration that requires the largest pond volume(s) will be used for the design. The average discharge rate from the pond(s) will be defined for a range of pond volumes and plotted as curves; the curves will be used in task RS29 to optimize the treatment system. These curves will be modified during RS21 Mine Water Balance study to include the reactive water from the pit that must also be treated. The possibility of overflow from the pond(s) due to storm events that exceed design conditions will be evaluated and discussed. The designs will be evaluated under average, wet, and dry cycles to define the frequency of overtopping during various historic long-term precipitation cycles. This data will provide a more comprehensive view of the risk of overflows. The potential impacts of any uncontrolled overflow from the pond(s) will also be investigated, along with options to minimize the impacts. This evaluation will be summarized and submitted to the MnDNR for review prior to the completion of the report.

2.4 Make-up Water Transfer

Portions of the waste water could be recycled to the plant and used as make-up water. The study will include a conceptual design of the pump and pipeline system that would be required. The location of the pump and pipeline to the plant will be determined based on the locations of stockpiles and other site facilities. Preliminary estimates of pipeline design parameters (capacity, size, type, lengths, etc.) and the pump type and capacity will be provided. The portion of waste water that is transferred to the plant will be estimated based on make-up water requirements, constraints on the conveyance system to the plant, and potential flow impacts to the Partridge River.

3.0 Investigation Report and Schedule

The results of the waste water management systems conceptual design and analyses will be summarized and incorporated into the Mine Waste Water Management Systems Report. The report will include information on the methodology and results from the hydrologic and hydraulic modeling, system alignment and configuration, holding pond design, make-up water transfer system design, along with conclusions and recommendations. Documentation supporting the analyses and results will be included in tables, figures, and appendices, as appropriate.

The study report will list the assumptions made and the modeling methods will be explained.

The majority of the work in this plan will be delayed until the final Mine Site Plan is finalized. The target date for submittal of the draft report is 7 weeks after receipt of the Stockpile Design Report (RS49). Delays in the Stockpile Design Report (RS49) and the waste characterization studies may impact this schedule.

Mine Waste Water Management Systems Work Plan (RS22)
for the PolyMet NorthMet Mine Site

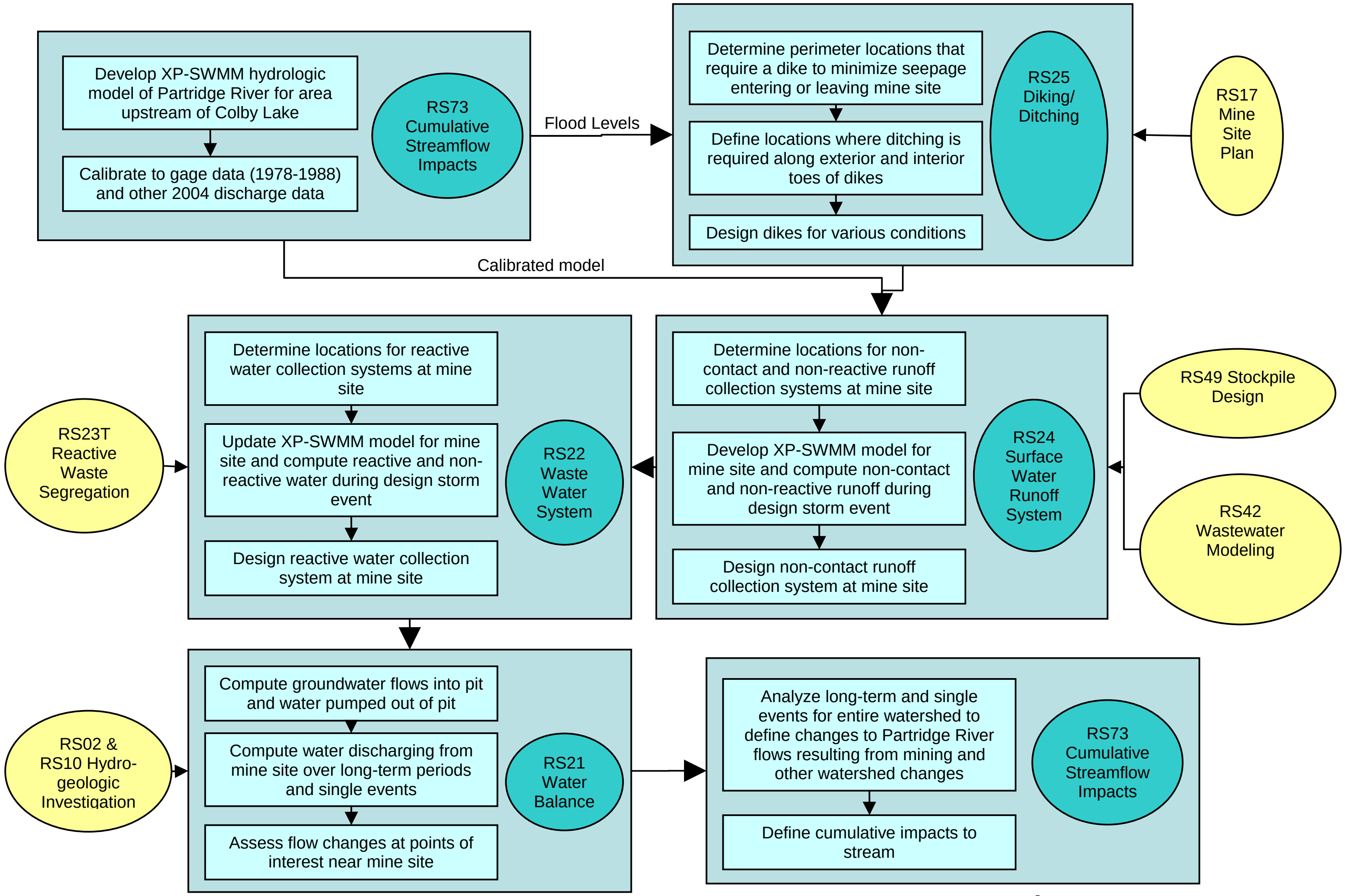


Figure 1
MINE SITE WATER MANAGEMENT

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.

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Appendix B

Groundwater Modeling

Appendix B
Groundwater Modeling of the NorthMet Mine Site
Draft-02

Table of Contents

1.0 Introduction ... 1
1.1 Objectives ... 1
1.2 Background ... 1
1.3 Report Organization ... 2
2.0 Conceptual Model ... 3
2.1 Geologic Units ... 3
2.1.1 Bedrock ... 3
2.1.2 Surficial Deposits ... 4
2.2 Sources and Sinks for Water ... 4
2.3 Local Flow System ... 5
2.4 Hydrologic Model Selection ... 5
3.0 Modeling Approach ... 7
3.1 Regional Model ... 7
3.1.1 Model Grid and Layers ... 7
3.1.2 Boundary Conditions ... 7
3.1.3 Hydraulic Conductivity Distribution ... 8
3.1.4 Calibration ... 8
3.2 Local-Scale Model ... 8
3.2.1 Model Grid and Layers ... 8
3.2.2 Boundary Conditions ... 9
3.2.3 Hydraulic Conductivity Distribution ... 10
3.2.4 Recharge Distribution ... 11
3.2.5 Storage Parameters ... 11
3.2.6 Model Calibration ... 12
3.3 Assumptions and Limitations of the Model ... 13
4.0 Model Simulations and Results ... 14
4.1 Mine Operation ... 14
4.1.1 Simulations ... 14
4.1.2 Results ... 15
4.1.2.1 Groundwater Inflow Rates ... 15
4.1.2.2 Impacts to Partridge River ... 15
4.2 West Pit Filling ... 16
4.2.1 Simulations ... 16
4.2.2 Results ... 16
4.3 Long Term Closure ... 16

4.3.1 Simulations ... 17
4.3.2 Results ... 17
4.4 Sensitivity Analysis ... 18
5.0 Summary and Conclusions ... 22
6.0 References ... 24

List of Tables (embedded)

Table 3-1 Hydraulic Conductivity Values used in the Regional Model ... 8
Table 3-2 Model Layer Bottom Elevations ... 9
Table 3-3 Hydraulic Conductivity Values used in Local-Scale Model ... 11
Table 3-4 Storage parameters used in the Local-Scale model ... 12
Table 4-1 Predicted Groundwater Flow Rates during Mine Operations ... 15
Table 4-2 Predicted Percent Reduction in Partridge River Baseflow ... 16
Table 4-3 Predicted groundwater inflow and outflow rates for the pits in post-closure ... 18
Table 4-4 Summary of Sensitivity Analysis ... 20

List of Figures (end of report)

Figure 1-1 NorthMet Mine Site (Year 20 Pit Layout)
Figure 2-1 Mine Site Bedrock Geology
Figure 2-2 Geologic Cross Section A-A’
Figure 2-3 Geologic Cross Section B-B’
Figure 2-4 Average Water Levels Measured in Site Piezometers
Figure 2-5 Groundwater Contours and Heads within the Bedrock at the NorthMet Mine Site
Figure 3-1 Model Areas
Figure 3-2 Model Boundaries in the Regional Model
Figure 3-3 Regional Model Calibration Results
Figure 3-4 Model Boundaries in the Local-Scale Model
Figure 3-5 Local-Scale Model Calibration Results – Surficial Aquifer
Figure 3-6 Local-Scale Model Calibration Results – Bedrock Aquifers
Figure 4-1 Predicted Groundwater Levels within the Surficial Deposits – Year 20
Figure 4-2 Predicted Groundwater Levels within the Bedrock – Year 20
Figure 4-3 Predicted Groundwater Inflow Rates During West Pit Filling
Figure 4-4 Predicted Groundwater Levels within the Surficial Deposits – Closure
Figure 4-5 Predicted Groundwater Levels within the Bedrock – Closure
Figure 4-6 Sensitivity Analysis – Duluth Complex Hydraulic Conductivity

Figure 4-7 Sensitivity Analysis – Virginia Hydraulic Conductivity
Figure 4-8 Sensitivity Analysis – Surficial Deposits Vertical Hydraulic Conductivity
Figure 4-9 Sensitivity Analysis – Bedrock and Surficial Aquifer Specific Storage
Figure 4-10 Sensitivity Analysis – Surficial Deposits Specific Yield
Figure 4-11 Sensitivity Analysis – Bedrock Aquifer Specific Yield
Figure 4-12 Sensitivity Analysis – River Cells Vertical Hydraulic Conductivity
Figure 4-13 Sensitivity Analysis – General Head Boundary Conductance

List of Attachments

Attachment 1 Technical Memorandum on NorthMet Bedrock Groundwater Elevation Measurements

Supplemental Data

Groundwater modeling files provided upon request

List of RS Documents Referenced

RS02 Hydrogeological – Drill hole monitoring and data collection – Phase 1

RS10 Hydrogeological – Drill hole monitoring and data collection – Phase 2

RS10A Hydrogeological – Drill hole monitoring and data collection – Phase 3

RS14 Wetlands Delineation

RS18 Mine Design and Schedule for Backfill Alternative

RS22 Mine Waste Water Management

RS25 Mine Diking/Ditching Effectiveness Study

RS49 Stockpile Design

RS52 Closure Plan

RS73 Cumulative Streamflow Impacts. This was separated into two documents:

- RS73A Streamflow and Lake Level Changes: Model Calibration Report
- RS73B Streamflow and Lake Level Changes: Model Results

RS78 Report on Mine Block Model – Ore and Waste

1.0 Introduction

The purpose of this report is to describe the technical approach, rationale, and scope for the groundwater flow modeling that was conducted to support the Mine Waste Water Management Plan for the PolyMet NorthMet Mine Site (RS22). This report describes the objectives of the modeling, the site conceptual model, the methodologies that were used, and the modeling results. The following description of the technical approach for this modeling was based on the current understanding of the Mine Site conditions and the proposed mine plan. The modeling results presented here are based on the Mine Site conceptual model and the proposed mine plan. These results may not be applicable if there are significant changes to the conceptual model or the mine plan.

1.1 Objectives

The primary objectives of this study are to predict the amount of groundwater inflow that can be expected into the PolyMet mine pits during operations and pit filling and to determine the groundwater flow conditions following pit closure. To meet these objectives, a series of numerical groundwater flow models of the Mine Site were developed. These models were designed to simulate current conditions and conditions during mining and closure.

1.2 Background

The final mine plan, which is presented in RS18, defines the proposed pit designs (Figure 1-1). In these designs, the pits are located primarily in the Duluth Complex, with a portion of the East Pit intersecting the Virginia Formation. Extensive diking and trenching is proposed around the pits to prevent water in the surficial sediments from flowing into the pits, as addressed in the Mine Diking/Ditching Effectiveness Study (RS25).

Three hydrogeologic investigations have been conducted at the Mine Site. The Phase I investigation (RS02) characterized the hydrogeologic properties of the surficial sediment and the Duluth Complex. The Phase II investigation (RS10) characterized the hydrogeologic conditions of the Virginia Formation. The Phase III investigation (RS10A) characterized the connection of the Virginia Formation and the overlaying wetlands. Results from these studies were incorporated into a groundwater model of the Mine Site.

1.3 Report Organization

This report is organized into five sections, including this introduction. Section 2 presents the conceptual model of the Mine Site. Section 3 discusses the modeling approach. Model results and sensitivity analysis are presented in Section 4. A report summary and conclusions are presented in Section 5. Appended to this report is a technical memorandum discussing NorthMet bedrock groundwater elevation measurements.

2.0 Conceptual Model

A *hydrogeologic conceptual model* is a schematic description of how water enters, flows, and leaves the groundwater system. Its purpose is to define the major sources and sinks of water, the division or lumping of hydrostratigraphic units into aquifers and aquitards, the direction of groundwater flow, the interflow of groundwater between aquifers, and the interflow of water between surface waters and groundwater. The hydrogeologic conceptual model is both scale-dependent (i.e. local conditions may not be identical to regional conditions) and dependent upon the questions being asked. It is important when developing a conceptual model to strive for parsimony: the model should be kept as simple as possible while still adequately representing the system for the purposes of analyzing the problem at hand.

2.1 Geologic Units

2.1.1 Bedrock

The proposed mine pits will be located primarily within the Duluth Complex, with a portion of the East Pit intersecting the Virginia Formation. Underlying the Virginia Formation is the Biwabik Iron-Formation (BIF). The site bedrock geology is shown on Figure 2-1. Cross sections through the proposed mine pits that show the relationship between the various units are presented on Figures 2-2 and 2-3. The BIF is generally considered to be the most permeable unit, locally acting as a water source for residential and community wells, with the Virginia Formation and Duluth Complex being less permeable (Siegel and Ericson, 1980).

Aquifer tests were conducted at the Mine Site to determine aquifer properties of the Duluth Complex and the Virginia Formation. Four pumping tests were conducted in monitoring wells constructed within the Virginia Formation. The hydraulic conductivity values measured in these wells ranged from 0.0024 ft/day to 1.0 ft/day (RS10). The geometric mean of the values is 0.17 ft/day. Aquifer tests were conducted within exploratory drill holes completed within the Duluth Complex. Hydraulic conductivity values measured in these boreholes ranged from 0.00026 ft/day to 0.041 ft/day, with a geometric mean of 0.0024 ft/day (RS02). As a comparison, the average hydraulic conductivity determined from specific capacity tests is 1 ft/day for the Biwabik Iron-Formation and 0.03 ft/day for the Giants Range batholith (Siegel and Ericson, 1980).

2.1.2 Surficial Deposits

Geomorphically, the Mine Site is part of the Superior Upland Province and is characterized by bedrock hills and ridges which are interspersed with peat bogs and wetlands (Olcott and Siegel, 1978). At the Mine Site, the bedrock surface appears to be hummocky. Much of the Mine Site is covered by peat/wetland deposits, with the remaining area covered by rolling to undulating topography formed from Wisconsin age Rainey Lobe drift. Rainey Lobe drift is generally a bouldery till with high clay content. In the region, it appears that only the Embarrass River basin northwest of the Mine Site and the Dunka River basins northeast of the Mine Site have significant quantities of outwash (sand and gravel), with thicknesses greater than 100 feet (Olcott and Siegel, 1978). Elsewhere in the region, including the Mine Site, the surficial deposits form a thin cover over the bedrock.

The bouldery drift of the Rainy Lobe that covers the Mine Site has an estimated hydraulic conductivity range of 0.1 to 30 ft/day (Siegel and Ericson, 1980). Based on test trenches and drill core from the site, the surficial deposits at the Mine Site consist primarily of silty sand, that is interbedded with clay and silt. Lab permeameter tests on the silty sand found the hydraulic conductivity values to be 0.00043 to 0.0081 ft/day, while field testing of the various unconsolidated deposits found a range in hydraulic conductivity values of 0.012 ft/day to 31 ft/day (RS02). The ability of this unit to transmit water is highly dependent on the thickness of the sediments (Adams et al., 2004; Siegel and Ericson, 1980). At the Mine Site, the thickness of the deposits average approximately 12 feet. They are generally less than 25 feet thick, with local depths over 50 feet.

2.2 Sources and Sinks for Water

Sources of water to the saturated flow system include:

- Infiltration of precipitation;
- Groundwater seepage from wetlands and losing segments of streams;
- Seepage from nearby mine-pit lakes.

Sinks that remove water from the saturated system include:

- Discharge to streams, rivers and wetlands;
- Discharge to local mine pits that are currently being dewatered or are in the process of filling.

Evaporation from soil and free-water surfaces is assumed to be accounted for in the recharge component (i.e., recharge from precipitation includes losses from evaporation).

2.3 Local Flow System

Saturated conditions exist within the unconsolidated deposits at the Mine Site. Groundwater divides in this area generally coincide with surface-water divides. However, groundwater flow is interrupted by bedrock outcrops, which cause deviations in the groundwater flow field (Siegel and Ericson, 1980). Regionally, groundwater within the surficial deposits flows primarily to the south, from the Embarrass Mountains to the Partridge River. Figure 2-4 shows water levels measured in the wetland piezometers installed at the Mine Site. At the Mine Site, groundwater flow is generally towards the Partridge River, a major discharge point for the area. Because of the shallow nature of the aquifer, flow paths are generally thought to be short, with the recharge areas being very near the discharge areas.

Groundwater flow within the bedrock is primarily through fractures and other secondary porosity features, as the rocks have low primary hydraulic conductivity. Near the surface, water in the bedrock is thought to be hydraulically connected with the overlying surficial aquifers, resulting in similar flow directions. Recharge to the bedrock aquifers is by infiltration of precipitation in outcrop areas and leakage from the overlying surficial aquifers (Siegel and Ericson, 1980). Accounting to Siegel and Ericson, the interaction between the surficial deposits and the bedrock aquifers is assumed to be insignificant due to the low permeability of the bedrock. Groundwater contours within the bedrock units are shown on Figure 2-5. These contours are based on water levels collected from bedrock monitoring wells and exploratory boreholes during December 2006 (see Attachment A). In general, groundwater in the bedrock flows from northwest to southeast.

2.4 Hydrologic Model Selection

Groundwater flow within fractured bedrock, such as at the Mine Site, is more challenging to simulate and predict than in unconsolidated deposits. The available fracture-based modeling codes require detailed characterization of the geometry and hydraulic properties of individual fractures. At a large scale (such as the scale of this study) the fractures can reliably be assumed to be sufficiently interconnected that the fractured rock medium behaves similar to a porous medium. By assuming that the aquifer acts as an equivalent porous medium at the scale of the problem, it is possible to use conventional porous media modeling codes such as MODFLOW (McDonald and Harbaugh, 1988; Harbaugh et al., 2000) to predict the general direction and magnitude of groundwater flow. In this manner, groundwater inflow into the mine pits was predicted at the various stages in pit development.

MODFLOW simulates three-dimensional, steady-state and transient groundwater flow (saturated) using finite-difference approximations of the differential equation of groundwater flow:

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right)-W=S_s\frac{\partial h}{\partial t}$$

where:

K_{xx}, K_{yy}, and K_{zz}: three principal directions of the hydraulic conductivity tensor

W: sources and sinks

S_s: specific storage

h: hydraulic head

t: time

For steady-state simulations, the partial derivative of head with respect to time is zero and the right side of Laplace's equation, above, equals zero.

MODFLOW was developed by the U.S. Geological Survey and is in the public domain. It is widely used and accepted. The version used in the study is MODFLOW 2000. The MODFLOW model was developed using the GUI Groundwater Vistas (Version 5 Build 7) (Environmental Simulations, Inc., 2004).

3.0 Modeling Approach

An approach called Telescopic Mesh Refinement (TMR) (Ward et al., 1987) was used for the Mine Site models. The TMR approach uses a local-scale model that is embedded in a regional model. The regional model is used to define the boundary conditions for the local-scale model. This approach is useful for sites were physical or hydraulic boundaries of the aquifer lie outside of the area of interest. At the Mine Site, it was not possible to determine *a priori* what the aquifer boundaries are for the bedrock units. The TMR approach was used to obviate uncertainty in the location of boundaries.

3.1 Regional Model

A single-layer Regional Model of the area surrounding the Mine Site was constructed. This model provided the boundary conditions for the smaller, Local-Scale Model that was used to make the predictions of groundwater inflow rates into the pits.

3.1.1 Model Grid and Layers

A single flat-lying model layer, covering approximately 1000 square miles was used to simulate groundwater flow within the various bedrock units (Figure 3-1). The bottom elevation of the model was set below the maximum depth of the proposed pits at an elevation of 640 feet above mean sea level (MSL). The model grid was rotated 45 degrees in order to better align with the axis of the mine. A uniform grid with a spacing of 500 meters (1,640 feet) was used.

3.1.2 Boundary Conditions

Internal boundary conditions were used to represent surface-water features. Major rivers and lakes were simulated as either river cells or constant head cells. River and lake stage information was obtained from 7.5 minute U.S. Geological Survey quadrangle maps. Boundary conditions are shown on Figure 3-2.

The upper model boundary was simulated as a specified-flux boundary that represents recharge to the bedrock aquifers. A single recharge zone was used. The value of recharge was allowed to vary during model calibration within expected upper and lower ranges.

Perimeter model boundaries were set as no-flow boundaries in the regional model. The model perimeter was set sufficiently far from the Mine Site so that the no-flow boundaries would not affect groundwater flow predictions at the Mine Site.

3.1.3 Hydraulic Conductivity Distribution

Hydraulic conductivity distribution was based on the bedrock geology of the area (Jirsa and Chandler, 2005). Four zones were used, with a single zone representing each of the four major bedrock formations: the Biwabik Iron-Formation, Giants Ridge Formation, the Duluth Complex, and the Virginia Formation. Hydraulic conductivity values for the Biwabik Iron-Formation and the Giants Range batholith were set using literature values (Siegel and Erickson, 1980). The hydraulic conductivity of the Duluth Complex and the Virginia Formation was set as the geometric mean of values calculated as part of the Phase I and Phase II Hydrogeologic Investigations, respectively. Hydraulic conductivity values used in the Regional Model are shown in Table 3-1.

Table 3-1 Hydraulic Conductivity Values used in the Regional Model

	Hydraulic Conductivity (ft/day)
Duluth Complex	0.0014
Virginia Formation	0.33
BIF	0.72
Giants Range batholith	0.029

3.1.4 Calibration

The Regional Model was calibrated to water levels measured within the wetland areas at the Mine Site. Approximately 25 water-level targets were used, as shown on Figure 2-4. During model calibration, recharge was adjusted until there was an acceptable match between measured and simulated heads. The model was calibrated using the automated calibration capabilities of MODFLOW-2000 (Hill et al., 2000). The results of the model calibration are shown on Figure 3-3. The optimized recharge value for the model is 0.001 inches/year (7.3 x 10^{-8} m/day). This low recharge rate is consistent with information from regional studies which indicate that there is likely little interaction between the surficial deposits and the bedrock aquifers due to the low permeability of the bedrock (Siegel and Ericson, 1980).

3.2 Local-Scale Model

3.2.1 Model Grid and Layers

A grid covering an area of approximately 100 square miles was extracted from the Regional Model and used for the Local-Scale Model (Figure 3-1). The model grid was further discritized at the Mine Site, with the final grid coarser at the boundaries and outside of the area of interest (cells of approximately 100-200 meters on a side) and more refined at the Mine Site (cell size of 10 to 30 meters) (Figure 3-4). The Local-Scale Model was vertically discritized into eight layers; seven layers simulating the various bedrock units and one layer simulating the surficial deposits. Vertical discritization was needed to accurately simulate the footwall and headwall geology of the pit at various stages of pit development.

The bottom of Layer 1 was set equal to the bedrock-surface elevation as defined in RS49. The bottom elevations were modified slightly in some locations to prevent portions of the layer from going dry during model simulations. Bottom elevations for Layers 2-7 were set to correspond to the elevations of major benches in the mine pits and pit bottom elevations at various stages of development. The bottom elevation for Layer 8 was set at -65 feet MSL, which corresponds roughly to the estimated bottom elevation of the BIF at the Mine Site. Model layer bottom elevations are shown in Table 3-2.

Table 3-2 Model Layer Bottom Elevations

	Bottom Elevation (ft MSL)
Layer 1	1400 – 1585
Layer 2	1350
Layer 3	1270
Layer 4	1050
Layer 5	890
Layer 6	700
Layer 7	330
Layer 8	-65

3.2.2 Boundary Conditions

The lateral model boundaries were extracted from the regional model as constant head cells, with head values corresponding to the regional model's simulated values at these locations. Internal boundaries from the Regional Model were further discritized near the Mine Site due to the finer grid cells in this area. Additional boundaries, such as constant head cells simulating the water levels in the Peter Mitchell Pits, were added during the calibration process. Figure 3-4 shows the final boundary conditions in Layer 1.

Drain cells were used to simulate the mine pits during periods when the pits are being dewatered. Drain cells are similar to river cells, but only interact with the aquifer if the simulated head exceeds the specified drain elevation. Drain cell elevations were set at the elevation of the pit wall or floor (depending on location). Drain cell conductance was set several orders of magnitude higher that the hydraulic conductivity of the aquifers, while still maintaining a stable solution with low mass-balance error. Pit extent and elevations were based on CAD drawings of the pits presented in the RS18 at Years 1, 5, 10, 15, and 20.

During Years 12-20, the East Pit will no longer be dewatered and will be filled with waste rock. The water level in this pit will rise as a result of the cessation of pit dewatering. For the Year 12, Year 15 and Year 20 model realizations, the East Pit was simulated using river cells. River cells were used rather than drain cells to allow for the option of the pits to loose water to groundwater if the head in the pit is higher than in the surrounding aquifer. The river cell heads were set equal to the level of the water in the pit, as determined as part of RS22. The conductance of the river cells was set equal to the conductance of the drain cells that were used to simulate the pits.

3.2.3 Hydraulic Conductivity Distribution

Five hydraulic conductivity zones were used to simulate the bedrock units in the local-scale model: one zone for the Duluth Complex, two zones for the Virginia Formation, one zone for the BIF and one zone for the Giants Range batholith. Specific capacity tests conducted as part of the Phase III Hydrogeologic Investigation (RS10A) show that the upper portion of the Virginia Formation is approximately twice as permeable as the lower portion. As such, one hydraulic conductivity zone was used to represent the upper portion of the formation (Layers 2-4) and one zone was used to represent the lower portion of the formation (Layers 5-8). For the various layers, the boundary between the zones representing the Virginia Formation and the Duluth Complex and the boundary between the zones representing the Virginia Formation and the BIF was based on the location of these contacts at the elevation of the center of each layer. A three-dimensional picture of these contacts was developed by PolyMet (RS78) and was used in this study. Two hydraulic conductivity zones were used to simulate the surficial deposits in Layer 1: one zone to simulate wetland deposits and one zone to simulate glacial deposits. Boundaries of the wetland deposits were based on the wetland delineation presented in RS14. An additional low hydraulic conductivity zone was used to simulate the cutoff dikes that will be installed around portions of the stockpiles as described in RS25 and discussed in Section 4.1.1.

Hydraulic conductivity values for the zones representing the unconsolidated deposits were allowed to vary during model calibration. For these two zones, hydraulic conductivity was assumed to be laterally isotropic and vertically anisotropic. Values for the remaining zones were based on hydraulic conductivity information presented in Section 2.1.1. Hydraulic conductivity of these zones was assumed to be isotropic. Table 3-3 shows the final hydraulic conductivity values used in the Local-Scale model. A sensitivity analysis (see Section 4.4) was performed to asses the affects that the horizontal hydraulic conductivity values of the Virginia Formation and Duluth Complex and the vertical hydraulic conductivity of the surficial deposits have on simulated groundwater fluxes.

Table 3-3 Hydraulic Conductivity Values used in Local-Scale Model

	Kx=Ky (ft/day)	Kz (ft/day)
Wetland Deposits	9.3	0.0000033
Glacial Drift	2.6	0.0000033
Duluth Complex	0.0024	0.0024
Virginia Formation – Upper Portion	0.34	0.34
Virginia Formation – Lower Portion	0.085	0.085
BIF	0.98	0.98
Giants Range batholith	0.029	0.029
Cuttoff dike (predictive models only)	0.0028	0.0028

3.2.4 Recharge Distribution

The same two zones that were used to represent the hydraulic conductivity of the surficial deposits were used to represent recharge in the Local-Scale Model. Recharge was applied to the upper-most active layer. Recharge values were allowed to vary during model calibration. The final recharge values used in the Local-Scale Model are as follows:

- Recharge to wetland deposits = 0.3 inches per year
- Recharge to the glacial deposits = 1.5 inches per year

These recharge rates are consistent with the groundwater recharge rate that was predicted by the XP-SWMM model of the mine site area. The XP-SWMM model, which was calibrated to stream flow data in the Partridge River (see RS73A), has an average recharge rate of 0.84 inches per year.

3.2.5 Storage Parameters

Two storage zones were used in the groundwater model: one zone for the unconsolidated deposits in Layer 1 and one zone for the bedrock units in Layers 2-8. Storage values are used in transient simulations. The storage parameters used in the model are shown in Table 3-4.

Table 3-4 Storage parameters used in the Local-Scale model

	Specific Yield	Specific Storage
Unconsolidated Deposits	0.25	1×10^{-5}
Bedrock Units	0.05	1×10^{-5}

Specific storage values are consistent with calculated values for the Virginia Formation, as reported in RS10, as well as with literature values for fractured rock (7×10^{-5} – 3×10^{-6})(Anderson and Woessner, 1992). The specific yield of the unconsolidated deposits was set based on an average literature value for sand and silt (Anderson and Woessner, 1992). The sensitivity of the model results to the storage parameter values was examined as part of the sensitivity analysis presented in Section 4.4.

3.2.6 Model Calibration

The Local-Scale Model was recalibrated using a combination of traditional trial-and-error methods and automated calibration methods. Automated calibration was conducted using MODFLOW-2000 (Hill et al., 2000). The baseline conditions model, which was used for calibration, was a steady-state model. During model calibration, the only parameters that were allowed to vary were hydraulic conductivity of the surficial deposits, recharge, and conductance of the river cells simulating the Partridge River.

The model was calibrated to the same water-level data in the unconsolidated deposits that were used to calibrate the Regional Model, plus additional water-level data measured in bedrock wells and exploratory boreholes during December 2006 (Attachment 1). Head calibration targets are shown on Figures 2-4 and 2-5. All bedrock head targets were located in Model Layer 2. In addition to head targets, the model was also calibrated to a prediction of baseflow in the north branch of the Partridge River just upstream of the confluence with the south branch of the Partridge River, monitoring station SW004 (Figure 3-5). The XP-SWMM model presented in RS73 predicted that baseflow at this location under current conditions is approximately 1.43 cfs. For this purpose, baseflow is defined as the groundwater contribution to streamflow.

Calibration results are shown on Figures 3-5 and 3-6. The baseline conditions model matches the general flow directions in both the unconsolidated deposits and the bedrock. In general, model simulated heads were higher than measured heads in Layer 1. In Layer 2, the model simulated gradient was slightly flatter than observed in the field, resulting in high heads simulated lower than measured and low heads simulated higher than measured. The predicted baseflow in the Partridge

River was 1.49 cfs, compared to the target baseflow of 1.43 cfs. Overall, the calibration was determined to be acceptable given the modeling objectives. The residual mean and absolute residual mean of the head targets were 0.02 meters and 1.57 meters respectively. The range of observed heads is 17 meters.

3.3 Assumptions and Limitations of the Model

The groundwater flow models that were constructed and calibrated for this evaluation are a necessary simplification of groundwater flow in the vicinity of the Mine Site. Several limitations to the model need to be acknowledged. These limitations are the result of assumptions and simplifications that are inherent to any groundwater modeling. The assumptions and limitations include:

- The use of a conventional porous media modeling code can accurately simulate flow within the bedrock units at the Mine Site, which is assumed to be primarily through interconnected fractures, at the scale of this study. It is assumed that the fractures are sufficiently interconnected such that the fractured rock medium behaves similar to a porous medium.

- The bedrock units at the Mine Site are assumed to be homogeneous in terms of hydraulic conductivity. In reality, all geologic material has variations resulting in heterogeneity. The assumption of homogeneity is considered appropriate given the modeling objectives for this evaluation.

- The model will not simulate any off-site well pumping or pit dewatering. The Peter Mitchell Pits, located north of the Mine Site, have historically been dewatered periodically. However, future operation of these pits cannot be anticipated and was not simulated. Affects of dewatering at the Peter Mitchell Pits was not evaluated as part of this work.

- The validity of the modeling results is based on the assumption that the conceptual model is a reasonable representation of the groundwater flow system. The conceptual model, in turn, is based on the data that are collected at the Mine Site and the interpretation of those data. Errors in the data or data interpretation that affect the groundwater flow model's conceptualization may result in errors in the flow simulation.

The groundwater flow model was designed with the specific goal of predicting groundwater flow rates into the mine pits during operation and closure. If the model is to be used for other purposes, the validity of the model for that purpose must be carefully evaluated.

4.0 Model Simulations and Results

4.1 Mine Operation

4.1.1 Simulations

Five model realizations were used to simulate conditions during mine operations. All model realizations were transient simulations. Model realizations are as follows:

- The first realizations simulated Years 1-10, during which time both the East Pit and the West Pit are to be mined and dewatered. Linear interpolation was used to determine pit elevations in years for which no pit design was available. The Years 1-10 Model had ten stress periods, each 365 days long, with five time steps. Initial heads were taken from the baseline conditions model.

- The second model realization simulated Year 11, when the East Pit is at its maximum extent. This realization had one stress period 365 days long with 5 time steps. The Year 11 Model used the final results from the Years 1-10 Model as initial conditions.

- The third model realization simulated Year 12, when the East Pit is first backfilled with waste rock. This realization had one stress period 730 days long with 5 time steps. The Year 12 Model used the final results from the Year 11 Model as initial conditions.

- The fourth model realization simulated Year 15, where the East Pit is partially filled with rock and water and the West Pit is still being mined and dewatered. The Year 15 model had one stress periods, 730 days long, with 10 time steps. The final results from the Year 12 Model were used as the initial conditions for the Year 15 model.

- The final model realization simulated Year 20, where the East Pit is filled with rock and water and the West Pit is at its maximum extent. The Year 20 Model had one stress period 1825 days long with 10 time steps. The final results from the Year 15 Model were used as the initial conditions for the Year 20 Model.

In the Year 12, Year 15 and Year 20 models, the head in the East Pit was defined using pit filling information presented in RS22.

4.1.2 Results

4.1.2.1 Groundwater Inflow Rates

These model realizations were used to predict the amount of groundwater that can be expected to flow into the mine pits during operations. Table 4-1 shows the predicted groundwater inflow rates.

Table 4-1 Predicted Groundwater Flow Rates during Mine Operations

	East Pit		Central Pit		West Pit	
	GW Inflow	GW Outflow	GW Inflow	GW Outflow	GW Inflow	GW Outflow
	gpm	gpm	gpm	gpm	gpm	gpm
Year 1	180	0	--	--	20	0
Year 5	820	0	--	--	90	0
Year 10	880	0	--	--	170	0
Year 11	950	0	--	--	130	0
Year 12	860	0	--	--	150	0
Year 15	770	0	50	0	325	0
Year 20	20	130	20	10	810	0

Groundwater inflow into the East Pit increases during Years 1 through 11 as the pit expands laterally and vertically. Starting in Year 12, backfill of the pit with rock and water will begin and dewatering of this pit will cease. By Year 20, the East Pit is predicted to lose more water to the groundwater system that it receives. This is due in large part to the continued dewatering of the West Pit, which creates a cone of depression that extends beyond the East Pit. Figures 4-1 and 4-2 shows the simulated drawdown the surficial aquifers and bedrock at this time.

The Central Pit, which will eventually become part of the East Pit, will be mined from Year 12 to Year 13. Starting in Year 14, the pit will be filled with rock and water and dewatering ceases. The filling of the Central and East Pits is described in RS22. Similar to the East Pit, the Central pit is predicted to have both groundwater inflow and outflow by Year 20. Groundwater inflow into the West Pit is predicted to increase during Years 1 through 20 as the pit expands laterally and vertically.

4.1.2.2 Impacts to Partridge River

The mine operations models were also used for prediction of impacts to the baseflow within the Partridge River. Although not a primary objective of the groundwater modeling, the model realizations can be used to predict average groundwater discharge rates to the Partridge River both during mine operations and during pre-mining conditions. The groundwater models predict baseflow reductions as a result of pit dewatering at three locations, show on Figure 3-5. Results are summarized in Table 4-2.

Table 4-2 Predicted Percent Reduction in Partridge River Baseflow

	Location		
	SW002	SW003	SW004
Year 1	1%	1%	-3%
Year 5	5%	3%	-2%
Year 10	10%	8%	0%
Year 15	14%	11%	7%
Year 20	19%	14%	10%

Baseflow impacts to the Partridge River increase at locations SW002 and SW003 as mining progresses and the pit dewatering increases. At SW004, there is initially an increase in baseflow, followed by a decrease starting after Year 10. This initial increase is likely due to the redirection of groundwater flow following the installation of the cutoff dikes that will be installed around portions of the stockpiles. A detailed assessment of water quantity impacts to the Partridge River is provided in RS73B.

4.2 West Pit Filling

4.2.1 Simulations

Following the completion of mining of the West Pit in Year 20, the dewatering of the pit will cease and additional water will be discharged into the pit, as discussed in the Closure Plan (RS52). As the West Pit fills with water, the groundwater flow into the pit will decrease. Several model simulations were run in order to predict groundwater inflow rates into the West Pit at various stages of pit filling. For each model simulation, the elevation of the water in the pit was set using the River Package, as discussed in Section 3.2.2.

4.2.2 Results

Groundwater inflow rates into the West Pit during filling were predicted for various water levels in the West Pit. Simulation results are shown in Figure 4-3. As expected, groundwater inflow rates decrease as the pit fills with water.

4.3 Long Term Closure

A constructed wetland will be built within the area of the former East Pit to provide additional treatment of the stockpile drainage water. This system is described in greater detail in RS52, but is discussed here as it pertains to the closure scenario models. The wetland treatment system will be a passive system, with an inflow area along the eastern boundary and an outflow structure to the West Pit along the western boundary. The wetland will be constructed above the waste rock fill in the East Pit and will be separated from the waste rock by a barrier layer constructed of compacted glacial till overburden. The invert of the outlet structure connecting the East Pit to the West Pit will be at an elevation of 1,592 ft-MSL.

The West Pit is predicted to fill in approximately 40 years. Prior to the completion of pit filling, an outlet structure will be constructed on the southeastern side of the West Pit at an elevation of 1,581 ft-MSL near the natural overflow location. Details on pit filling and the outlet structure are provided in RS52.

4.3.1 Simulations

Two final model realizations were constructed: one to predict the maximum steady-state water level in the mine pits assuming no outlet structures, and one to predict final groundwater conditions at post-closure. The first simulation was performed to determine if outlet structures will be needed for the pits or if the water level in the pits would stabilize prior to overflowing. To simulate this, the mine pits were represented in the model as high hydraulic conductivity zones following the methodology of Anderson et al. (2002). A hydraulic conductivity of 3,280 ft/day was used.

The second simulation was performed to predict final groundwater conditions in post-closure, i.e. once the system has reached equilibrium. In this simulation, the West Pit is simulated using river cells, with the head set at the outlet elevation of 1,581 ft-MSL. The portion of the East Pit that is backfilled with waste rock was simulated as a high hydraulic conductivity zone (K = 33 ft/day). The constructed wetland above the waste rock was simulated using the river package with a head set equal to the outlet elevation of 1,592 ft-MSL. The vertical hydraulic conductivity of the wetland was assumed to be equal to the vertical hydraulic conductivity of the native till.

4.3.2 Results

The predicted stable water level in the East Pit assuming no outlet is predicted to be 1,601 ft MSL. The predicted stable water level in the West Pit is predicted to be 1,597 ft MSL assuming no outlet. The baseline conditions model, presented in Section 3.2.6, had heads in Layer 1 that were generally higher than measured in the field. As such, the predicted stable water elevations may be higher than would be expect and should be considered approximate.

Groundwater contours in the surficial deposits and bedrock at final closure (i.e. when the system has reached steady state) are shown in Figures 4-4 and 4-5. General flow directions have also been added to these figures. With the proposed outlet elevations, both pits are predicted to have groundwater flow into the pit along a portion of the pit perimeter and groundwater flows out of the pit over a portion of the pit perimeter. Predicted seepage rates are shown below in Table 4-3.

Table 4-3 Predicted groundwater inflow and outflow rates for the pits in post-closure

		Groundwater Inflow (gpm)	Groundwater Outflow (gpm)
West Pit	Surficial Aquifers	30	20
	Bedrock Aquifers	30	0
East Pit	Surficial Aquifers	20	<5
	Bedrock Aquifers	20	<5

Both pits are expected to have a net positive flux of groundwater. Only the West Pit is predicted to lose a significant quantity of water to the groundwater system, in this case the surficial aquifer to the south.

4.4 Sensitivity Analysis

An analysis was performed to assess the sensitivity of the model predictions to uncertainties in model parameters. For this analysis, the baseline conditions model and Years 1-10 model realizations were used. The following parameters were adjusted in the sensitivity analysis:

- Hydraulic conductivity of the Duluth Complex: The base model used the geometric mean of hydraulic conductivity values from the Phase I Hydrogeologic Investigation (0.0024 ft/day). For the sensitivity analysis, the minimum value (0.00026 ft/day) and the maximum values (0.041ft/day) were used.

- Hydraulic conductivity of the Virginia Formation: The base model used the geometric mean of hydraulic conductivity values from the Phase II Hydrogeologic Investigation (0.17 ft/day). For the sensitivity analysis, the minimum value (0.0024 ft/day) and the maximum values (1.0 ft/day) were used.

- Vertical hydraulic conductivity of the surficial deposits: The base model used a vertical hydraulic conductivity of 3.28×10^{-6} ft/day. For the sensitivity analysis, a vertical hydraulic conductivity ranging from 3.28×10^{-8} to 3.28×10^{-4} ft/day was used.

- Storage parameters: A range of storage parameters (specific yield and specific storage) for the bedrock and surficial deposits were used as part of the sensitivity analysis.

- Conductance of river cells: The hydraulic conductivity of the river cells simulating the Partridge River was adjusted during the calibration process. The calibrated value, 25 ft/day, was varied over four orders of magnitude as part of the sensitivity analysis.

- Lateral model boundary conditions: As discussed in Section 3.2.2, lateral model boundaries for the site model were extracted from the regional model as constant head cells. As part of the sensitivity analysis, these boundaries were converted to general head boundaries. The sensitivity of the model to the conductance of the new boundary was tested.

For each parameter tested as part of the sensitivity analysis, calibration statistics (residual mean and absolute residual mean) and predictions at Year 10 of inflow rates to the pits and baseflow reductions at SW003 were determined. Results of the sensitivity analysis are shown on Figures 4-6 to 4-13. For each parameter, the sensitivity of the model to the parameter was assessed using the ASTM standard for sensitivity analysis for groundwater flow models (ASTM D 5611-94), which defines four types of sensitivities:

- Type I – Variation of the input parameter causes insignificant changes in the calibration residuals as well as the model's predictions;

- Type II – Variations of the input parameter cause significant changes in the calibration residuals but insignificant changes in the model's predictions;

- Type III – Variations of the input parameters cause significant changes to both the calibration residuals and the model's predictions; and

- Type IV – Variation of the input parameter cause insignificant changes in the calibration residuals but significant changes in the model's predictions.

If an input parameter is used only in the predictive realizations, the sensitivity is automatically either Type I or IV. Of greatest concern are the parameters with Type IV sensitivity because the parameter value has a significant effect on the model prediction but cannot be constrained by model calibration. Table 4-4 summarizes the sensitivity types for each of the parameters tested.

Because the calibration model is a steady-state model, all of the storage parameters must be either Type I or IV. Changes in the specific yield of the bedrock do not effect the model predictions and as such it has Type I sensitivity. Conversely, changes to the specific storage of the bedrock and surficial aquifer and the specific yield of the surficial aquifer do result in significant changes in the model predictions. Specific storage is likely between 10^{-6} and 10^{-4} meters^{-1}. In this range, there is little variability in the predicted inflow rate to the pits. However, at higher specific storages, the prediction of Partridge River baseflow reduction decreases. Specific storage values in the model

Table 4-4 Summary of Sensitivity Analysis

Change in Predictions \ Change in Calibration	Insignificant	Significant
Insignificant	**Type I** • Specific yield of bedrock	**Type II** • Lateral model boundary
Significant	**Type IV** • Specific storage • Specific yield of surficial aquifer	**Type III** • Hydraulic conductivity of Duluth Complex • Hydraulic conductivity of Virginia Formation • Vertical conductivity of surficial aquifer • River cell conductance

were set based on results from the pumping tests done on site, as discussed in Section3.2.5. Changes in the specific yield of the surficial aquifer result in significant changes to the predicted Partridge River baseflow reductions. Specific yield was set based on literature values.

The horizontal hydraulic conductivity of the Duluth complex and the Virginia Formation, the vertical hydraulic conductivity of the surficial aquifer and the conductance of the river cells simulating the Partridge River all have Type III sensitivity. The groundwater flux prediction is most sensitive to the values of hydraulic conductivity of the Virginia Formation and the Duluth Complex. As discussed in Section 3.2.3, these values were set based on results of site specific pumping tests. The predicted impacts to the Partridge River are most affected by the vertical hydraulic conductivity of the surficial deposits, which was adjusted during model calibration. At high conductivity values (10^{-5} to 10^{-4} meters/day), the model calibration is moderately sensitive to this parameter, with little calibration sensitivity at lower values. Results from the Phase III Hydrogeologic Investigation (RS10A), which showed a poor connection between the bedrock and the surficial deposits, further substantiates the low value used in the model.

The model predictions were not sensitive to variations in the lateral model boundaries. This is important to note because the cone of depression associated with pit dewatering does intersect the model boundaries in the layers simulating bedrock. By changing the constant head boundary to a general head boundary, it is possible to test the effects the model boundaries have on the predictions of pit inflow rates. The model calibration was sensitive to the variability of the conductance of the general head boundary. However, over the eight orders of magnitude that were tested, there was very little variability in the predicted groundwater inflow rates to the pits. To further test the sensitivity of the model predictions to the boundary conditions, the water balances for the baseline conditions and operating conditions were examined for the bedrock aquifers. In general, groundwater flows towards the model boundaries, with the net flux of the boundaries being negative (i.e. more water leaving the model through the boundaries than entering the model through the boundaries). During pit dewatering, these boundaries continue to act as a sink for water, with the total flux to the boundary decreasing roughly 20% (less than 200 gpm) by year 20. Because of this, the boundaries are not acting as an infinite source of water for the mode. The boundaries may, however, be retarding the predicted extent of drawdown in the bedrock and as such should heads near the boundaries should be considered approximate.

5.0 Summary and Conclusions

A major component of the Mine Site water balance is the groundwater flow into the mine pits. Groundwater inflows from surficial deposits, the Duluth Complex, and the Virginia Formation were predicted using the industry standard finite difference groundwater modeling code MODFLOW. A three-dimensional model was constructed for the 100-square mile area encompassing the proposed mine pits. Data collected as part of the Phase I, Phase II, and Phase III Hydrogeologic Investigations, which provided information on the hydraulic conductivity of the Duluth Complex, the Virginia Formation and the surficial deposits, was incorporated into the model (see RS02, RS10 and RS10A). The model was calibrated to groundwater levels in both the bedrock aquifers and the surficial deposits.

Several transient model realizations simulating the pits in various stages of development (i.e. Years 1, 5, 10, 15 and 20) were constructed based on the proposed mine plan. Groundwater inflow rates to the pits were predicted in each model realization. A sensitivity analysis was performed to address uncertainties in model parameters. In addition to predicting groundwater flow rates into the pits during operations, the groundwater model was used to predict impacts to the Partridge River during operations, predict groundwater flow rates during pit filling, and predict groundwater flow at closure.

The following conclusions can be drawn from the work presented here:

- Groundwater flow into the East and West Pits will increase from 200 gpm to 1080 gpm between Year 1 and Year 11 as the pits expand laterally and vertically. Groundwater flow into the East Pit will begin to decrease starting in Year 12 as the pit is backfilled with rock and water. Groundwater flow into the West Pit will continue to increase through Year 20, reaching a maximum predicted inflow rate of 810 gpm.

- As a result of pit dewatering, baseflow in the Partridge River is predicted to be reduced between 10% and 19% at the four locations examined.

- In closure, both the East Pit and the West pit are predicted to have a net positive flux of groundwater into the pits. The East Pit is predicted to lose a small amount water (<10 gpm) to both the surficial aquifer and the bedrock aquifers; the West Pit is predicted to loss 20 gpm to the surficial aquifer.

- Predicted groundwater flow rates are most sensitive to the hydraulic conductivity values for the Duluth Complex and the Virginia Formation. Predicted impacts to Partridge River baseflow are most sensitive to the vertical hydraulic conductivity and specific yield of the surficial deposits.

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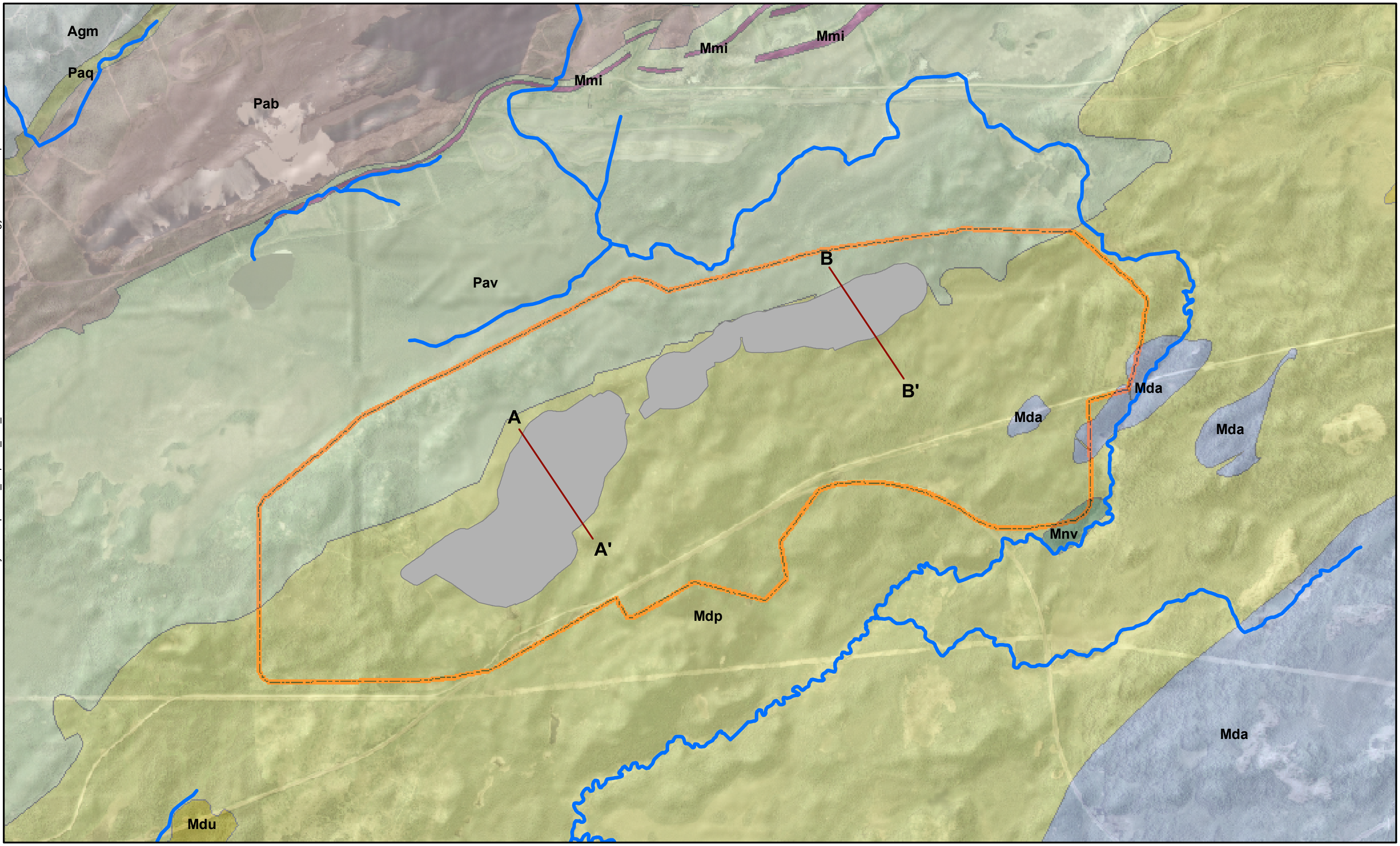

Geology Data from Jirsa, M.A., V.W. Chandler, and R.S. Lively, 2005. Bedrock Geology of the Mesabi Iron Range, Minnesota. Miscellaneous Map Series map M-163

Bedrock Geology

Mesoproterozoic - Duluth Complex

- Mda Anorthositic Series Substitute of the Duluth Complex
- Mdg Gabbro
- Mdp Partridge River Intrusion
- Mdu Ultramafic, Oxide-rich Intrusions
- Mmi Mafic Intrusions
- Mnv North Shore Volcanic Group

Paleoproterozoic

- Pab Biwabik Iron Formation
- Paq Pokegama Quartzite
- Pav Virginia Formation

Neoarchean - Giants Range batholith

- Agm Quartz Monzontie and Monzodiorite

- Mine Site
- Mine Pits
- River or Stream
- X-Section Location

0 1,0002,000 4,000

Feet

Figure 2-1

MINE SITE
BEDROCK GEOLOGY

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

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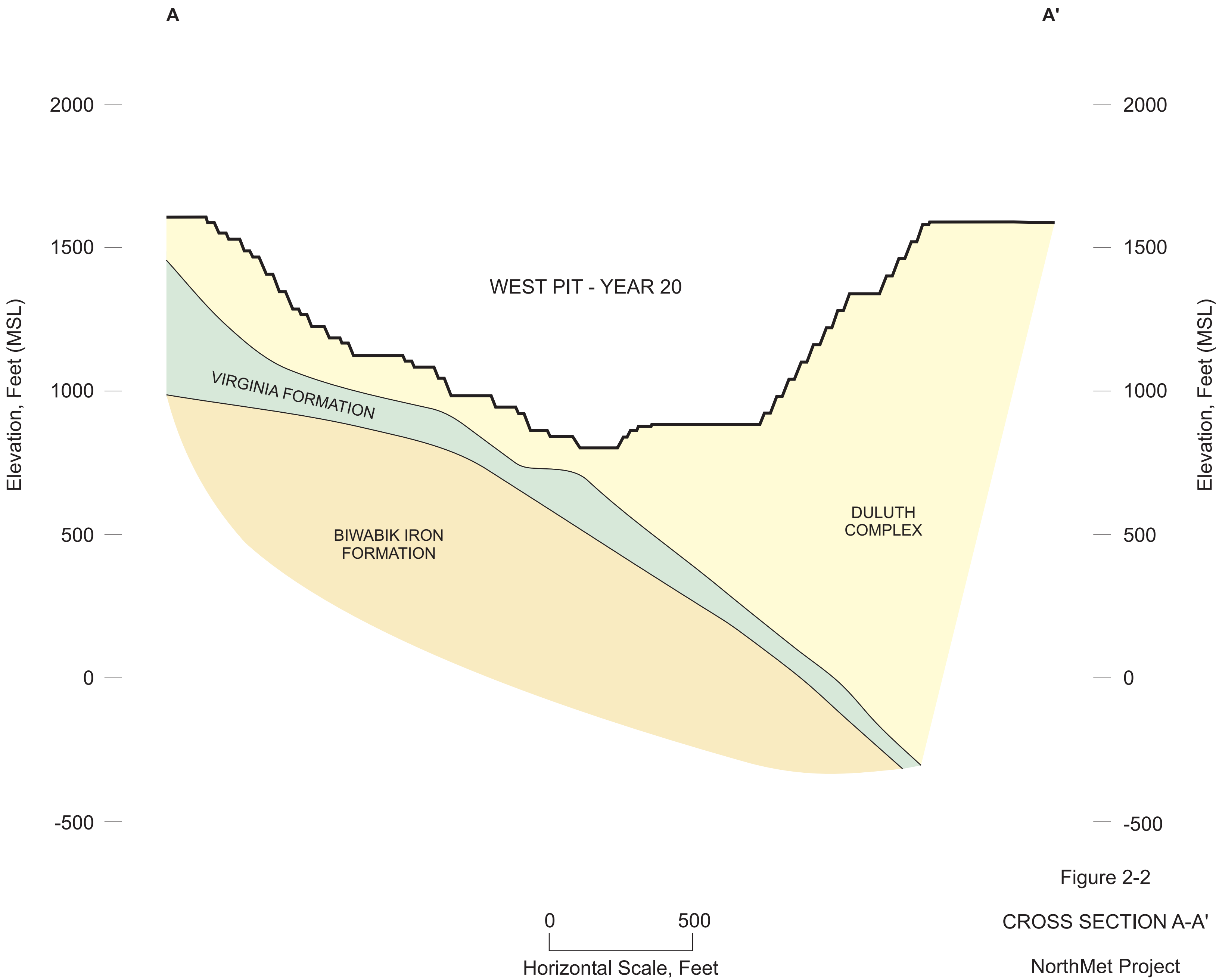

Figure 2-2

CROSS SECTION A-A'

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

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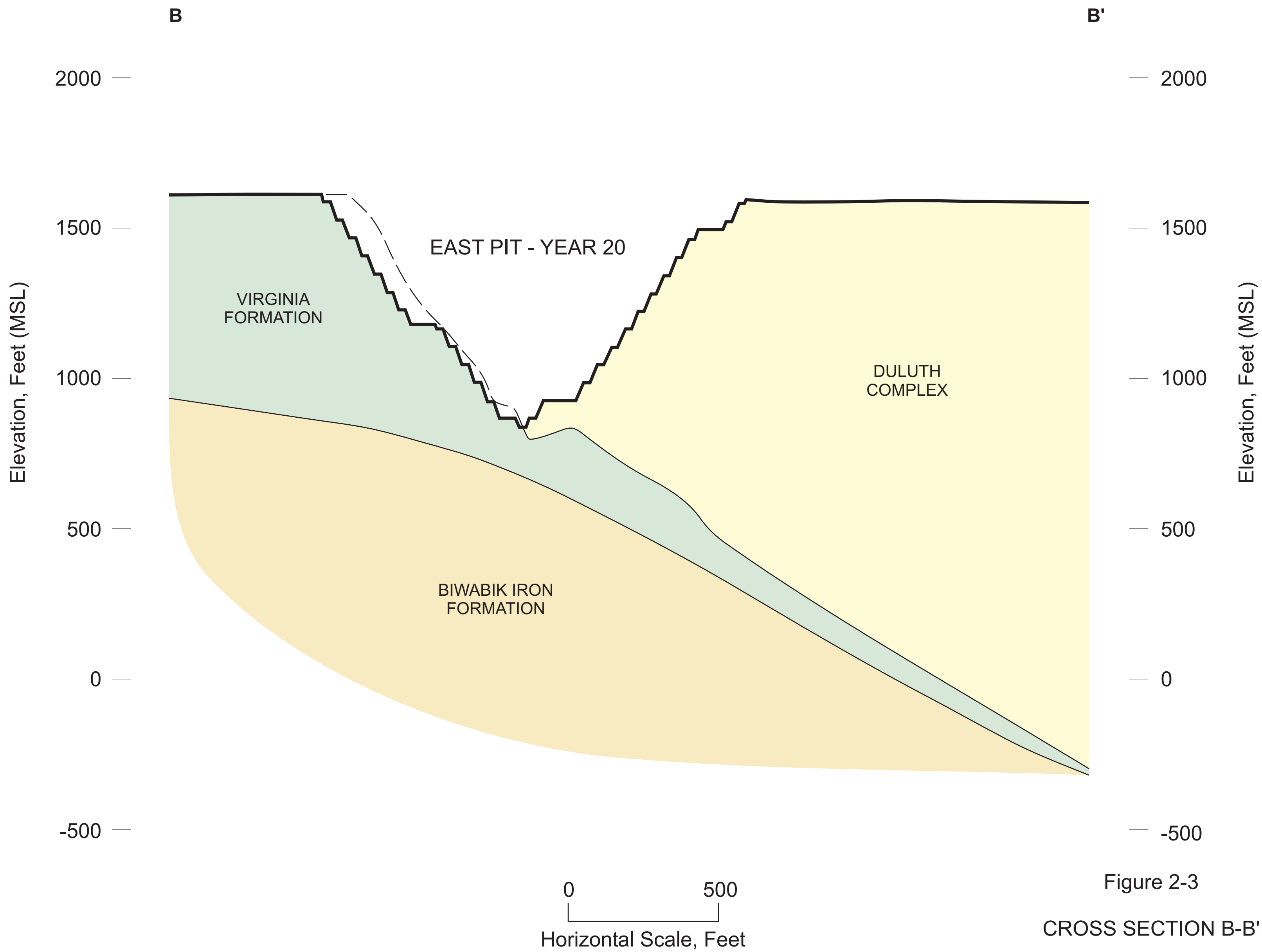
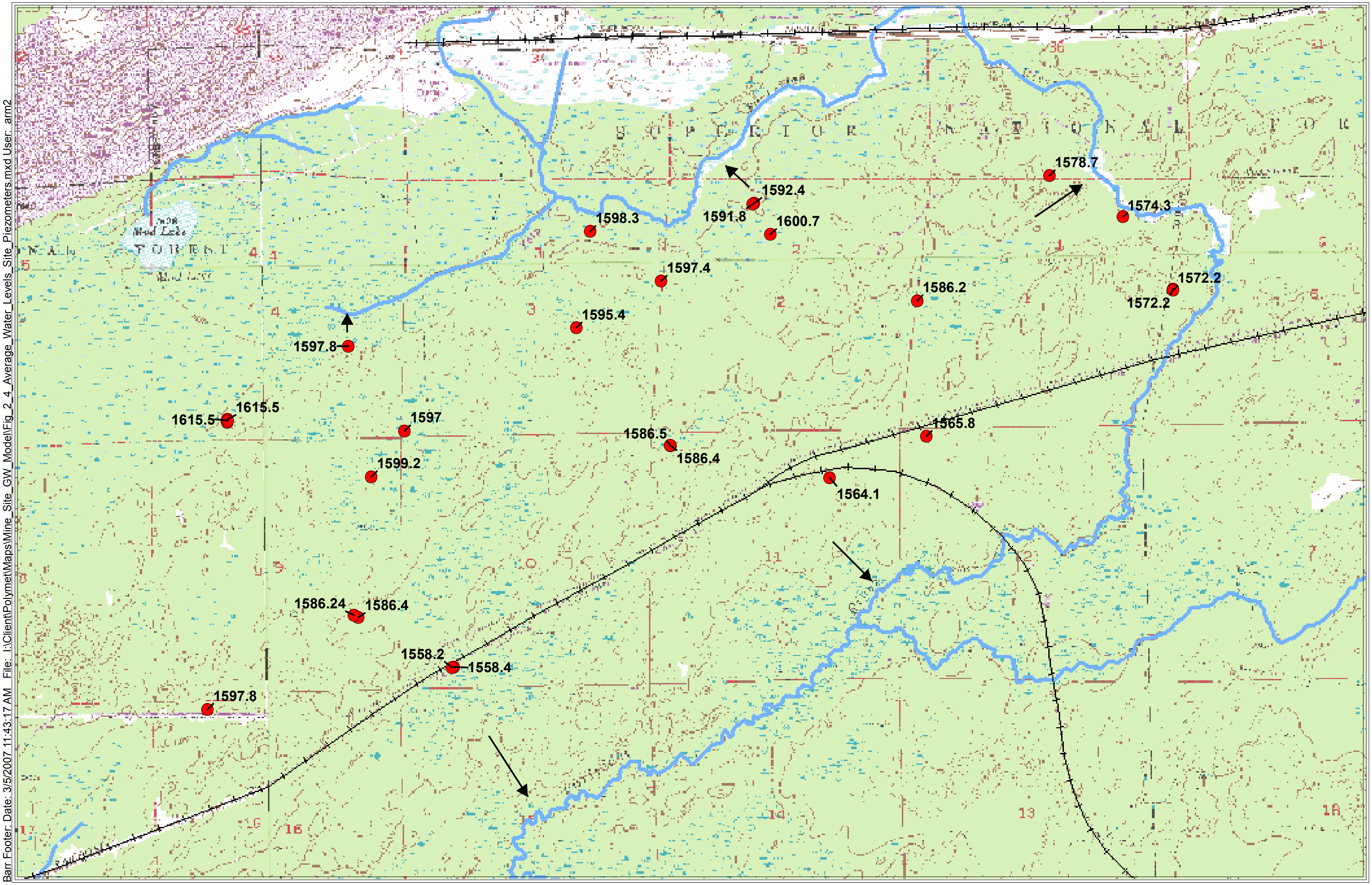


Figure 2-3

CROSS SECTION B-B'

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



● Wetland Monitoring Locations

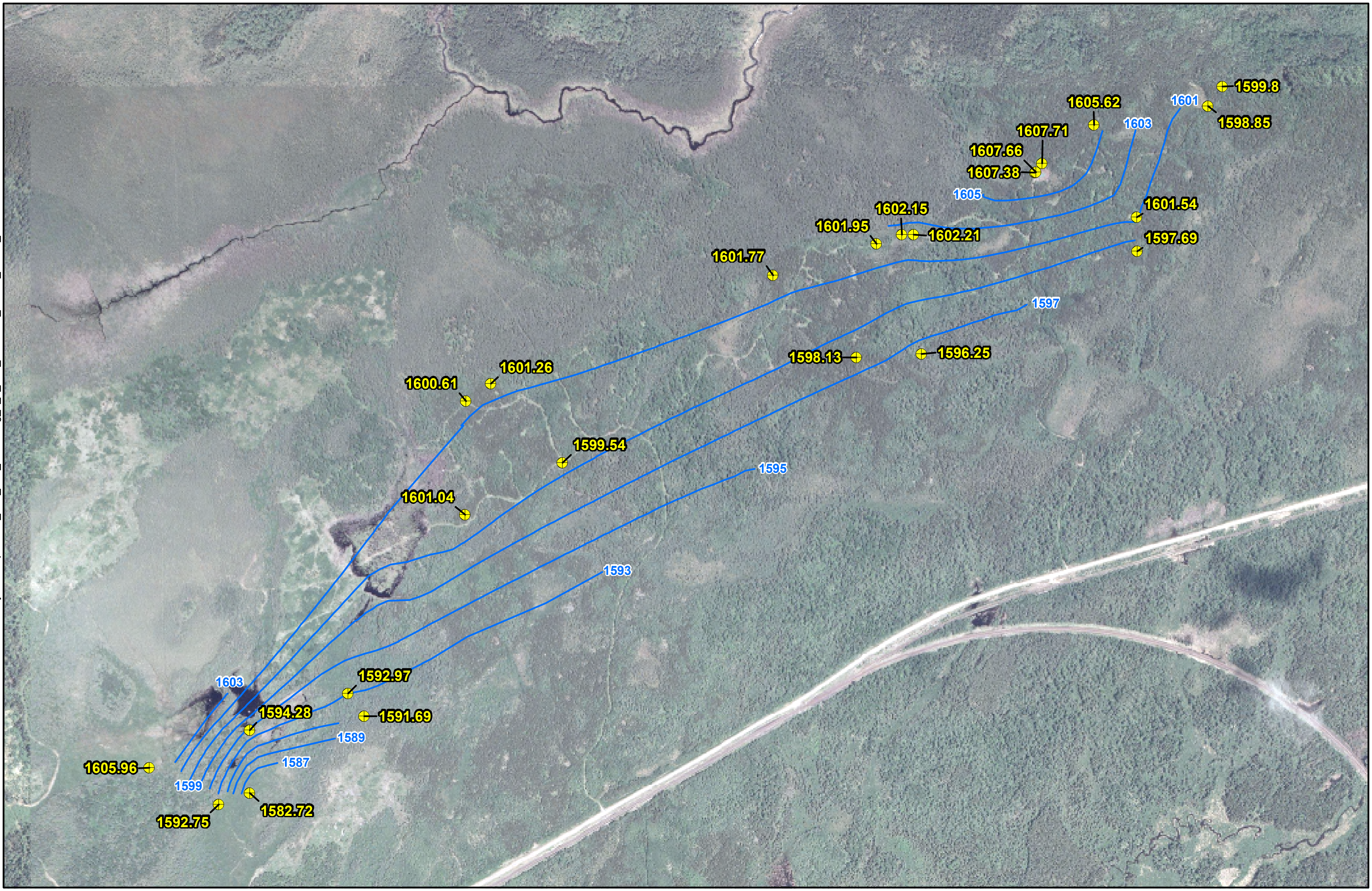
→ Flow Direction

0 2,500 5,000
Feet

Figure 2-4

AVERAGE WATER LEVELS
MEASURED IN SITE PIEZOMETERS

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Groundwater Elevation Measurement Used for Contouring-Feet MSL

Groundwater Contour-Feet MSL (contour interval = 2ft)

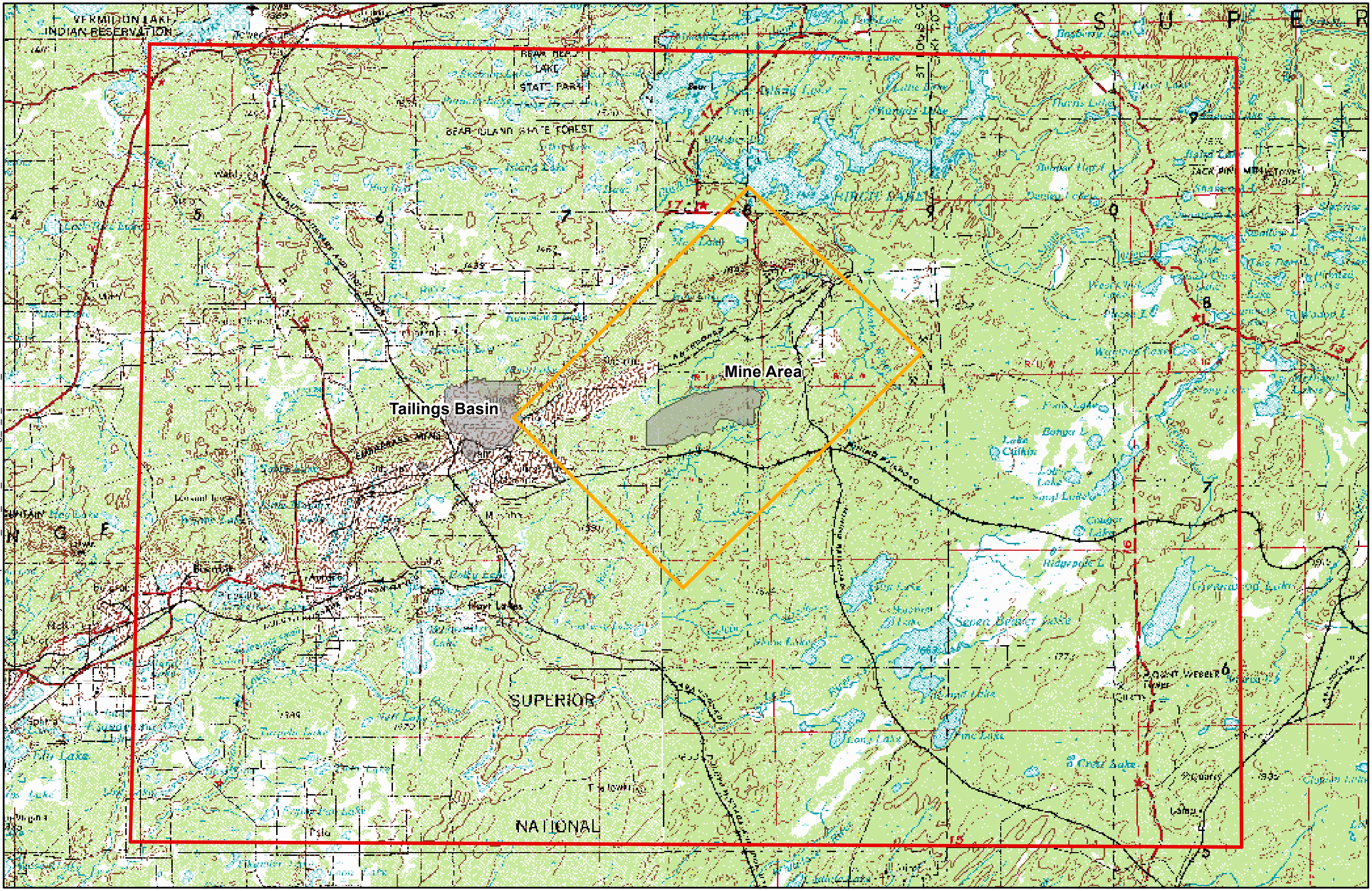
0 500 1,000 2,000 3,000

Feet

Figure 2-5

GROUNDWATER CONTOURS AND HEADS WITHIN THE BEDROCK AT THE NORTHMET MINE SITE

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Legend

- Regional Model Area
- Local-Scale Model Area
- Project Areas

0 2.5 5 10
Miles

Figure 3-1

MODEL AREAS

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

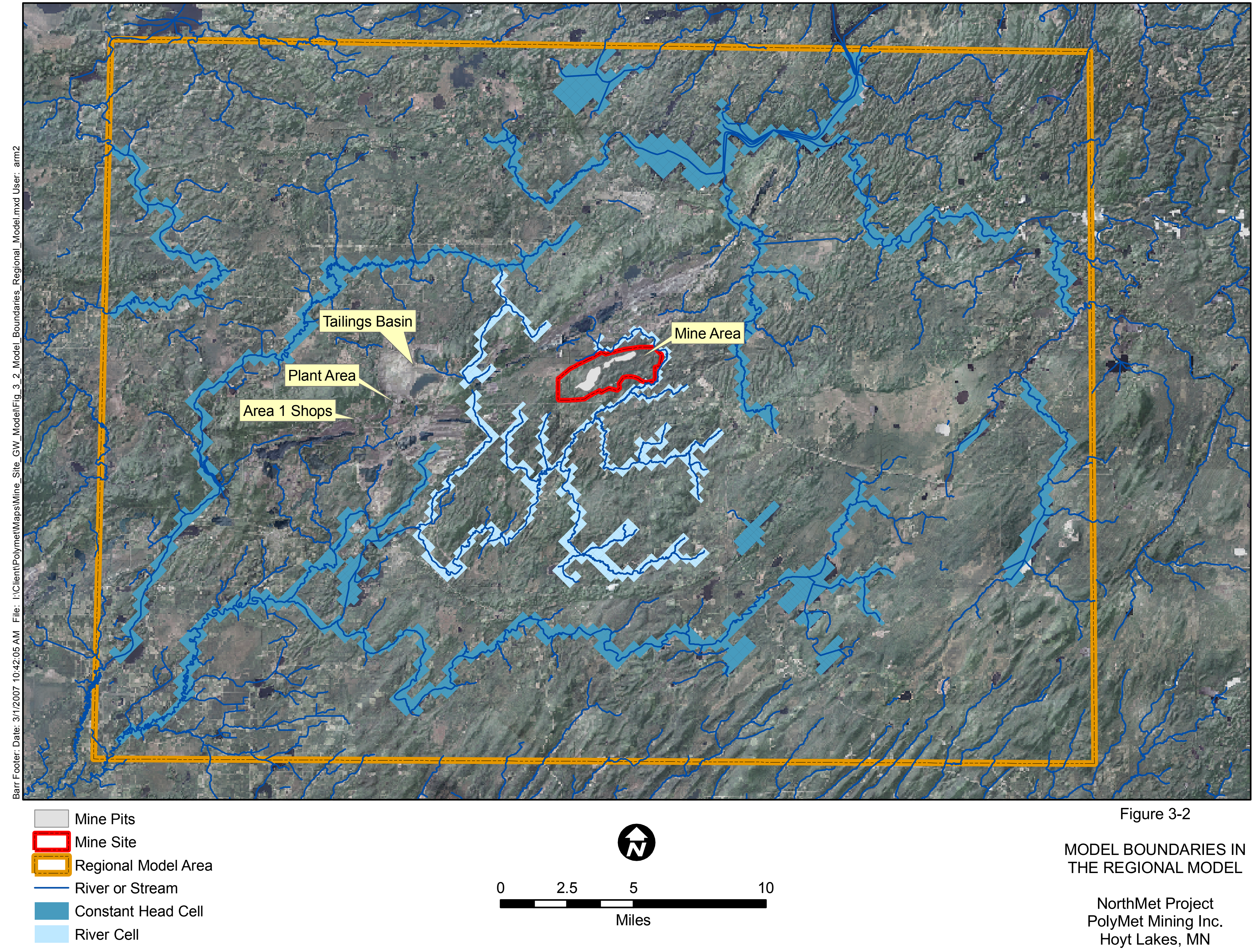
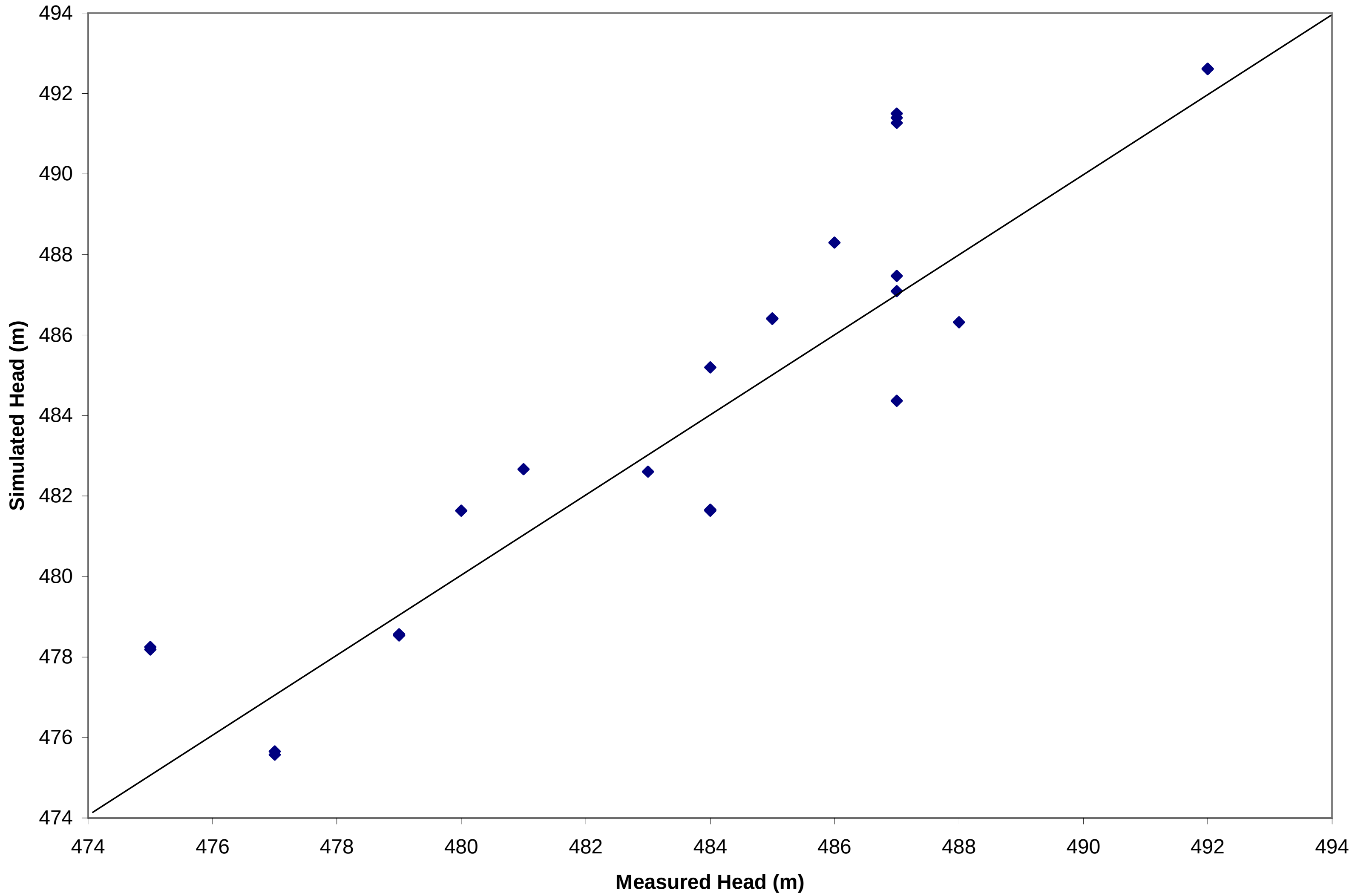


Figure 3-2

MODEL BOUNDARIES IN THE REGIONAL MODEL

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

Figure 3-3 Regional Model Calibration Results



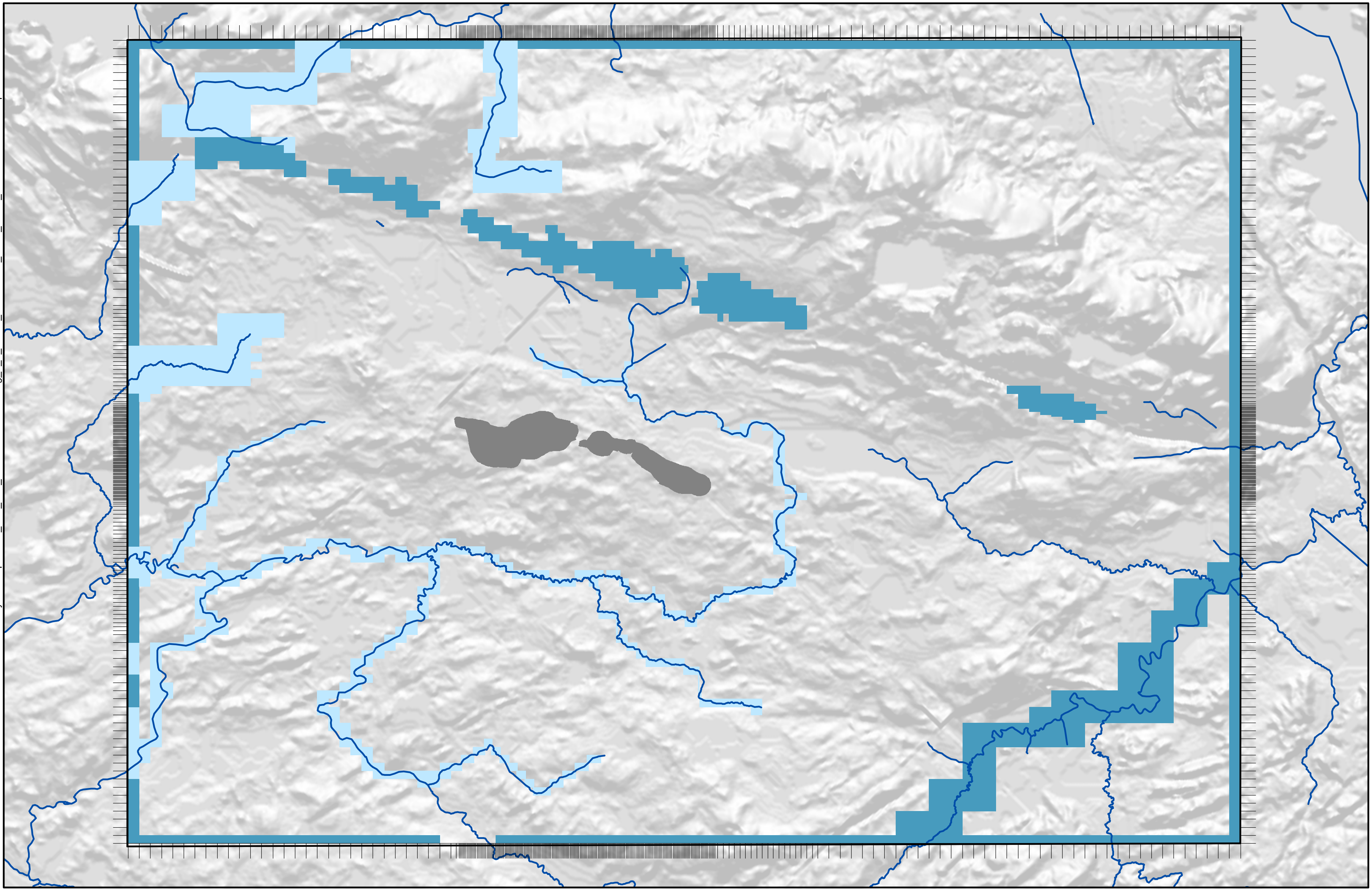


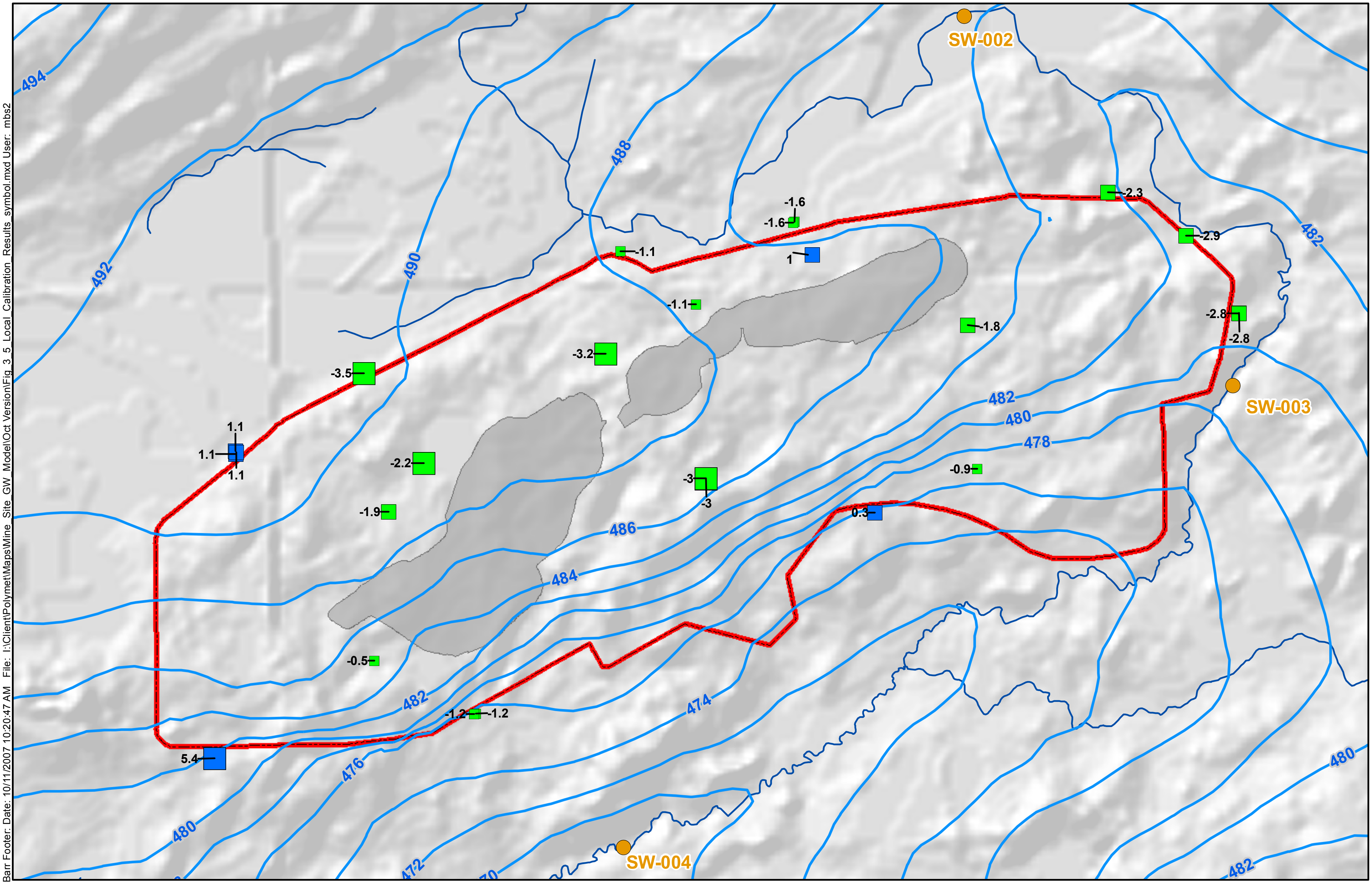
Figure 3-4

Local-Scale Model Area
Model Grid
Mine Site
River or Stream
Constant Head Cell
River Cell
Mine Pits

0 0.5 1 2 3
Miles

MODEL BOUNDARIES IN
THE LOCAL-SCALE MODEL

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Monitoring Location
River or Stream
Mine Pits
Mine Site
Simulated Water Table (meters)
Contour Interval 2 meters

Layer 1 Calibration Residuals

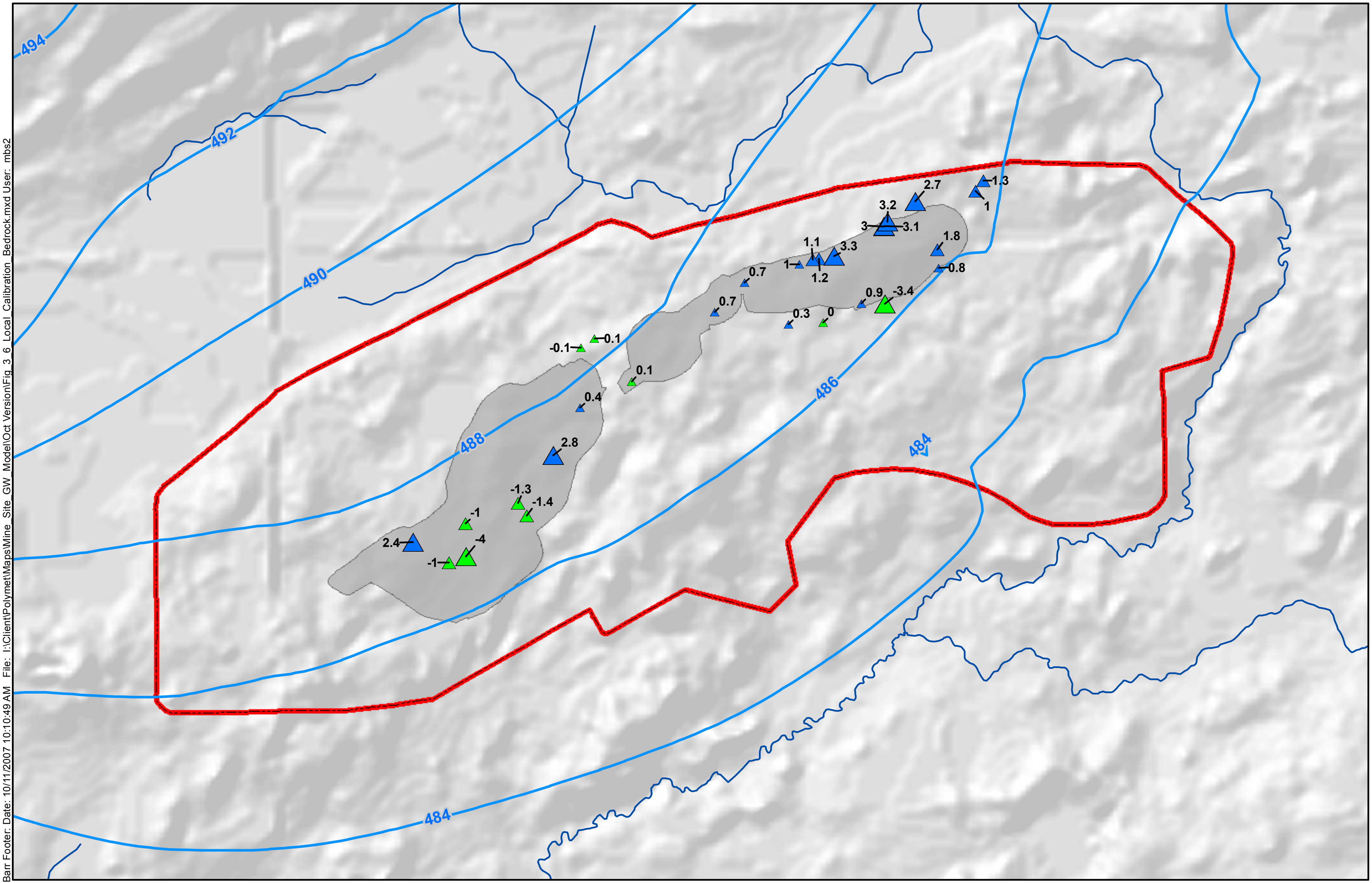
- > 2.00 meters
- 1.00 - 1.99 meters
- 0.00 - 0.99 meters
- 0.00 - -0.99 meters
- -1.00 - -1.99 meters
- < -2.00 meters

0 1,000 2,000 4,000
Feet

Figure 3-5

LOCAL-SCALE MODEL CALIBRATION
RESULTS - SURFICIAL AQUIFER

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



- River or Stream
- Mine Pits
- Mine Site
- Simulated Piezometric Surface (meters) Contour Interval 2 meters

Layer 2 Calibration Residuals

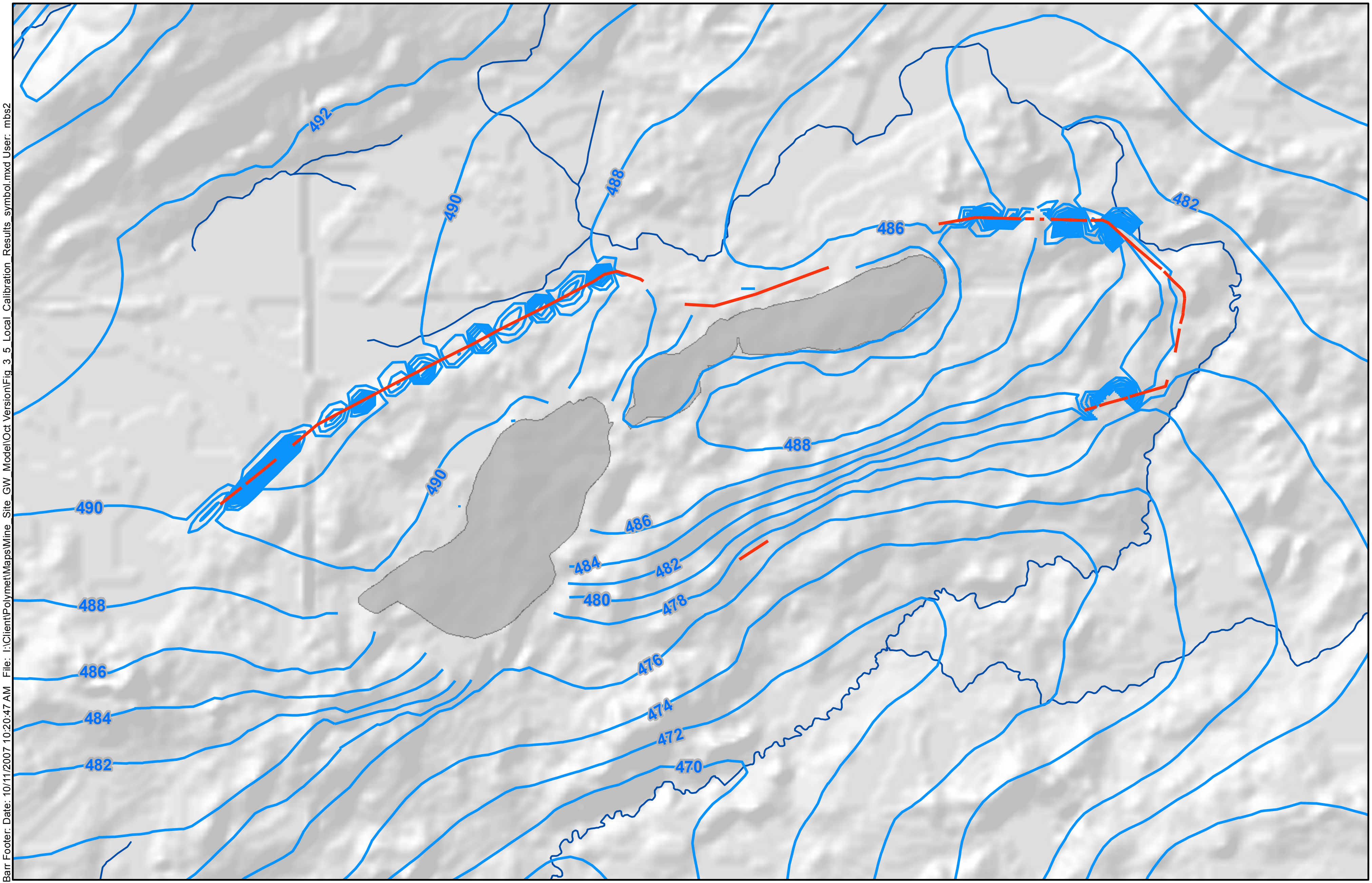
- > 2.0 meters
- 1.00 - 1.99 meters
- 0.00 - 0.99 meters
- 0.00 - -0.99 meters
- -1.00 - -1.99 meters
- < -2.00 meters

0 1,000 2,000 4,000
Feet

Figure 3-6

LOCAL-SCALE MODEL CALIBRATION RESULTS - BEDROCK AQUIFERS

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



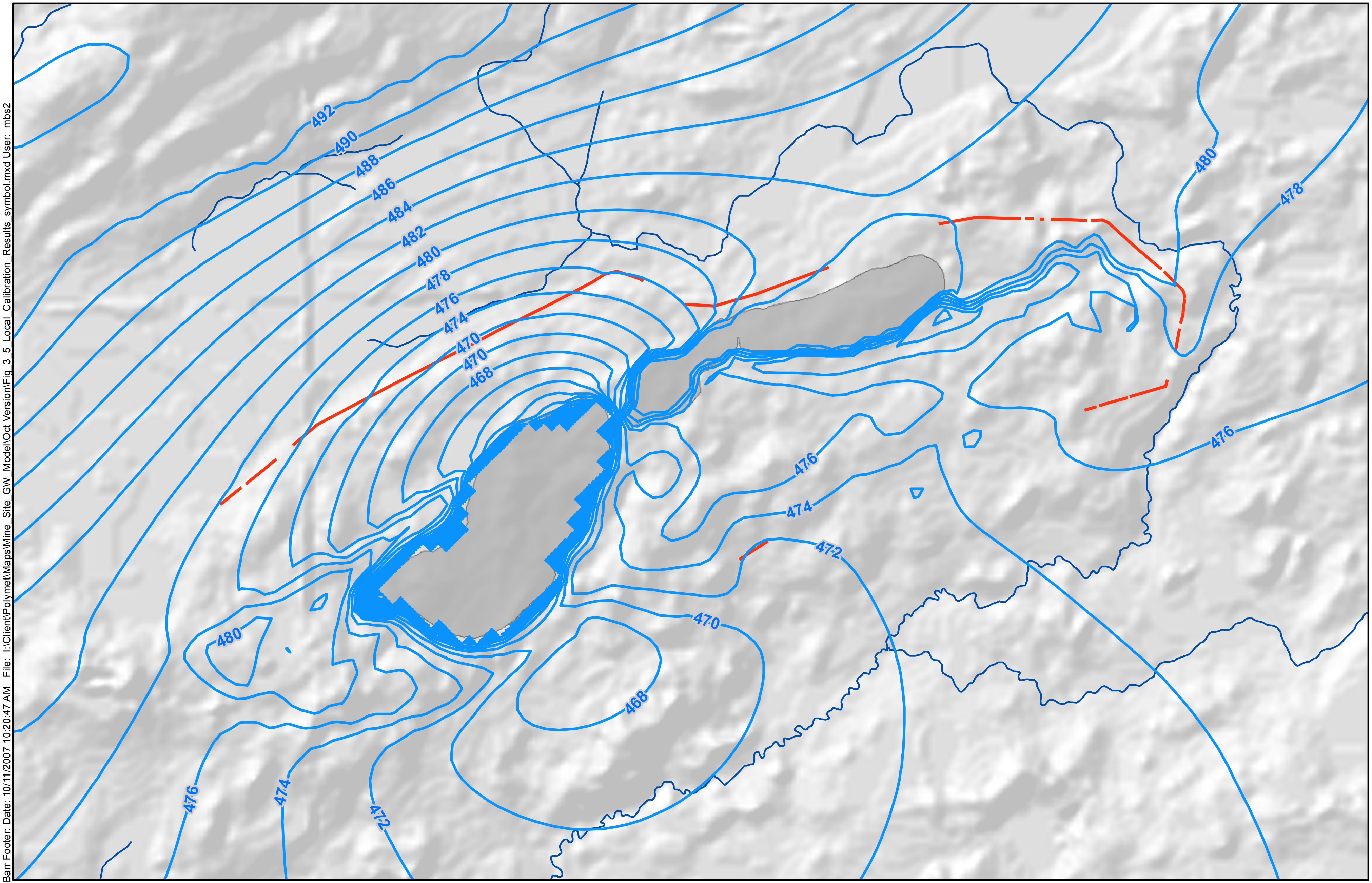
Perimeter Dike
River or Stream
Mine Pits
Simulated Water Table (meters)
Contour Interval 2 meters

0 1,000 2,000 4,000
Feet

Figure 4-1

PREDICTED GROUNDWATER LEVELS
WITHIN SURFICIAL AQUIFER - YEAR 20

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



Perimeter Dike
River or Stream
Mine Pits
Simulated Piezometric Surface (meters)
Contour Interval 2 meters

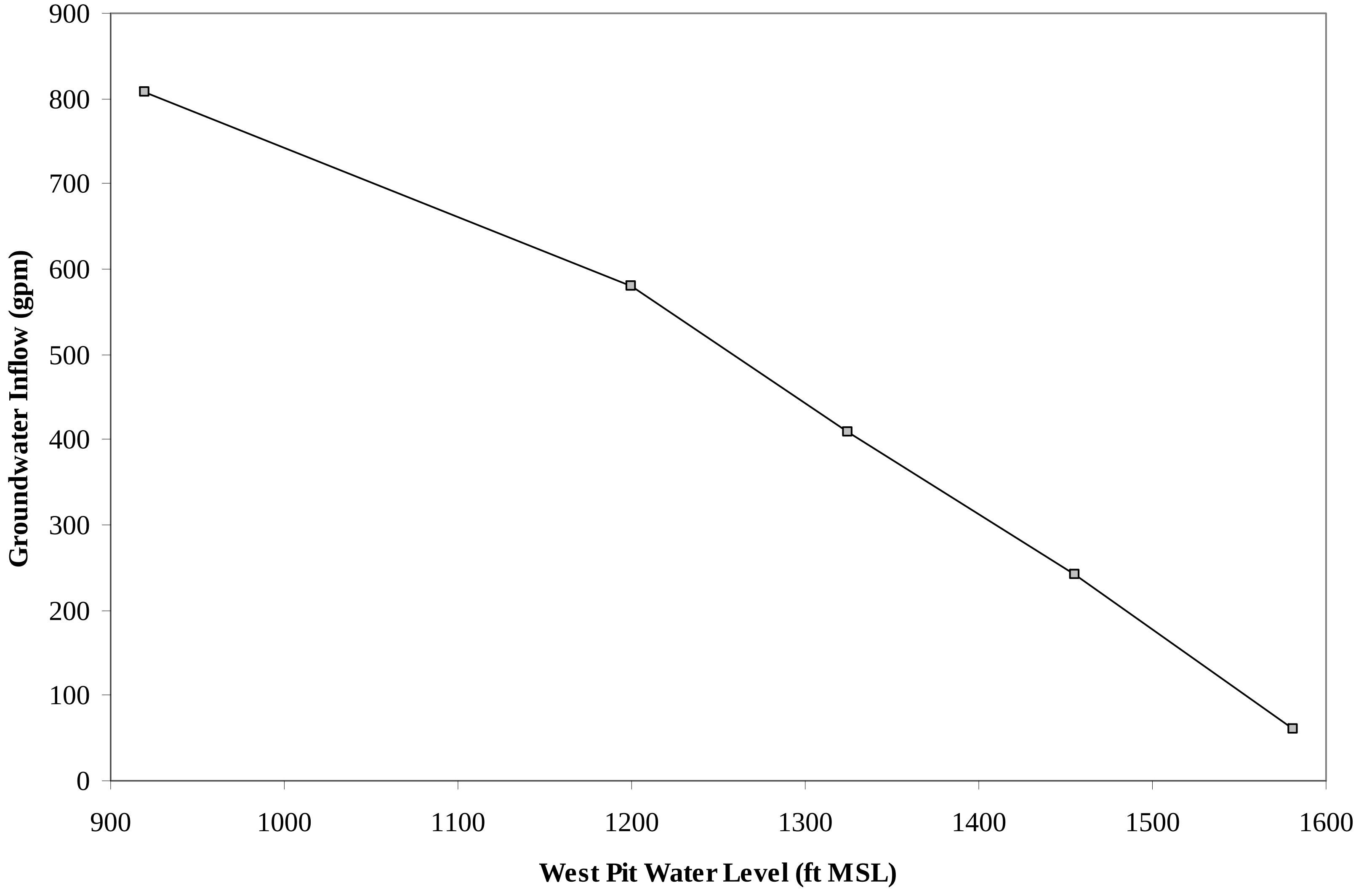
0 1,000 2,000 4,000
Feet

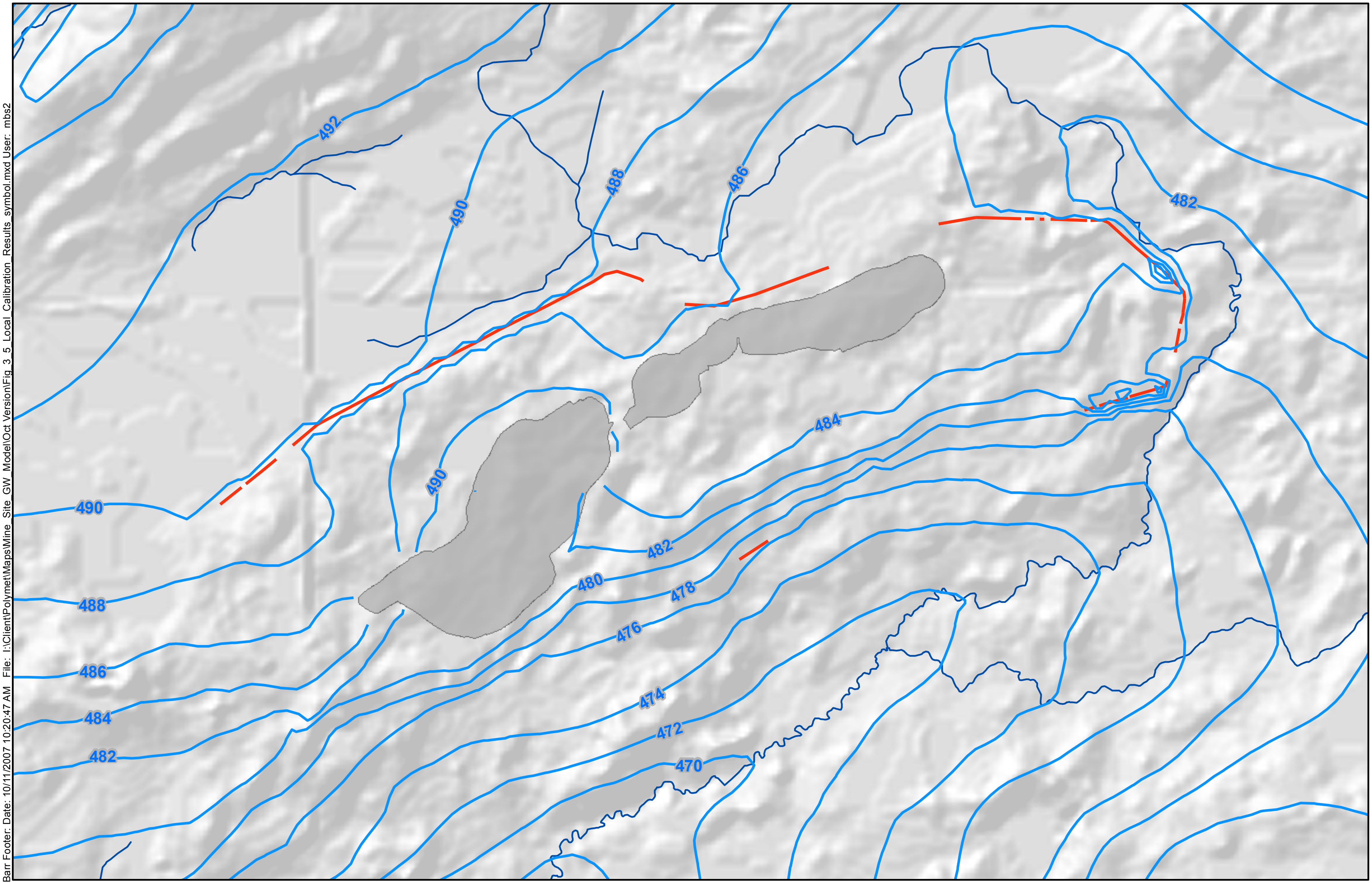
Figure 4-2

PREDICTED GROUNDWATER LEVELS
WITHIN THE BEDROCK - YEAR 20

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN

Figure 4-3 Predicted Groundwater Inflow Rates During West Pit Filling





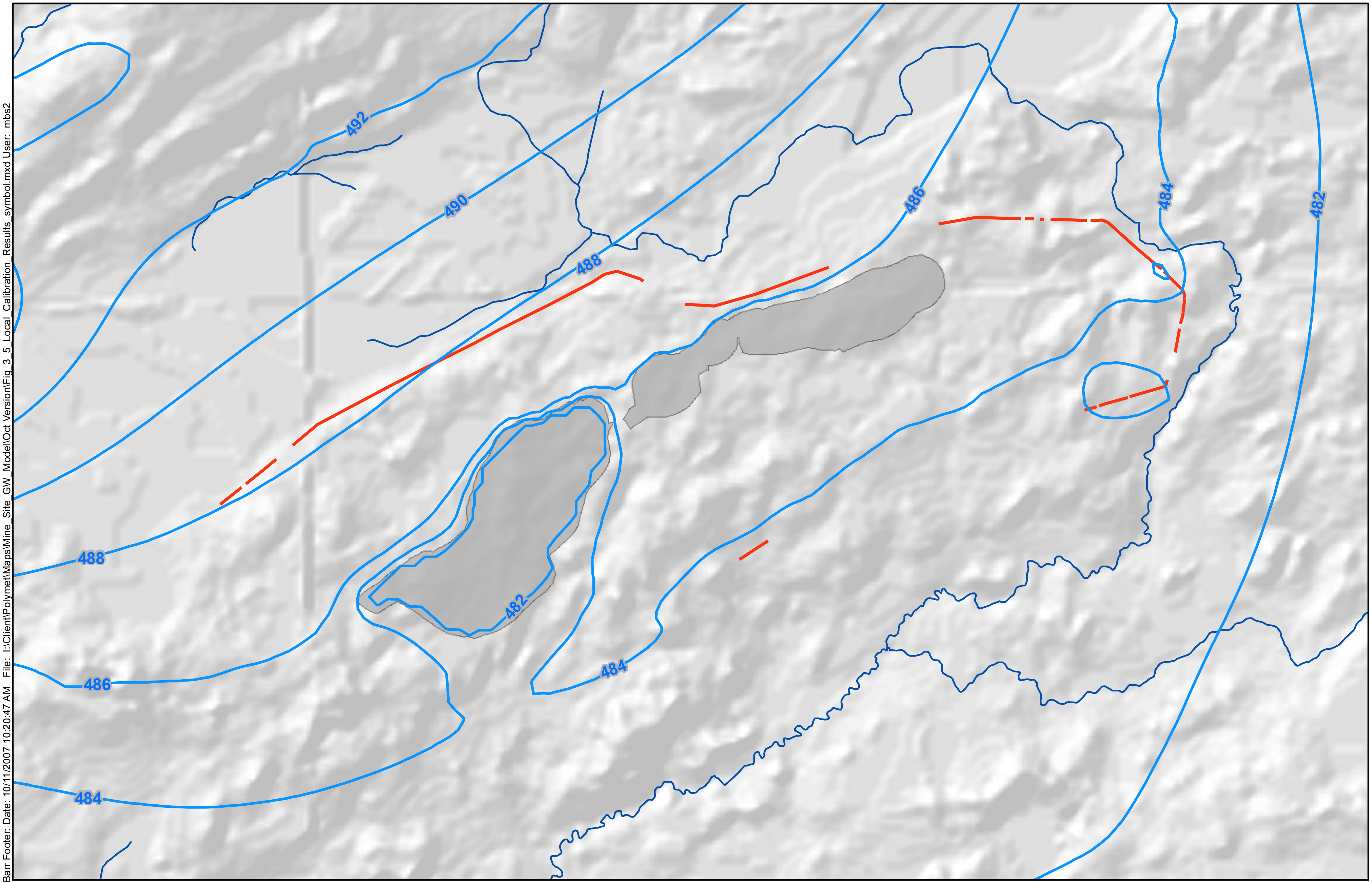
Perimeter Dike
River or Stream
Mine Pits
Simulated Water Table (meters)
Contour Interval 2 meters

0 1,000 2,000 4,000
Feet

Figure 4-4

PREDICTED GROUNDWATER LEVELS
WITHIN SURFICIAL AQUIFER - CLOSURE

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



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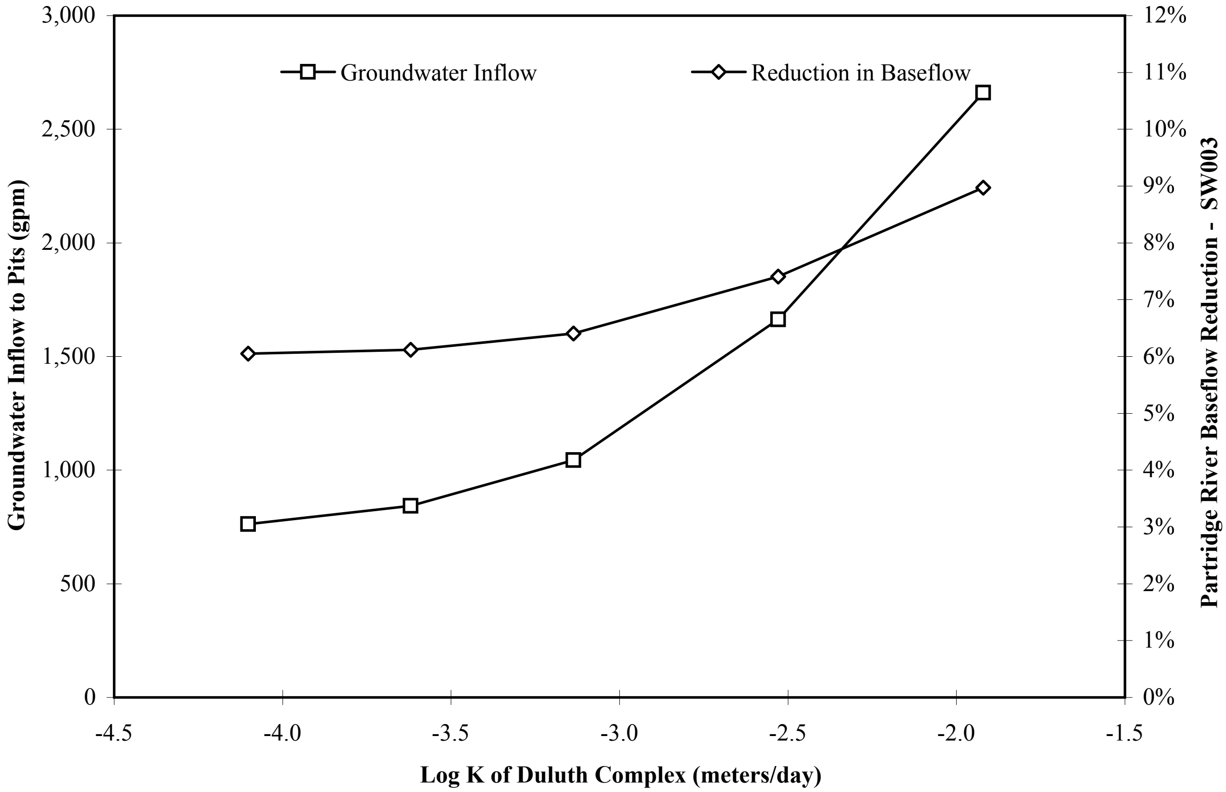
Perimeter Dike
River or Stream
Mine Pits
Simulated Piezometric Surface (meters)
Contour Interval 2 meters

0 1,000 2,000 4,000
Feet

Figure 4-5

PREDICTED GROUNDWATER LEVELS
WITHIN BEDROCK- CLOSURE

NorthMet Project
PolyMet Mining Inc.
Hoyt Lakes, MN



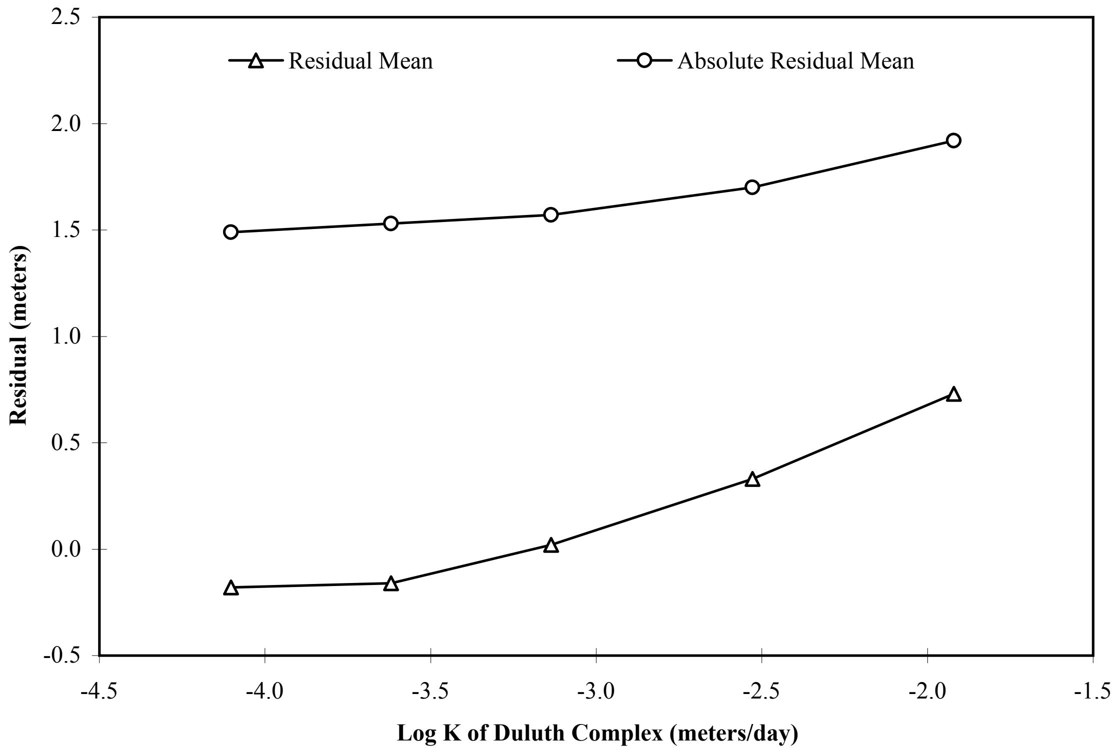
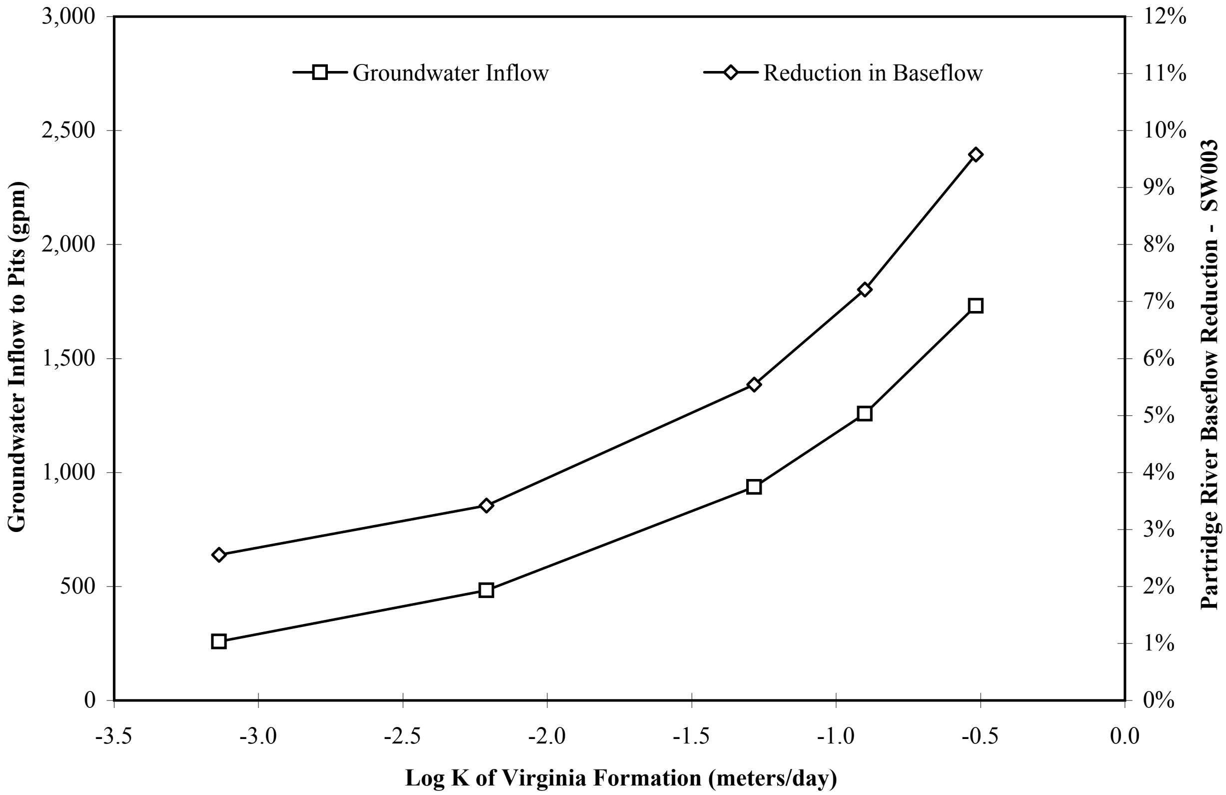


Figure 4-6

SENSITIVITY ANALYSIS
DULUTH COMPLEX
HYDRAULIC CONDUCTIVITY

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN



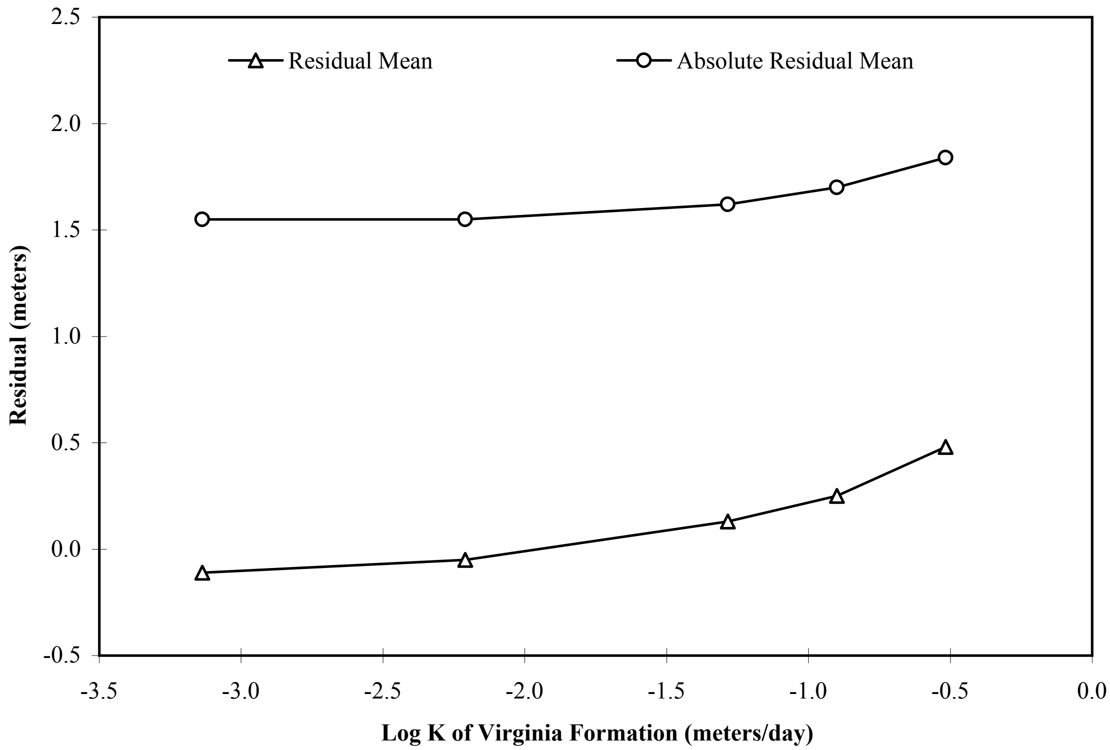
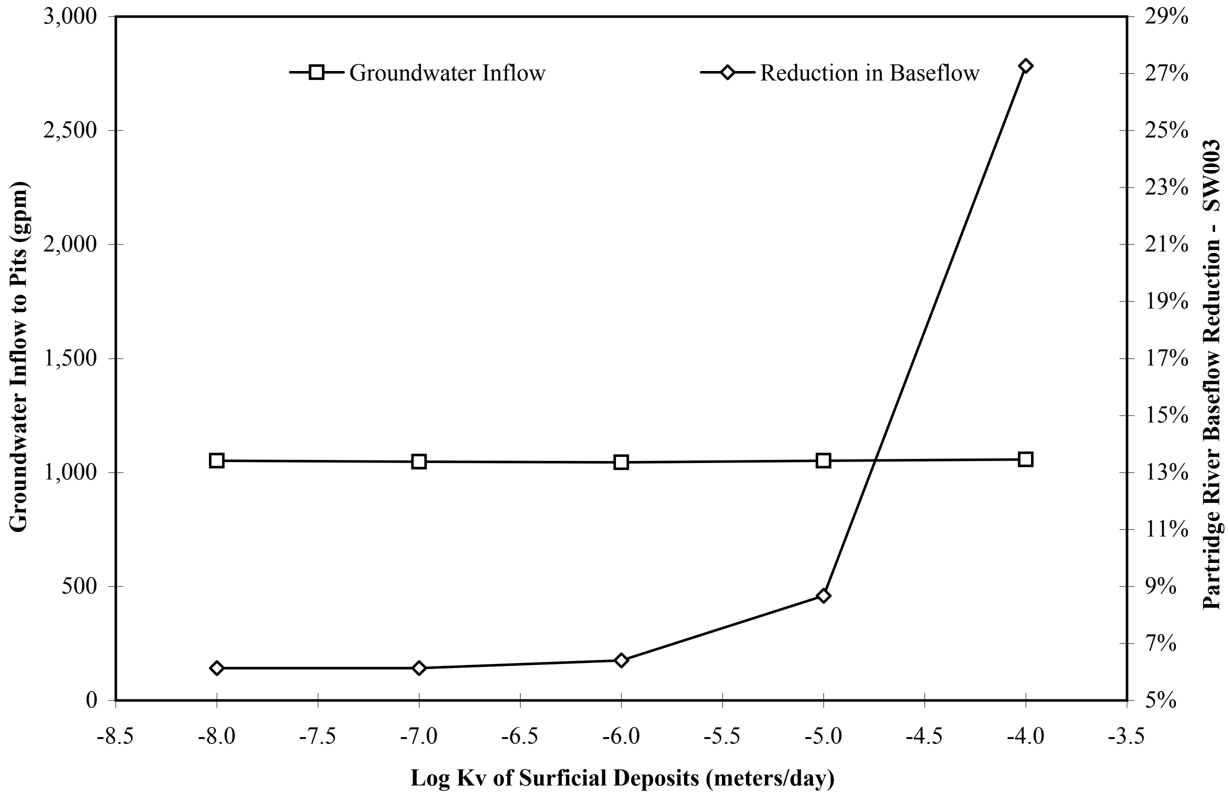


Figure 4-7

SENSITIVITY ANALYSIS
VIRGINIA FORMATION
HYDRAULIC CONDUCTIVITY

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN



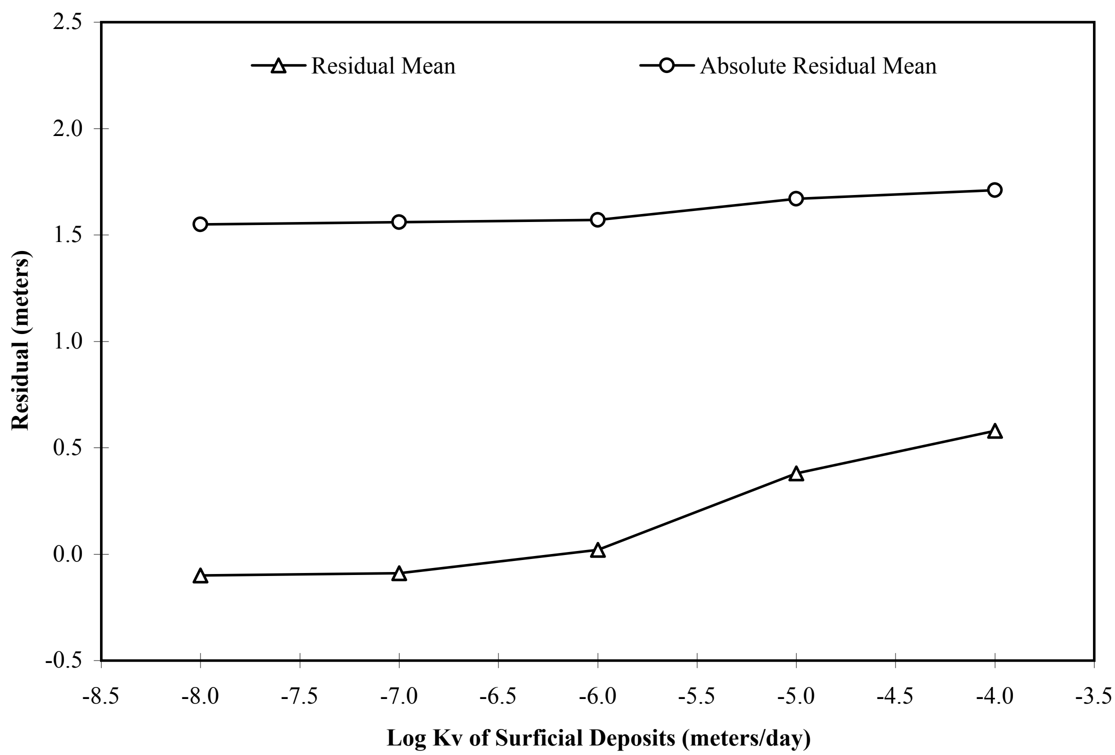
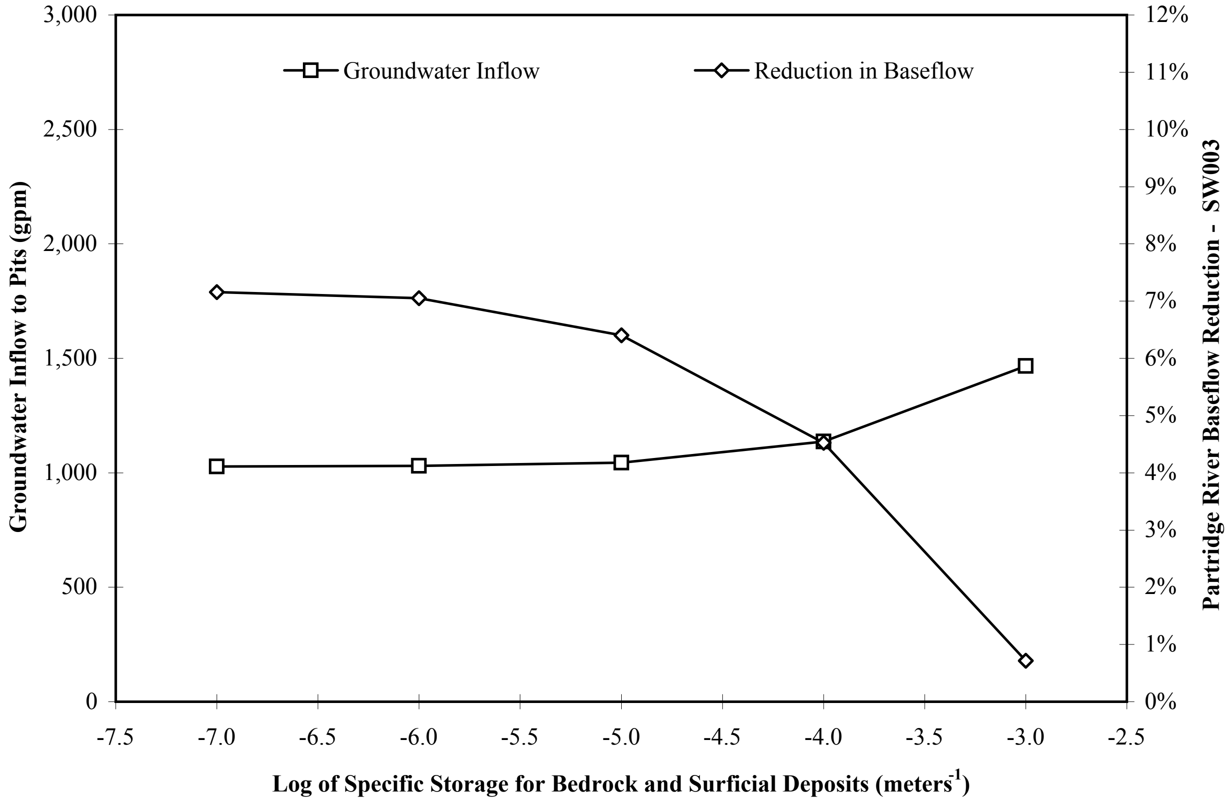


Figure 4-8

SENSITIVITY ANALYSIS
SURFICIAL DEPOSITS VERICAL
HYDRAULIC CONDUCTIVITY

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN



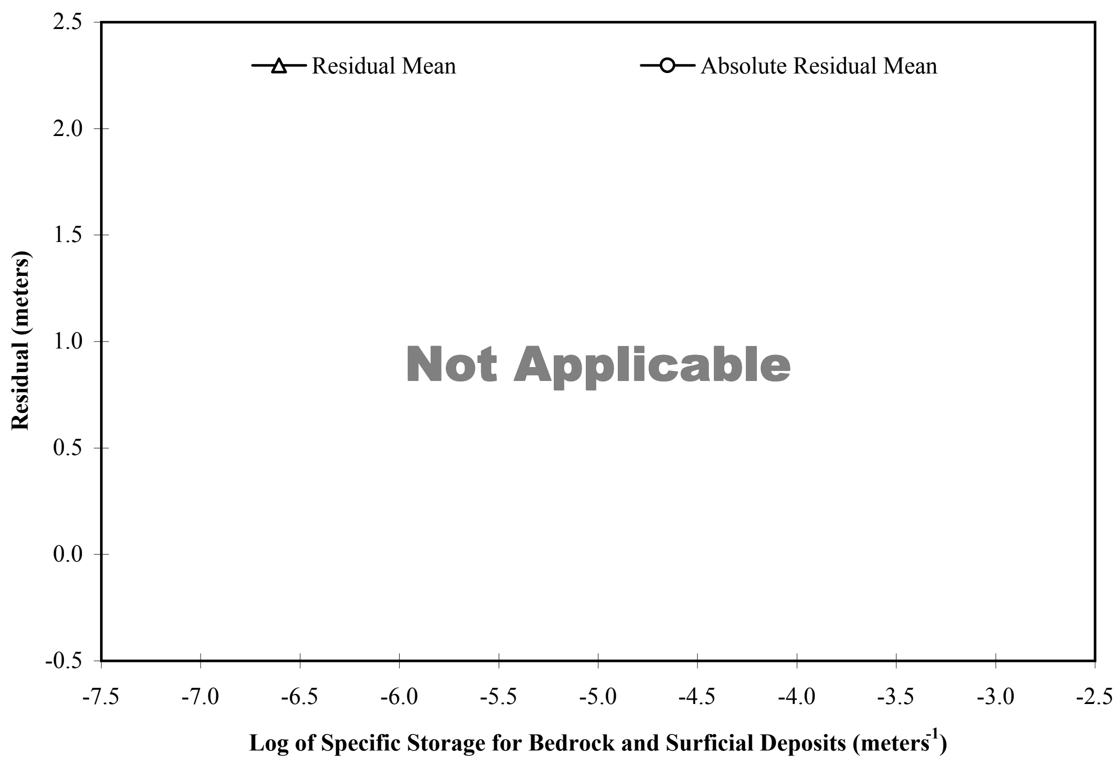
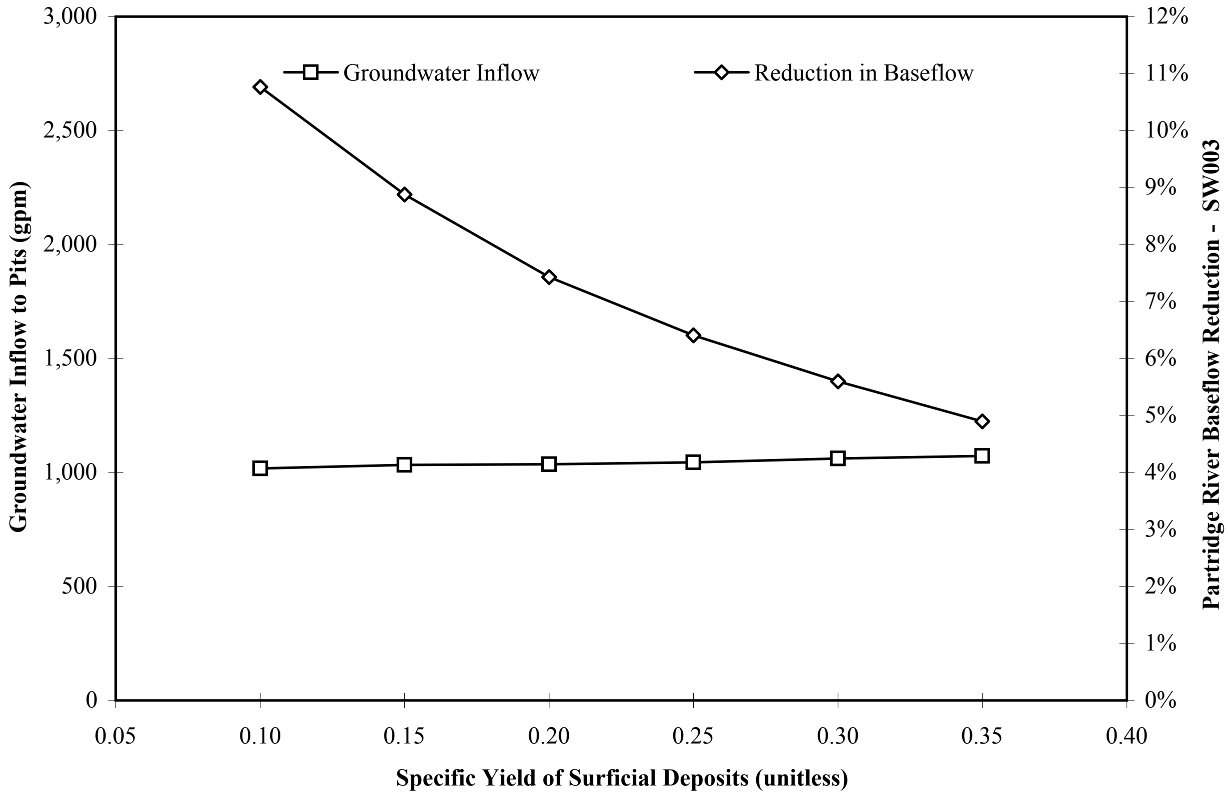


Figure 4-9

SENSITIVITY ANALYSIS
BEDROCK AND SURFICIAL
AQUIFER SPECIFIC STORAGE

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN



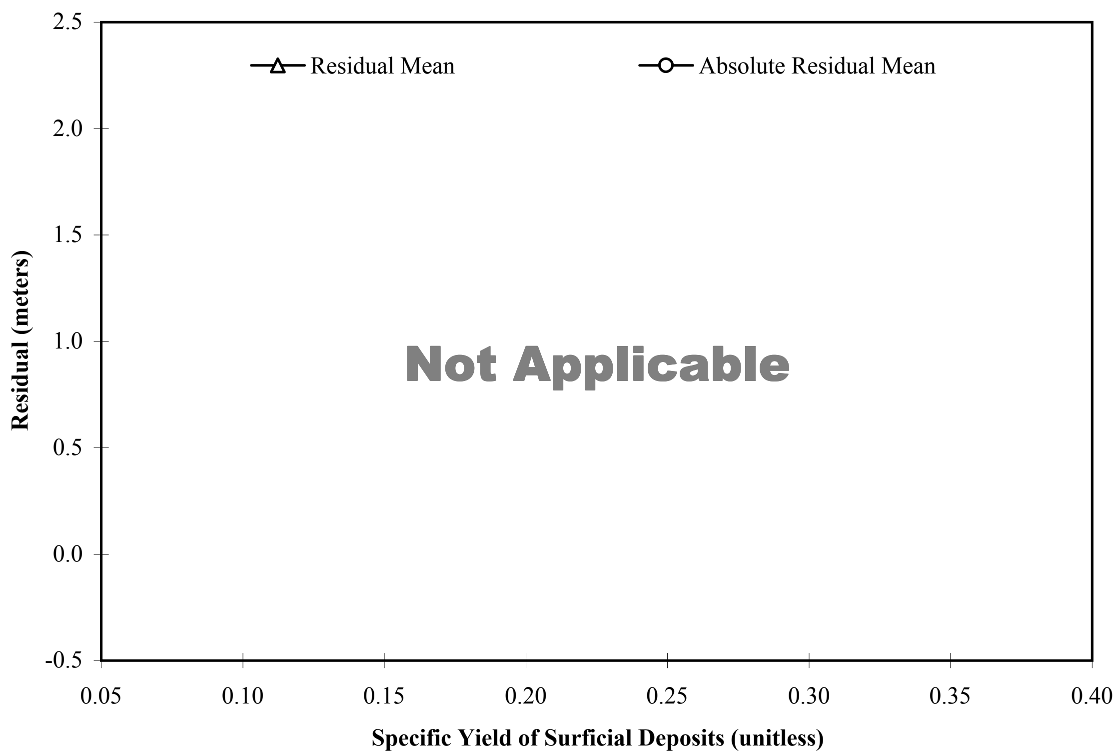
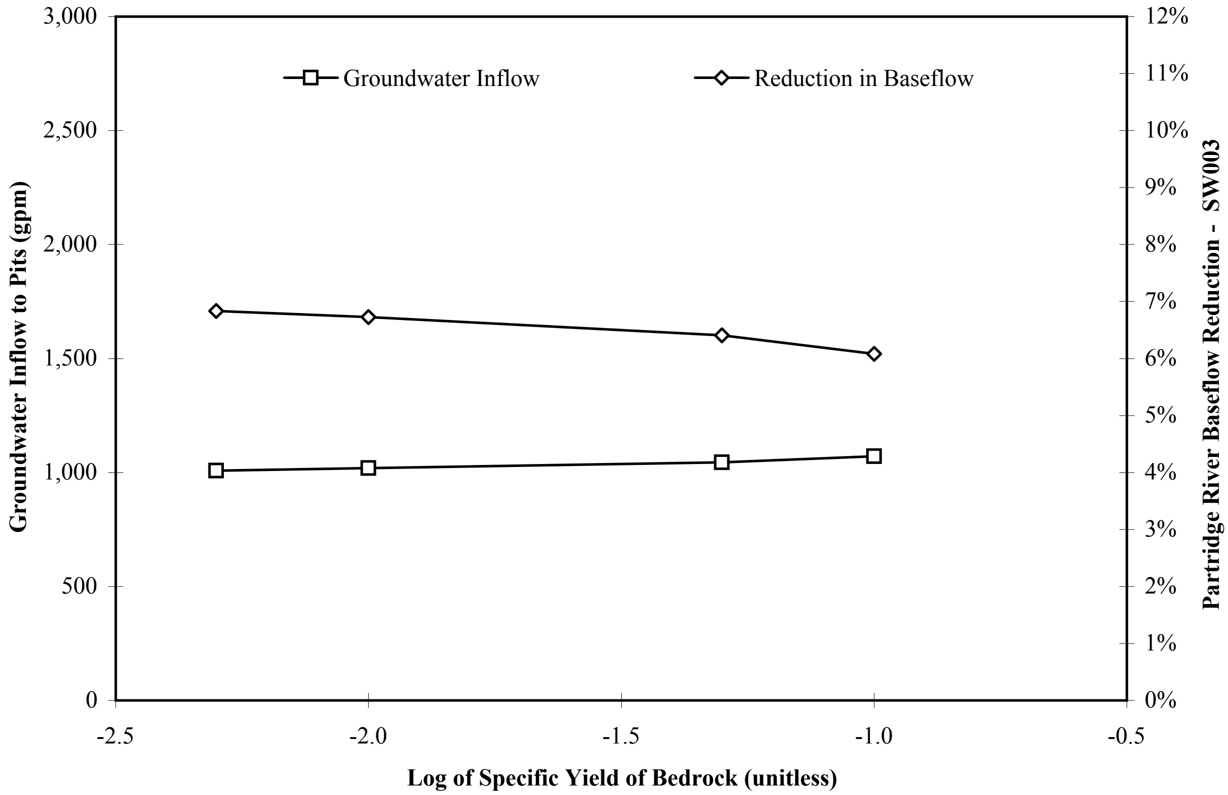


Figure 4-10

SENSITIVITY ANALYSIS
SURFICIAL DEPOSITS
SPECIFIC YIELD

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN



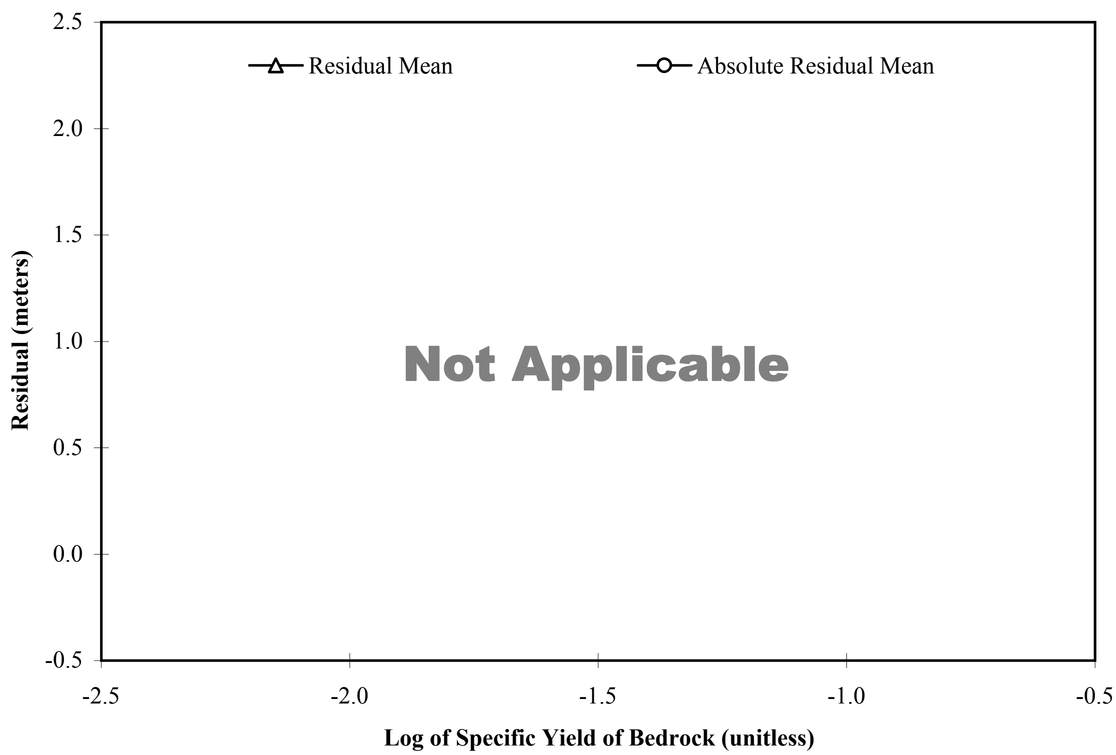
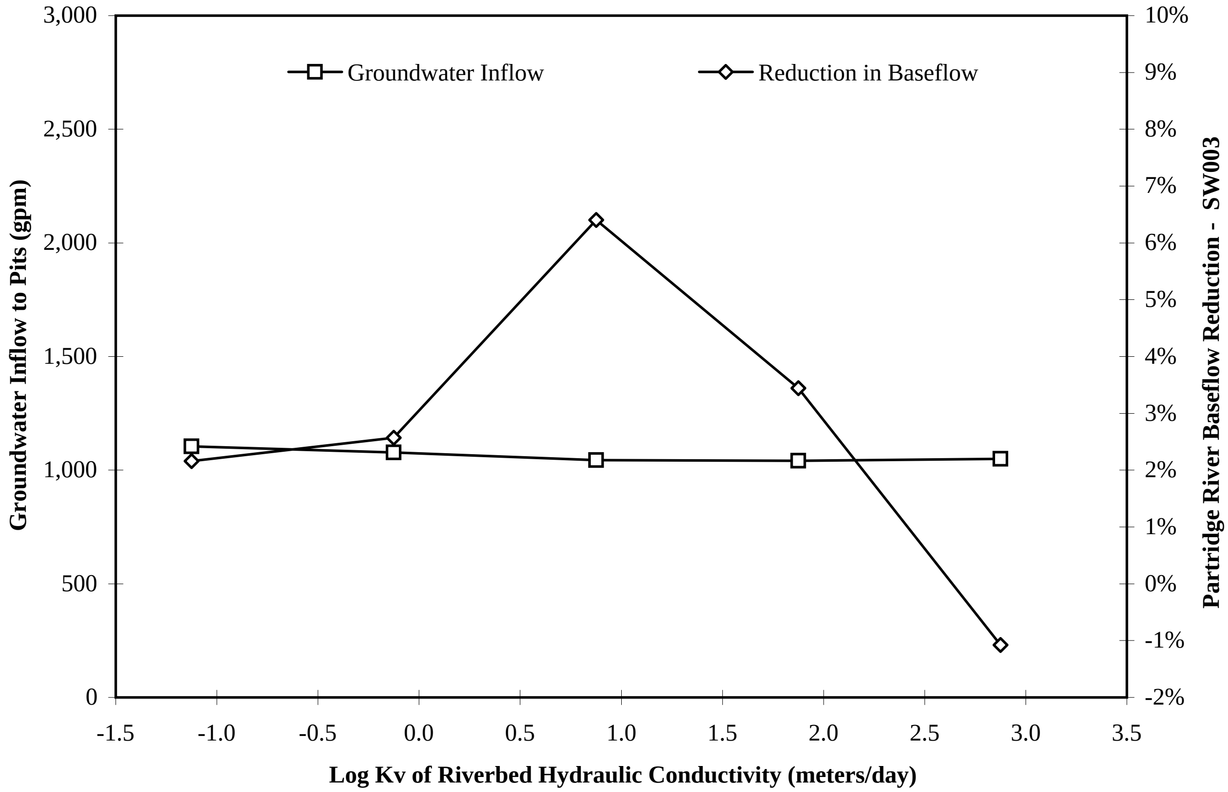


Figure 4-11

SENSITIVITY ANALYSIS
BEDROCK AQUIFER
SPECIFIC YIELD

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN



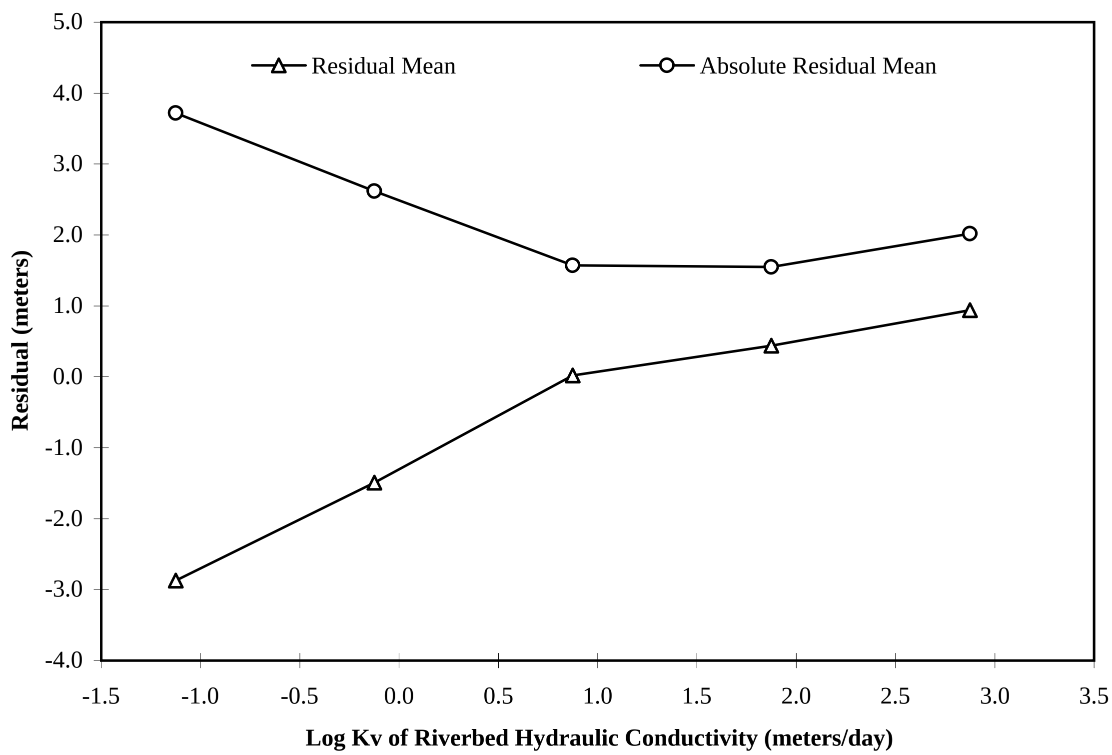


Figure 4-12

SENSITIVITY ANALYSIS
RIVER CELLS VERTICAL
HYDRAULIC CONDUCTIVITY

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN

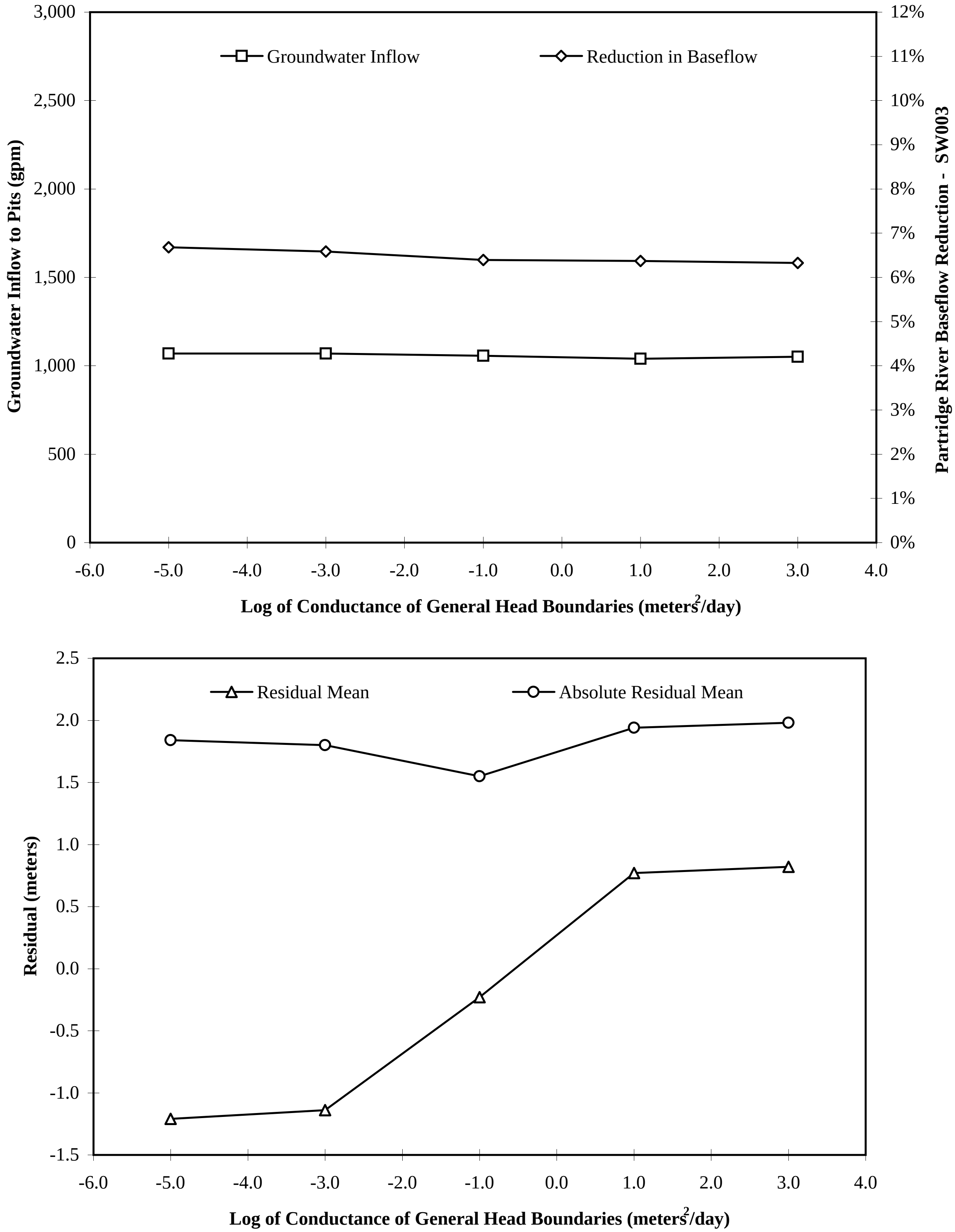


Figure 4-13

SENSITIVITY ANALYSIS
GENERAL HEAD BOUNDARY
CONDUCTANCE

NorthMet Project
PolyMet Mining, Inc.
Hoyt Lakes, MN

Attachment 1

Barr Engineering Company
332 West Superior Street, Suite 600 • Duluth, MN 55802
Phone: 218-727-5218 • Fax: 218-727-6450 • www.barr.com *An EEO Employer*

Minneapolis, MN • Hibbing, MN • Duluth, MN • Ann Arbor, MI • Jefferson City, MO

Technical Memorandum

To: Project File
From: Jeré Mohr and Tina Pint
Subject: NorthMet Bedrock Groundwater Elevation Measurements
Date: January 11, 2006
Project: 23/69-862 007 02D

This memorandum summarizes field activities and data analysis conducted to evaluate groundwater elevations and flow direction at the NorthMet Mine Site (Site). These results will be used for calibration of the groundwater model for the Site.

Proposed groundwater elevation measurement locations were selected to provide relatively uniform coverage across the Site. Groundwater levels were measured on December 13-14, 2006. Due to access issues, final groundwater elevation measurement locations were selected in the field. A total of 31 water levels were measured. The majority of measurements (19) were taken from PolyMet 2005 exploratory drill hole locations, as these were the easiest locations to access and open. Two measurements were collected from 1970s US Steel drill holes. The remainder of the measurements (10) were taken from Barr wells, which were installed in 2005 as part of Phase II of the Hydrogeologic Investigation. Borehole locations and measuring point elevations were surveyed by Northern Lights Surveying and Mapping of Virginia, MN between December 18 and December 29, 2006. The survey was completed using a real-time kinematic GPS survey system. Elevation measurements were referenced to mean sea level (MSL) and x,y coordinates were provided in both UTM (Zone 15 North, NAD83) and State Plane coordinate systems. Depth to groundwater measurements taken in angled boreholes were corrected to vertical depths in order to calculate groundwater elevations at these locations. Measurement locations and groundwater elevations are summarized in Table 1.

In order to check the accuracy of the survey data, the x,y coordinates of each location were compared to x,y coordinates supplied by PolyMet. Apparently, two locations were surveyed incorrectly, as there are large discrepancies (>50 ft) between the surveyed location and the PolyMet provided location. These two locations were removed from Table 1 and are not shown on Figure 1. By comparing the surveyed coordinates to the comprehensive list of borings provided by PolyMet, it believed that the location surveyed as 05-433M is actually 26128 and the location surveyed as 05-442N is actually 05-441. The surveyed ground surface elevations at each location were also compared to TIN elevation data provided by PolyMet. Except for the two locations mentioned previously, surveyed and estimated elevations appeared to generally coincide. As a final check, the casing stick up measured in the field at each location was compared to the stick up calculated using the survey data.

Groundwater elevation measurement locations and contoured groundwater elevations are shown in Figure 1. Groundwater elevations at 05-449N, 05-472N, 50-499N, 05-500Q, and 05-506H appeared to be anomalous and were not used for contouring. Groundwater appears to flow from northwest to southeast, which is generally consistent with the conceptual regional hydrogeologic model.

Barr Footer: Date: 1/11/2007 8:30:49 AM File: I:\Client\Polymet\Maps\Mine_Site_GW_Model\Groundwater_contours.mxd User: JAM2

0 750 1,500

Feet

Groundwater elevation contour (ft MSL)
Contour interval = 2 ft

Groundwater elevation measurement used for contouring (ft MSL)

Groundwater elevation measurement not used for contouring (ft MSL)

Figure 1

GROUNDWATER ELEVATIONS
DECEMBER 13-14, 2006
PolyMet Mining Co.
Hoyt Lakes, MN

Table 1
Groundwater Elevations - December 13-14, 2006
PolyMet Mining, Inc.

Boring/Well	UTM Coordinates (m)		State Plane Coordinates (ft)		Dip (deg)	Borehole length (ft)	Vertical depth (ft)	Ground Elevation (ft)		Surveyed measuring point elevation (ft)	Measured distance to groundwater (ft)	Vertical depth to groundwater (ft)	Groundwater elevation (ft)	Comments
	x	y	x	y				Estimated from TIN	Surveyed					
26054	576265.1	5273526.7	2899614.0	735167.3	90	776	776.0	1598.2	1598.0	1600.32	6.04	6.04	1594.28	
26141	576169.1	5273296.6	2899299.7	734411.7	90	1585	1585.0	1592.4	1592.5	1595.12	2.37	2.37	1592.75	
05-405N	575952.8	5273409.7	2898589.4	734781.8	72	769	731.4	1606.8	1606.2	1607.29	1.40	1.33	1605.96	
05-414N	576265.1	5273331.4	2899614.7	734526.2	65	1438	1297.9	1592.5	1592.7	1593.64	12.10	10.92	1582.72	water level rising
05-424N	576571.1	5273641.3	2900617.4	735544.7	66	1087	992.3	1594.3	1593.6	1595.33	2.59	2.36	1592.97	
05-434N	576621.9	5273570.5	2900784.6	735312.5	65	729	662.8	1593.2	1592.6	1593.44	1.92	1.75	1591.69	
05-447G	577239.3	5274359.2	2902807.2	737903.5	46	499	359.0	1603.6	1605.3	1605.79	8.69	6.25	1599.54	water level rising
05-449N	578719.5	5274808.1	2907662.8	739383.2	64	1136	1016.2	1603.0	1601.7	1602.15	20.19	18.07	1584.08	
05-456Q	579027.4	5275016.7	2908672.1	740068.9	63	1169	1042.1	1598.9	1598.0	1599.47	2.00	1.78	1597.69	
05-472N	578578.1	5274808.2	2907198.6	739382.7	65	925	837.7	1603.9	1605.4	1606.42	8.11	7.34	1599.08	
05-473G	579026.5	5275122.8	2908668.8	740417.3	64	1059	954.2	1612.1	1609.4	1609.49	8.82	7.95	1601.54	
05-487N	578153.6	5274687.2	2905806.2	738983.7	65	906	821.3	1605.4	1604.6	1604.87	7.43	6.74	1598.13	
05-490N	577893.3	5274942.7	2904950.7	739821.4	45	218	154.1	1608.7	1609.5	1609.92	11.53	8.15	1601.77	
05-495N	578356.5	5274697.6	2906472.0	739018.9	66	1208	1100.1	1601.5	1599.8	1600.11	4.24	3.86	1596.25	
05-498N	576935.1	5274197.4	2901809.5	737371.3	66	598	544.2	1604.6	1610.7	1610.94	10.88	9.90	1601.04	
05-499N	576952.1	5273956.0	2901866.4	736579.0	65	908	821.6	1617.9	1620.7	1621.00	7.13	6.45	1614.55	
05-500Q	576775.7	5273922.2	2901287.7	736467.6	64	838	755.1	1614.7	1614.3	1614.64	7.86	7.08	1607.56	
05-505H	578331.7	5275068.9	2906388.8	740237.3	89	298	297.9	1608.4	1609.4	1609.62	7.41	7.41	1602.21	
05-506H	578420.4	5275083.1	2906680.0	740284.3	71	329	311.4	1617.5	1618.9	1619.23	10.98	10.39	1608.84	
OB-1	576938.8	5274551.3	2901820.1	738532.7	90	100	100.0	1610.2	1611.1	1613.21	12.60	12.60	1600.61	
OB-2	578216.3	5275040.0	2906010.4	740141.9	90	100	100.0	1609.0	1608.7	1610.70	8.75	8.75	1601.95	
OB-3	578710.1	5275261.2	2907629.7	740870.1	90	100	100.0	1615.9	1616.1	1617.85	10.47	10.47	1607.38	
OB-3A	578711.5	5275263.2	2907634.5	740876.5	90	50	50.0	1615.8	1615.8	1617.05	9.39	9.39	1607.66	
OB-4	578893.1	5275409.5	2908229.6	741357.3	90	100	100.0	1618.7	1619.5	1620.94	15.32	15.32	1605.62	
OB-5	579292.7	5275528.9	2909540.5	741750.9	90	100	100.0	1609.0	1609.3	1611.73	11.93	11.93	1599.80	
P-1	577016.0	5274605.2	2902073.2	738709.9	90	610	610.0	1614.1	1616.9	1617.79	16.53	16.53	1601.26	
P-2	578294.3	5275068.9	2906266.2	740237.3	90	610	610.0	1606.2	1606.3	1607.85	5.70	5.70	1602.15	
P-3	578730.5	5275289.7	2907696.7	740963.5	90	610	610.0	1615.4	1615.0	1619.57	11.86	11.86	1607.71	
P-4	579248.2	5275468.8	2909394.6	741553.5	90	485	485.0	1607.0	1608.2	1609.66	10.81	10.81	1598.85	

Appendix C

U.S. Forest Service Soil Map Descriptions (Draft)
Superior National Forest Ecological Classification System
(See Figure 4.1.3.3-A)

RS22, APPENDIX C
USFS SOIL MAP DESCRIPTIONS – ONLY DESCRIPTIONS FOR THOSE AT THE MINE SITE
(SEE FIGURE 4.1.3.3-A)

ECOLOGICAL LANDTYPE
Lowland Loamy Moist – LLM 1

Background

This Ecological Landtype (ELT) is scattered throughout the Forest and occupies about 4 percent of the land. It is common in Landtype Associations 1, 3, 5, 8a, 9,10a, 10b, 11, 12, 18, 19, and 20. This ELT encompasses Landtype Phases 7, 34, 38, and 62. This unit is classified as riparian and is classed as upland according to the wetland classification system.

Terrain and Geology

This ELT occurs on slightly concave, simple slopes which have a gradient of less than 6 percent and are associated with nearly level to gently sloping, somewhat poorly drained ground, moraines, recessional moraines, and drumlins comprised of Rainy and Superior till. Landscape positions include faults, drain-ways, lower slopes in transition to swamps, and depressions. Local relief is very low.

Soil Resources

The population of mineral soils associated with this ELT has developed in deep, somewhat poorly drained, yellowish and reddish-brown, sandy loam, loam, clay loam, and silt loam till. The depth to bedrock is more than five feet. Aquic Dystrochrepts, Aquic Glossoboralfs, and Aquic Udipsomments make up the populations of soils. An estimated 5 to 20 percent of the root zone is gravel, cobbles, and boulders. Depth to seasonally saturated soil ranges from 12 to 40 inches. Shallow rooting by trees is common. Natural fertility is reduced because of the coarse fragments and seasonally saturated soil. The latter also results in lower soil temperatures in the root zone during the growing season.

Water Resources

Some surface flow and ponding occurs on this unit during the spring and high intensity short duration storms. Water flowing from adjacent uplands collects in this ELT and gradually flows into poorly drained wet areas or percolates into ground water systems. This ELT is generally associated with intermittent first order streams. Water commonly flows through this unit in subsurface cobbly channels in situations where thin deposits over bedrock dominate the surrounding uplands.

Vegetation

This unit supports vegetation communities that are transitional between uplands and lowlands; therefore, upland and lowland species are often present in a community. Upland trees except red pine frequently dominate the overstory. However, the shrub and forbs layers are made up of species having high moisture requirements. A jack pine overstory can be associated with a shrub layer comprised of tag alder and red osier dogwood.

Shrubs common to this unit are more than two feet in; height with some over four feet tall. The species include tag alder, red osier dogwood, willow, and some elderberry. At some positions, there may be an inclusion of beaked hazel and mountain maple. The forbs are made up of broadleaf plants, with mosses.

The potential structure and density is:

	Mature	Immature	Juvenile
Hardwoods	Os-4, Sh-1, Fo-2	Os-4, Sh-0-1, Fo-2	Os-0, Sh-4, Fo-3
Pines	Os-4, Sh-1, Fo-2	Os-4, Sh-0, Fo-1	Os-0, Sh-4, Fo-0
Fir, Spruce	Os-4, Sh-1, Fo-1	Os-4, Sh-0, Fo-0	Os-0, Sh-4, Fo-0

The duff is dark brown and black in a mature hardwood stand. Wind thrown trees are common and the knolls typically support a forbs community that is in contrast to the surrounding areas.

Micro Climate

The climate associated with this unit is moist and cool during the growing season. Frost pockets are common in openings and probably result from the collection of cold air. Snow and frost remain longer in this landtype than adjacent upland landtypes. Because of these factors, the temperature in the root zone rises very slowly. These micro climatic features probably determine the growing season.

Interpretations

Engineering: Hydrological properties severely restrict engineering activities. Water collects from surrounding uplands creating unstable saturated soil conditions, especially during the spring, early summer, and fall seasons. Infiltration and permeability rates are moderate. The surface five feet has a Unified Classification of SM and ML. This unit is poorly suited for road locations. High construction and maintenance costs can be appended for this unit. This ELT is unsuited for soil absorption fields and sanitary landfill developments. Alternate sewage disposal systems should be considered, if needed. Brush control and seasonal limitations for vehicle access are problems relating to utility corridors.

Fire: This ELT provides a break in fuel types from adjacent plant communities. The low shrub density in mature stands results in a poorly developed fuel ladder. Fine fuels consist of duff and broadleaf forbs, which ordinarily have high moisture content due to the moist site. Ground fog, frost, cool ground temperatures, and high humidity near the surface are more common in this unit than in the surrounding uplands units. Explosives, hand tools, and heavy equipment are suitable for constructing fire lines. Heavy equipment must be directed away from unstable, wet soil conditions. For site preparation, prescribed summer burns will be necessary. Burning will not adversely affect the productivity of this ELT.

Recreation: This unit is not suited for most intensive and dispersed recreation facilities. Wind throw hazard is moderate to high for pole and sawtimber. Trails, spurs, and roads will require design features that will solve the water related problems. On-site sources of borrow are inadequate for most development needs.

Timber: Potential productivity is fair to good for pulpwood and poor for sawtimber. Heavy equipment operability is poor on this unit. Shallow root systems result from prolonged saturated condition in the root zone and wind throw hazard is moderate to severe for pole and sawtimber. Plant competition, saturated soil, and short-term ponding can reduce the survival of seedlings. Mechanical site preparation is limited to the period when the ground is frozen. Recommended species are black ash, balsam fir, yellow birch, white cedar, black spruce, white spruce, trembling aspen, and tamarack.

Wildlife: Diversity on this unit is medium. The diversity relates to the structure and density of plant communities and the inclusions of other ELT's, which range from open grass meadows to fully stocked hardwood or conifer stands. The unit has a fair potential for managed food plots because of the poor workability associated with the moist loamy and silty soils. Considerable edge occurs in the transition from the upland dry sites to the lowland wet sites. Intermittent streams and ponding from surface runoff are common. This unit has a fair potential for ponds. Beaver activity is widespread in this unit.

ECOLOGICAL LANDTYPE
Lowland Loamy Wet – LLW - 2

Background

This Ecological Landtype (ELT) occupies about 9 percent of the Forest. This ELT is common throughout the Forest except in Landtype Associations (LTA) 2, 7b, and 16. This unit is comprised of Landtype Phases (LTP) 1, 22, 47, 71, 72, and 73, is classified as riparian, and according to wetland classification, is Palustrine, Forested Wetland, or Scrub-Shrub Wetland.

Terrain and Geology

This ELT occurs on slightly concave, uniform slopes which have a gradient of less than 6 percent and are associated with nearly level to gently sloping, poorly drained ground moraines comprised of Rainy and Superior till. Landscape positions include broad drain-ways and depressions, floodplains, and swamps. The local relief is very low.

Soil Resource

The population of mineral soils associated with this unit are developed in deep, poorly drained, yellowish-brown and reddish-brown sandy loamy to clay loam tills. Aquepts prevail in this unit and thick A1 horizons are widespread. Occasionally, there is an accumulation of organic matter that ranges from 6 to 18 inches thick. About 10 to 25 percent of the root zone is gravel, cobbles, or boulders and locally these range to about 90 percent. These soils are saturated to the surface about 85 percent of the year. Very shallow rooting of trees is common. The natural fertility of this unit is low. Average forest floor (duff) layer on this unit is 5 to 6 inches.

Water Resource

The water flowing to this unit moves as surface and subsurface flow. In the spring, fall, and during high intensity short duration storms, floodplains along intermittent and narrow, shallow perennial streams become active. The floodplain will vary in width from a few feet to 75 feet and the depth of water will range from less then 1 inch to about 8 inches. Water collects in this unit and flows to open water bodies, with some percolating into the soil and to subsurface water systems. Streams include first, second, third, and fourth order. Surface ponding is widespread.

Vegetation

This unit supports plant communities typical of wetlands. Lowland hardwoods and conifers are the major stand types. Trembling aspen and paper birch grown in some locations of this unit. Plants that prevail in this unit have adapted to prolonged saturated and cool root zones.

Shrubs common to this unit are tag alder, Labrador tea, willow, leatherleaf, red osier dogwood, ribes, and blueberries. Shrub height is variable and ranges from 2 to 7 feet. Broadleaf plants and some mosses make up the forbs.

The potential structure and density is:

	Mature	Immature	Juvenile
Lowland hardwoods	Os-3, Sh-1, Fo-3	Os-3, Sh-0, Fo-2	Os-0, Sh-4, Fo-1
Lowland conifers	Os-3, Sh-0, Fo-4	Os-3, Sh-0, Fo-2	Os-0, Sh-4, Fo-1

Wind throw is common and the knoll created by the root mass results in a contrasting better-drained site and associated plant communities. Well-established open shrub and grass communities will exist for many decades in this unit.

Micro Climate

The climate is wet and cool during the growing season. Frost pockets and cold air drainage are widespread in this unit. Snow and frost remain in this unit long after the uplands are clear of both. The temperature in the root zone remains low through the growing season. The duration of the growing season is determined by these site conditions.

Interpretations

Engineering: Unfavorable hydrological properties severely restrict many engineering activities. The surface 5 feet has a Unified Classification of SM or ML. Surface flooding, prolonged saturated soil and frost conditions, and relative high contents of silt and clay result in costly construction and maintenance projects. There is a very low potential for sources of borrow. This unit is unsuited for sewage disposal and sanitary landfills. Local conditions will severely restrict the construction and maintenance of utility corridors.

Fire: This unit can function as a natural break in fuel types and can be a source of water. Plant communities on this unit contrast sharply with adjacent plant communities. In well-stocked mature stands, the fuel ladder will be incomplete because of the very low density or lack of a shrub layer. Poorly stocked stands can have well developed grass and thus a continuous arrangement of fine fuels. In some situations, an organic mat underlain with boulders will become very dry during droughts or extended seasonal dry periods. Ground fires in those situations will be difficult to control. Explosives and Bombardiers are practical tools for constructing fire lines in this unit. Prescribed burning is an acceptable management practice in this unit but during abnormally dry periods burning should be restricted to spring or late fall. Ordinarily, fire will not result in an adverse impact on this unit.

Recreation: This unit is unsuited for facilities associated with intensive recreation developments and for most dispersed recreation facilities. Trail and road design will have to allow for wet site conditions.

Timber: This unit has a low productivity potential for growing wood fiber. Operability is poor due to wetness. Mechanical site preparation and harvest activities are limited to periods when the ground is frozen. There is a severe wind throw problem resulting from the permanent wet conditions and associated shallow root systems. Brush competition in managed stands is low, however, hardwood sprouting will have to be controlled when converting to softwoods. Wetness and local concentrations of boulders will hamper tree-planting operations. Seeding and natural regeneration can be used in this unit. Recommended trees are lowland hardwoods and conifers.

Watershed: Water flows slowly through this unit. Infiltration and permeability rates of the soils are moderate to slow. This unit collects water in large amounts from surrounding areas and most of this water into open water systems. Some water infiltrates the soil and percolates to underground water systems. The potential for erosion is low. There is a fair potential capacity for buffering lakes and streams from chemicals applied to the watershed.

Wildlife: Natural diversity is low but contrasting properties (vegetation, gradient, moisture, and natural openings) with adjacent uplands result in considerable edge. There is a good potential for wildlife ponds. Many natural small ponds collect surface water and are quite warm during the summer. The potential for managed food plots is poor because of the wet site conditions. Beaver and woodcock activity on this unit is common.

ECOLOGICAL LANDTYPE
Lowland Organic Acid to Neutral– LPN - 6

Background

This Ecological Landtype (ELT) occurs in Landtype Associations throughout the forest and occupies about 16 percent of the land. This unit is comprised of Landtype Phases (LTP) 24 and 32. It is classified as riparian and according to wetland classification, is Palustrine Forested Wetland or Scrub-Shrub Wetland.

Terrain and Geology

This unit occurs on uniform slopes having a gradient of less than 6 percent and is associated with nearly level plans and ground moraines having a thick deposit of organic material. Landscape positions include drain-ways, basins, and bogs. The local relief is very low.

Soil

The organic soils associated with this unit have developed in thick (the majority are more than 5 feet thick) deposits of organic materials derived from decaying woody plants and forbs. These deposits are underlain with glacial drift and lacustrine sediments. These soils are classified as Borohemist and Terric Borohemist. Gravels, cobbles, and boulders are very uncommon and are absent in most areas. The pH ranges from strongly acid to neutral. The soils are permanently saturated.

Water

Typically, the water from adjacent uplands collects in this unit and flows as runoff to open water systems. The runoff is most common in the spring. Flowage through this unit is slow due to the irregular and hummocky ground surface. Some water percolates into the mineral substrata. This unit is associated with third and fourth order streams.

Vegetation

This unit supports a wide range of plant communities, which are adapted to permanently wet organic soils. Spruce and fir are common in the overstory. Less common are tamarack, black ash, and white cedar. The height of the shrubs ranges from 2 to more than 4.5 feet and the common species are tag alder, red osier dogwood, Labrador tea, leatherleaf, and less common is bog birch. The forbs consist of a variety of broad leaf plants and a few grasses and mosses.

The potential structure and density is:

	Mature	Immature	Juvenile
Lowland hardwoods	Os-2, Sh-2, Fo-4	Os-3, Sh-1, Fo-4	Os-0, Sh-4, Fo-3
Lowland conifers	Os-4, Sh-2, Fo-4	Os-4, Sh-0, Fo-4	Os-0, Sh-4, Fo-2

Plant communities with a low overstory typically have a wide variety of shrubs and forbs. However, as the conifer overstory closes the variety of these is reduced to a relative few.

Micro Climates

This unit is wet and cool in the growing season. Frost pockets and cold air drainage is widespread. Snow and frost remain until late spring or early summer. The root zone is wet and cool and the temperature rises very slowly. Fog and high humidity near the ground are widespread.

Interpretations

Engineering: Hydrological properties, saturated conditions, and unstable peat severely restrict engineering activities. Water collects in this unit and flows away very slowly. The organic material is typically 6 to 10 feet thick and is poorly suited for road locations. This unit is unsuited for sewage disposal and sanitary landfills. Vehicle access seriously limits the use of this unit for utility corridors.

Fire: This unit will provide a meaningful break in fuel types from adjacent uplands in many situations. A well-stocked mature black spruce can have a complete full ladder. The water-saturated mosses are an effective fuel break for ground fires. During drought conditions, the organic material can become sufficiently dry enough to create conditions that could lead to a "peat fire". Ground fig, frost, cool ground temperatures, and high humidity near the ground surface are common. This unit is a source of water. Fire line construction is best accomplished with explosives or bombardiers, with hand tools being less effective. Heavy equipment is not suited for constructing fire lines. Fire lines will have open water. Except during droughts, fire will have little impact on this unit and can be used for site preparation.

Recreation: This unit is unsuited for recreation developments. Wind throw hazard is very severe for pole and sawtimber trees. Trails and roads will require design features that will solve the problems associated with the permanently saturated deep organic material. There are no on-site sources of borrow. Sewage disposal facilities would have to be located elsewhere.

Timber: Potential productivity for pulpwood is low and for sawtimber is very low. Wind throw is common in mature stands. The operability is poor. Shallow root systems are widespread due to the prolonged saturated condition. Surface ponding is common. Mechanical site preparation is limited to the frost period. Recommended species are black spruce, white spruce, white cedar, balsam fir, tamarack, and black ash.

Watershed: Permeability rate is slow and the water holding capacity is high. Standard conditions and ponding are commonplace. Potential for an aquifer is undetermined at this time.

Wildlife: Diversity in this unit is low. Some edge is present at the contact with the uplands. Vegetation is typical of moderately acid to neutral bog conditions. Ponds could be easily developed and water will often have a high content of suspended organic material. Evidence of wildlife activity typically consists of trails, beds, and browsing.

ECOLOGICAL LANDTYPE
Upland Deep Loamy Dry Course – UDLDC - 13

Background

This Ecological Landtype (ELT) is widespread and occurs in all Landtype Associations (LTA) except 3, 6, 7a, 12, 14, 17, and 18. This unit occupies about 8 percent of the land and is comprised of Landtype Phases (LTP) 3, 9, 30, 33, 39, 64, 66, and 70. It unit is classified as non-riparian and according to wetland classification is Upland.

Terrain and Geology

This unit occurs on convex slopes that have a gradient, which ranges from 6 to 35 percent and is associated with Rainy and Superior ground moraines. Landscape positions include ridges and side slopes. The local relief is low.

Soil

The population of mineral soils has developed in deep, well drained, grayish-brown, and brown, sandy loam and loamy sand that have a water table below 5 feet. These soils are classified as Typic Dystrochrept. These soils commonly have a loess cap of about 12 to 30 inches that is underlain with till. Gravels and cobbles make up about 15 percent of the loess cap and about 35 percent of the till. In the till, this percent will range from 15 to 70 percent of which a major portion consists of boulders. The content of coarse fragments on the ground surface is typically less than 5 percent.

Water

This unit yields water to ground water systems. There is no surface runoff or ponding while the soil is free of frost.

Vegetation

This unit supports plant communities that are representative of dry uplands having a moderate level of fertility. Overstory species include aspen, paper birch, pine, spruce, and fir. Shrubs include hazel, honeysuckle, and mountain maple. Broad leaf plants, some grasses, and few mosses make up the forbs.

The potential structure and density are:

	Mature	Immature	Juvenile
Hardwoods	Os-4, Sh-2, Fo-2	Os-4, Sh-1, Fo-1	Os-0, Sh-4, Fo-1
Pines	Os-4, Sh-2, Fo-2	Os-4, Sh-1, Fo-1	Os-0, Sh-4, Fo-0
Cedar, Fir, Spruce	Os-4, Sh-0, Fo-1	Os-4, Sh-0, Fo-1	Os-0, Sh-4, Fo-0

Micro Climate

This unit is dry and warm during the growing season. Frost pockets will occur but are uncommon. The duration of the snow pack, and ground frost will depend mainly on the forest type.

Interpretations

Engineering: This unit is well suited for most engineering activities. There is good internal drainage and the water table is below 5 feet. The surface 5 feet are primarily SM and the silt and clay content
decreases with depth. This unit has a good potential for road locations. It has a fair potential for sanitary landfills and sewage disposal facilities, but the design of each will have to include measures to compensate for the porous substratum. There is a good potential for use as a utility corridor.

Fire: This unit will provide some break in the fuel type with adjacent areas. There will be a weakly developed fuel ladder in mature stands. Most fuels will be aerial. The fine fuels will readily dry because of the good air movement through the relative open mature stands. Ground fog is uncommon. Fire lines can be easily constructed with heavy equipment or hand tools. Local concentrations of boulders on the surface will hamper fire line construction. There is no readily available water. In site preparation, spring burns and cool summer burns are acceptable. Hot summer burns are not acceptable due to the need for maintaining the duff, the low level of organic matter, and the low moisture holding capacity in the soil.

Recreation: This unit is adequately suited for most intensive and dispersed recreation facilities. There is some on site borrow for constructing trails. Day light will increase the density of the shrubs and forbs. Grass can be easily maintained in openings. This unit is suitable for sewage disposal facilities.

Timber: The potential productivity is medium for pulpwood and low for sawtimber. The operability is good. Site preparation will be necessary for most reforestations and usually one release will suffice for establishing new conifer stands. Recommended species are aspen, paper birch, pine, and spruce.

Watershed: This unit has a moderately rapid rate of infiltration and permeability. There is no surface runoff or ponding while the soil is free of frost. There is a medium potential for erosion of base soil. This unit is associated with third and fourth order streams; however, streams are uncommon. The natural drainage system is poorly developed. There is some potential for an aquifer.

Wildlife: Diversity is fair and relates to the structure and variety of vegetation. There is a fair potential for managed wildlife openings and this unit is unsuited for developed ponds. Some edge occurs at the contact with adjacent ELT's. Natural streams and ponds are common.

ECOLOGICAL LANDTYPE
Upland Shallow Loamy Dry – USLD - 16

Background

This Ecological Landtype (ELT) is widespread and occurs in all Landtype Associations (LTA) except 9, 10a, and 19. It is most common in the northern half of the forest and along the north shore of Lake Superior. It includes Landtype Phases (LTP) 15, 18, and 63. About 19 percent of the land in the forest is USLD.

Terrain and Geology

This unit occurs on strongly convex and irregular slopes that have a gradient of 6 to 35 percent and is associated with a thin glacial till underlain with bedrock in gently to steeply sloping ground moraines. Landscape positions include ridges and side slopes. The shape of the slopes is strongly influenced by the underlying bedrock, with some slopes having a stair step pattern. Bedrock outcrops occur at ridge tops and at sharp breaks in the side slopes. The local relief ranges up to 200 feet.

Soil

The population of mineral soils has developed in a shallow, (20 to 40 inches thick) well-drained, dark brown and yellowish-brown, sandy loam or loam that are underlain with bedrock. The thickness of the soil often relates to the topography of the underlying bedrock. There is a discontinuous fragipan in some soils that can extend to the bedrock. These soils are classified as Typic Fragiochrepts and Typic Dystrochrepts. Cobbles and boulders occupy about 10 percent of the ground surface. In the soil, the content of gravels and cobbles is about 25 to 35 percent. Bedrock outcrops occupy less than 1 percent of the ground surface.

Water

This unit yields water to lower landscape positions mainly through flowage along the interface of the soil and bedrock. The soils on gently sloping landscape can be saturated for a short time during the spring thaw. There is no permanent water table in the soil. There is no ponding while the soil is free of frost.

Vegetation

This unit supports plant communities that have adapted to the shallow depth to bedrock. Soil moisture might strongly influence the length of the growing season. Common trees are aspen, paper birch, pine, spruce, and fir. Shrubs are typically less than 4.5 feet tall and include hazel, green alder, and blueberries. Forbs are made up of broad leaf plants and some mosses. Grasses are uncommon. Wind thrown trees are common.

The potential structure and density are:

	Mature	Immature	Juvenile
Hardwoods	Os-3, Sh-2, Fo-1	Os-4, Sh-1, Fo-2	Os-0, Sh-4, Fo-1
Pines	Os-3, Sh-2, Fo-1	Os-4, Sh-0, Fo-1	Os-0, Sh-4, Fo-0
Cedar, Fir, Spruce	Os-3, Sh-1, Fo-1	Os-4, Sh-0, Fo-1	Os-4, Sh-4, Fo-0

The density of the overstory decreases as the stands mature and there is an increase in shrub density. Thus, older stands will become open and brushy.

Micro Climate

This unit is dry and warm during the growing season. There are some frost pockets in openings in gently sloping landscape. South facing slopes in steeply sloping landscape are somewhat drier than the north facing slopes.

Interpretations

Engineering: This unit is well suited to those engineering activities that require a minimum of excavation and is poorly suited for those requiring deep cuts and fills. There is good internal drainage, but water saturated soils do occur during the early spring. The surface 40 inches is SM and some ML. This unit has a fair potential for road location. It is unsuited for sanitary landfills and sewage disposal systems that discharge an effluent into the soil. There is a poor potential for use as a utility corridor.

Fire: This unit provides some break in fuel types with adjacent areas. There is an incomplete fuel ladder in mature stands and most fuels are aerial. The fuels tend to dry readily in this unit due to the low water storage capacity in the soil and the movement of air through the open stands. Ground fog is uncommon in the unit. Fire lines can be readily constructed with heavy equipment and explosives, but the cobbles and boulders will limit the effectiveness of hand tools. For site preparation, spring burns are acceptable. However, summer burns should be only used when the fire ground fuels are moist; otherwise, long-term adverse impacts will result in a reduction in potential productivity.

Recreation: This unit is poorly suited for intensive developments but can support most facilities for dispersed recreation. There is a limited on site borrow for trail construction. Day light will increase the density of the shrubs and forbs. Sewage disposal systems are limited to those that do not discharge an effluent into the soil.

Timber: This unit has a medium potential productivity for pulpwood and a low potential for sawtimber. The operability is fair. Plant competition is low in managed stands. Some site preparation will be necessary for establishing new conifer stands. Ordinarily, one release will suffice. Recommended species are aspen, pine, spruce, and fir.

Watershed: This unit had moderate infiltration and permeability rates. Deep percolation does not occur and the storage capacity is low due to the shallow depth to bedrock. Thus, the water levels in streams and lakes will vary considerably and the stream levels will tend to be “flashy” in nature. Perennial streams commonly flow in bedrock-controlled valleys, are third, and fourth order. Lakes often have bedrock shorelines. There is no potential for an aquifer.

Wildlife: The diversity is medium and is related to the variety in the plant communities, their structure, and the inclusions of contrasting sites. This unit has a fair potential for managed wildlife openings and is unsuited for wildlife ponds.