Red Dog Mine
Closure and Reclamation Plan

SD C8: Preliminary Spillway Design, Red Dog Tailings
Main Dam, Ultimate Closure Configuration
(URS, 2008)

REPORT
PRELIMINARY SPILLWAY DESIGN
RED DOG TAILINGS MAIN DAM
ULTIMATE CLOSURE
CONFIGURATION
RED DOG MINE, ALASKA

FOR

TECK COMINCO ALASKA, INC.
URS JOB NO. 33760401
November 14, 2008

November 14, 2008

Mr. Jim Swendseid, P.E.
Teck Cominco Alaska, Inc.
3105 Lakeshore Drive, Blvd. A, Suite 101
Anchorage, Alaska 99517

> **Preliminary Spillway Design**
> **Red Dog Tailings Main Dam**
> **Ultimate Closure Configuration**
> **Red Dog Mine, Alaska**
> **PO 1257477, Contract RD-02-06**
> **URS Job No. 33760401**

Dear Mr. Swendseid:

URS Corporation is pleased to submit three copies of this report to Teck Cominco Alaska, Inc. (TCAK) on the preliminary spillway design for the Red Dog tailings main dam at the ultimate closure configuration. The work was completed in accordance with Contract No. RD-02-06, Purchase Order 1285941-SVC dated October 17, 2007, and amendments to the purchase order during 2008.

This preliminary spillway design report replaces the conceptual spillway design part of a conceptual design report for future raises to closure that was completed in 2007 in support of a mine closure plan. The preliminary design assumes that the mine will operate until 2031, and that the dam will be raised to a height needed store all anticipated tailings and meet the post closure water cover and freeboard needs.

We thank you for the opportunity to continue working on the tailings main dam and to prepare the preliminary spillway design. Please call if you have any questions or need additional information.

> Sincerely,
> URS Corporation

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EXECUTIVE SUMMARY

The tailings main dam at Red Dog Mine is a 182-foot high rock fill embankment that was raised in stages to the current Stage VII-B crest at elevation 960 feet (El. 960). The mine operator, Teck Cominco Alaska, Inc. (TCAK), and SRK Consulting Inc. are developing a mine closure plan that is based on operations to the year 2031. This will require raising the dam from its current crest to its ultimate closure configuration.

In order to provide technical input to the closure plan for the tailings facility, URS Corporation is completing a preliminary design of the ultimate closure configuration of the tailings main dam. As part of the preliminary dam design, URS completed a preliminary design of a spillway to prevent overtopping of the dam during extreme precipitation events.

The required surcharge capacity of the tailings impoundment was determined so that the dam could be sized to a height sufficient to prevent the discharge of water from the impoundment to Red Dog Creek. A spillway was then sized and designed to a preliminary level of completion as an additional fail safe measure to prevent overtopping of the dam in case of very extreme precipitation events.

The required spillway capacity was determined so that it could route an inflow design flood equivalent to one half of the probable maximum flood (PMF) with adequate freeboard without overtopping the dam. The spillway was sized to pass a full PMF in an extreme event. However, a parapet berm will be required along the upstream edge of the dam crest to protect the dam against wave overtopping during the PMF.

The components of the surcharge capacity and the spillway capacity for an inflow design flood and corresponding spillway and dam crest elevations are tabulated below:

	Water surfaces	Incremental height (ft)	Design elevation (ft)
Design Criteria	Top of Tailings	-	975.0
	Water Cover	2.00	977.0
Surcharge Capacity	Spring Freshet	1.35	978.4
	PMF Series	4.25	982.6
	100-year Flood	0.92	983.5
	Preliminary Spillway Crest	-	**983.5**
Spillway Capacity	Inflow Design Flood	1.42	984.9
	Freeboard	1.12	986.0
	Preliminary Dam Crest	-	**986.0**

A preliminary spillway design was developed for an open-channel, side-hill conveyance aligned along the west hillside of the tailings impoundment and outside of the west abutment of the tailings main dam so it will discharge downstream of the seepage collection system. The spillway will not be located within the embankment of the tailings main dam.

Future spillway options were considered for the long-term post-closure possibility of water in the tailings impoundment achieving a sufficiently "clean pond" water quality status that complies with state and federal discharge requirements to discharge from the impoundment to Red Dog Creek.

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1.0 INTRODUCTION

The tailings main dam at Red Dog Mine is a 182-foot high rock fill embankment that was raised in stages to the current Stage VII-B crest at elevation 960 feet (El. 960). The mine operator, Teck Cominco Alaska, Inc. (TCAK), and SRK Consulting Inc. are developing a mine closure plan that is based on operations to the year 2031. This will require raising the dam from its current crest to its ultimate closure configuration.

In order to provide technical input to the closure plan for the tailings facility, URS Corporation is completing a preliminary design of the ultimate closure configuration of the tailings main dam. This work is being performed in accordance with Contract No. RD-02-06, Purchase Order 1285941-SVC dated October 17, 2007, Amendments 1 dated April 20, 2008, and Amendment 2 which is pending.

As part of the preliminary dam design, URS completed a preliminary design of a spillway to prevent overtopping of the dam during extreme precipitation events. This design required the completion of hydrologic and hydraulic analyses so the dam could be sized to provide the tailings impoundment with adequate surcharge capacity to prevent the discharge of water from the impoundment to Red Dog Creek.

This preliminary spillway design report supersedes the conceptual-level spillway design described in the tailings main dam conceptual design report for future raises to closure (URS 2007a). The preliminary spillway design builds off the conceptual spillway design by using updated mine site climate and stream data to determine the surcharge and spillway capacities required to achieve "zero-discharge" at closure.

For the preliminary dam design, URS prepared 19 drawings (TCP-1 to TCP-19). Eight of these drawings are relevant to the spillway and are included in this preliminary spillway design report. These drawings are TCP-1 and TCP-2 which provide general information, TCP-5 which is a plan of the proposed dam and spillway at closure, and TCP-15 to TCP-19 which show spillway plan views, profiles and details.

2.0 PURPOSE AND SCOPE

2.1   PURPOSE

The purpose of the preliminary spillway design for the Red Dog tailings main dam at the ultimate closure configuration is to ensure that the dam and spillway are constructed to achieve the "zero discharge" closure requirement and to assure long term safety and stability of the dam.

A critical assumption underlying the preliminary spillway design is that the selected alternative for the closure of the tailings impoundment is the "Clean Pond" option, which is described in the Red Dog Mine Closure and Reclamation Plan (SRK, 2007a) as having the following objectives:

- *Covering the tailings with water to restrict oxidation and acid generation;*
- *Managing contaminated water to keep the pond as clean as possible*
- *Ensuring long-term stability of the dams, while minimizing any seepage; and*
- *Reclaiming surface disturbances.*

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SRK (2007a) states "*A permanent pond will be maintained over the tailings. Through many years of experience at other sites, it has been demonstrated that the most effective way to prevent oxidation and acid generation from sulfidic tailings is to keep them underwater*". The preliminary spillway design assumes that the post-closure period will continue into perpetuity, and that it will consist of "active" and "passive" phases of "Clean Pond" water management as follows:

- "Active" water management will be similar to the current operations water treatment and discharge practice, and will start immediately upon closure with the treatment of "cleanish" water and the discharge of this treated water to Red Dog Creek.

- "Passive" water management could be initiated at some time in the future if the "cleanish" water achieves a water quality that meets state and federal discharge criteria limits and can be discharged without treatment to Red Dog Creek.

This preliminary design report is focused on the "active" water management where the "cleanish" water in the tailings impoundment needs to be treated before it is discharged. However Section 8.0 of the report addresses future spillway considerations that could be implemented in the event that "passive" water management is able to be implemented.

The preliminary spillway design also complies with comments made by State of Alaska representatives on the conceptual spillway design at a Large Mines Project Team (LMPT) workshop in Anchorage on December 18 and 19, 2007, and a Failure Modes and Effects Analysis (FMEA) workshop in Vancouver on March 25 and 26, 2008. The comments were earlier expressed in an interoffice memo from the State Dam Safety Engineer to the LMPT Board on November 15, 2007, as follows:

> "*From a "fatal flaw" perspective, Dam Safety believes that the final, relative configuration of the dam crest, spillway, and tailings elevations are fundamentally incorrect for a "clean pond" alternative. As presented, the design assumes water treatment for a "very long time" and the spillway is designed for emergency situation only. As designed, the invert of the emergency spillway is only 1.5 feet below the crest of the dam, and the operational and flood surge stages are accounted for below the spillway invert. For a clean pond, the freeboard would be measured above the base flow through an operational spillway, resulting in the dam crest elevation several feet above the spillway invert. If a clean pond is ever actually achieved, a substantial modification of the dam will be required in order to safely operate the dam passively, e.g., without active pumping. Due to the limited space available at the Main Dam site, Dam Safety recommends that passive operation contingency is accounted for in the proposed final configuration of both dams at the Red Dog Mine Tailings Storage Facility.*"

For the purposes of the preliminary spillway design, the tailings impoundment hydrology and the spillway hydraulics were re-evaluated to update the conceptual design, and provide a preliminary spillway design as part of the preliminary design of the tailings main dam ultimate closure configuration.

2.2 SCOPE

This preliminary spillway design required that hydrologic and hydraulic analyses be completed to provide adequate surcharge capacity in the tailings impoundment, and sufficient spillway capacity to route an inflow design flood (IDF). The actual preliminary design includes the location and details of the spillway channel, and future spillway options for possible "clean pond" conditions.

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The scope of work to prepare the preliminary spillway design consisted of the following technical tasks: :

- Confirm project description
- Develop technical approach
- Estimate surcharge (storage) capacity
- Estimate spillway capacity
- Develop preliminary spillway design
- Consider future spillway options
- Prepare preliminary design report

Initial drafts of the technical approach, surcharge capacity and spillway capacity sections of the report were submitted to TCAK and SRK during May and June 2008 for review. The purpose of the reviews was to ensure that the assumptions were compatible with those used in the closure plan being finalized by SRK, and that the findings integrated well with the closure plan.

A preliminary spillway design review meeting was held by means of a conference call on June 5, 2008. It was attended by Gary Coulter of TCAK; Daryl Hockley and Kathleen Willman of SRK; and Cecil Urlich, Anand Prakash, Rod DenHerder and Tobey Clarkin of URS. On the basis of this meeting, this draft preliminary spillway design report was completed and submitted to TCAK and SRK for review.

The preliminary spillway design report will be finalized following receipt of comments from TCAK and SRK on the draft report. It is understood that the preliminary spillway design report will be an appendix to the closure plan that is being finalized by SRK.

3.0 PROJECT DESCRIPTION

3.1   SITE DESCRIPTION

The Red Dog Mine is a zinc and lead mine in northwestern Alaska. The mine is located in the Northwest Arctic Borough of Alaska near the southwestern end of the DeLong Mountains of the western Brooks Range. It is situated approximately 90 miles north of the Arctic Circle, 82 miles north of Kotzebue, and 47 miles inland from the Chukchi Sea. The main components of the mine consist of the following:

- Open pit mine for the extraction of metal bearing ore
- Mill and concentrate facility
- Tailings impoundment that is contained by the tailings main dam

The tailings main dam is located at GPS Coordinates Lat. N. 68° 04' 10.170" Long. W. 162° 51' 44.202". The dam is on the South Fork of Red Dog Creek immediately south-south-west of the mill facility which is west of the open pit. These components of the dam can be seen in Figure 5-1.

The area around the tailings impoundment that will contributes water to the impoundment for purposes of the preliminary spillway design for the tailings main dam at the ultimate closure configuration consists of three distinct sub-areas of the mine:

- Main waste stockpile east of the tailings impoundment

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- Tailings impoundment which is mostly covered with water
- Artic tundra natural ground west of the tailings impoundment

These three areas constitute the total catchment area of 2.52 square miles that contributes surface runoff entering the tailings impoundment. The spillway will be designed to meet the "zero-discharge" intent and the standard dam design practices as described in the following sections.

3.2 "ZERO-DISCHARGE" INTENT

The preliminary spillway design is based on a "zero discharge" intent that precludes discharge from the tailings pond into Red Dog Creek and the surrounding environment. The post-closure operations intent is that the current pumping, treatment, and discharge operations are to continue until such time as a "clean pond" is achieved.

It is recognized that an absolutely "zero discharge" impoundment, i.e., with zero discharge under all circumstances and for all conceivable climatic conditions in the future may not be feasible or practical. The best alternative is to minimize the frequency and magnitude of potential discharges from the tailings pond as much as practicable.

3.3 RELEVANT REGULATORY DESIGN PRACTICES

The standard design practices for tailings impoundments and spillways relevant to this preliminary design require accommodation of an IDF and associated freeboard. Relevant guidelines of The State of Alaska Department of Natural Resources (ADNR), U.S. Federal Emergency Management Agency (FEMA), U.S. Natural Resource Conservation Service (NRCS) and U.S. Nuclear Regulatory Commission (NRC) for the design of dams, tailings dams and spillways are summarized in the following subsections.

3.3.1 Inflow Design Flood and Storage Capacity

A document titled "*Guidelines for Cooperation with the Alaska Dam Safety Program*" (ADNR, 2005) was prepared by the Dam Safety and Construction Unit, Water Resources Section, Division of Mining, Land and Water, Alaska Department of Natural Resources. The guidelines were based on the Alaska dam safety regulations articulated under Article 3 of Alaska Administrative Code 11 AAC 93.

These guidelines are intended to establish a consistent basis for communication between the ADNR, dam owners and operators, and various other entities involved in the planning, design, construction, operation, and regulation of dams in Alaska. These guidelines specify the IDF as the primary objective of the hydrological portion of the design. The IDF is further defined in 11 AAC 93.195(c) as:

> "*the flood flow above which the incremental increase in the downstream flood caused by a failure of the dam does not result in any additional danger downstream.*"

The guidelines specify the maximum possible IDF as the following:

> "**Maximum standard for all hazard potential class dams** – *IDF based on probable maximum flood (PMF) based on probable maximum precipitation (PMP).*"

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The ADNR guidelines also reference the FEMA "*Federal Guidelines for Dam Safety: "Selecting and Accommodating Inflow Design Floods for Dams*" (2004) as guidance for the IDF selection. The FEMA guidelines state:

> "*The PMF is the upper limit of floods to be considered when selecting the appropriate IDF for a dam.*"

NRCS Technical Release 60, "Earth Dams and Reservoirs" (2005) stipulates that an auxiliary spillway on a "significant" dam should be able to handle an IDF from the routing of $P_{100} + 0.12(PMP - P_{100})$ and the IDF from the routing of $P_{100} + 0.26(PMP - P_{100})$ on a "high hazard" dam, where P_{100} is the precipitation for 100-year return period event.

To achieve a "zero-discharge" condition, and store the IDF rather than pass it through an emergency spillway, the NRC Regulatory Guide 3.11 "*Design, Construction, and Inspection of Embankment Retention Systems for Uranium Mills*" (NRC, 1977) requires storage of the average annual runoff, PMF series and a 100-year flood as described below:

> "*Either the surcharge capacity of the retention system should be sufficient to store runoffs over its service life or there should be an emergency discharge capacity capable of passing the probable maximum flood. The emergency discharge capacity may be obtained by constructing a spillway or by other means. The surcharge capacity should be adequate to store a probable maximum flood series preceded or followed by a 100-year flood, assuming a pool elevation equivalent to the average annual runoff. Probable maximum flood series as used herein comprises two floods: the Probable Maximum Flood and the flood equivalent to about 40% of the PMF and about 3 to 5 days prior to the occurrence of the main flood.*"

3.3.2   Freeboard Standard Design Practices

Freeboard provides a margin of safety against a possible overtopping failure of dams and is provided below the dam crest and above the maximum anticipated water surface elevation during IDF routing through the reservoir. The ADNR guidelines reference the FEMA "*Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams*" (2004) in determining freeboard standard design practices. These FEMA guidelines define minimum freeboard guidelines as the following:

> "*For **minimum** freeboard combinations, the following components, when they can reasonably occur simultaneously, should be added to determine the total minimum freeboard requirement:*
>
> - *Wind-generated wave runup and setup for a wind appropriate for the maximum reservoir stage for the IDF.*
>
> - *Effects of possible malfunction of the spillway and/or outlet works during routing of the IDF.*
>
> - *Settlement of embankment and foundation not included in crest camber.*
>
> - *Landslide-generated waves and/or displacement of reservoir volume (only cases where landslides are triggered by the occurrence of higher water elevations and intense precipitation associated with the occurrence of the IDF).*

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The NRCS requires routing the freeboard hydrographs resulting from $P_{100} + 0.40(PMP - P_{100})$ and a PMP to set the minimum dam crest elevations for a "significant" and "high hazard" class dam, respectively. NRC Regulatory Guide 3.11 "*Design, Construction, and Inspection of Embankment Retention Systems for Uranium Mills*" (NRC, 1977) specifies the following freeboard requirements:

> "*Freeboard should be sufficient at all times to prevent overtopping by wind-generated waves and should include an allowance for settlement of the foundation and dam. Adequate slope protection should be provided for the embankment against wind and water erosion, weathering, and ice damage.*"

3.3.3 Adopted Standard Design Practices

URS reviewed regulatory guidelines and technical literature on standard design practices for tailings impoundments and spillways. The NRC Regulatory Guide 3.11 (NRC, 1977) provided the most comprehensive and conservative design approach for tailings impoundment spillways. The NRC standards either met or exceeded other relevant standards and provided the only surcharge specific guidelines for a "zero-discharge" tailings pond based on water storage.

For these reasons, URS used the NRC Regulatory Guide 3.11 standards for the preliminary spillway design at the ultimate closure configuration of the tailings main dam, and maintain the "zero-discharge" objective for the tailings impoundment.

4.0 PROJECT APPROACH

4.1   DESIGN BASIS FOR ZERO DISCHARGE

URS has evaluated various approaches for determining storage and spillway capacities for the closed, "zero-discharge" tailings impoundment configuration that has been selected in the mine closure and reclamation plan (SRK, 2007a). The surcharge capacity and spillway capacity required to design a "zero discharge" system are estimated using conservative assumptions. The rationale and conservatism of the selected design are described in the following paragraphs.

The current pumping and treatment system at the mine is designed to pump out any surface runoff as it accumulates in the tailings impoundment, treat the pumped out water, and discharge it to Red Dog Creek in compliance with State regulations. This system will be maintained until when the tailings impoundment attains a "clean pond" status. Thus, normally the water surface will be maintained at El. 977.0, which includes tailings deposited up to El. 975.0 and a water cover of two feet above the tailings.

Under normal conditions, the pumping, treatment, and discharge system will operate so as to avoid significant accumulation of water above El 977.0. This means that the net average addition of water into the pond from year to year will be effectively reduced to almost zero.

For a conservative estimation of the surcharge capacity for the tailings pond, it is assumed that the pumping, treatment, and discharge system breaks down in the critical "freshet" month of May. Thus, the average spring freshet from the snowfall and rainfall during the months of November to April and the runoff from the month of May will be stored within the surcharge capacity of the tailings pond. Usually, the system remains idle during the winter months of November to April.

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In addition, it is hypothesized that a PMP event also occurs in the same month of May. Although the persistence of a long-duration PMP on a relatively small catchment of 2.52 square miles is a very low-probability event, it is conservatively assumed that the PMP has a relatively long duration of 24 hours. Further, it is hypothesized that the PMP event is preceded or succeeded by a 40% PMP and also a 100-year precipitation event of the same relatively long duration of 24 hours in the same month. The total storage capacity required to store the runoff resulting from these successive extreme events is equivalent to the storage capacity specified by the NRC to achieve zero discharge conditions from uranium mill tailings impoundments (NRC, 1977).

A portion of the precipitation depths during the above successive events will be lost in interception, evapotranspiration, infiltration, and depression storage in the watershed before reaching the tailings impoundment as surface runoff. To be conservative, it is assumed that these losses will be as small as on impervious areas and water surfaces.

4.2   DESIGN BASIS FOR EMERGENCY SPILLWAY

NRC Regulatory Guide 3.11 (NRC, 1977) specifies the use of either the surcharge capacity or an emergency discharge capacity such as a spillway, but not both. The provision of the surcharge capacity implies that it provides adequate storage capacity to achieve a "zero-discharge" condition and so a spillway is not necessary. However, to add further redundancy to the system an emergency spillway capable to pass the PMF is included in the design. The spillway is designed to pass one-half a PMF with adequate freeboard for coincident wind wave activity. The spillway can also pass a full PMF without the maximum water surface elevation rising above the dam crest

Since an emergency spillway is provided in addition to the surcharge capacity, the provision of additional freeboard capacity for wind wave activity below the spillway crest is not considered necessary. So far as safety against dam overtopping is concerned, a space of 2.5 feet will be available between the dam crest at El. 986.0 and top of surcharge capacity at El. 983.5. Potential for wind wave splashes over the spillway crest at El. 983.5 is a very low probability and very short duration phenomenon.

The provision of permanent storage capacity for such a temporal activity over and above the conservatively estimated surcharge capacity appears to be overly conservative. In fact, the purpose of the emergency spillway is to discharge such occasional overflows such as wind wave splashes and rare coincidence of a PMP or less intense precipitation events with the combined event of a PMF series plus 100-year flood. As described in Subsection 6.3.7, a boom can be provided at the spillway entrance to minimize overflows through the spillway due to wind-induced wave splashes.

It may be emphasized that the emergency spillway is provided to add a degree of redundancy and conservatism and is not expected to discharge any water from the impoundment except in extreme events of very low probability. In the unlikely event of any outflow through the spillway, only the top layer of water above the spillway crest (El. 983.5) would be discharged through the spillway. This top layer of water will be separated from the tailings by about an 8.5 feet (El. 983.5 – El. 975.0) thick water cover and should be relatively clean. Thus, even under this almost improbable situation, there may be no significant impact on the water quality of Red Dog Creek.

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4.3   DESIGN FREEBOARD ESTIMATION

Freeboard against wind wave activity and potential dam overtopping is provided above the maximum water surface elevation estimated for the case when a one half PMF is routed through the spillway.

The adopted design freeboard includes wind setup and wave runup on the slope of the beach, which is assumed to be 1 vertical to 100 horizontal (1:100) due to wind wave activity coincident with a one half PMF flow through the emergency spillway. The wind setup and wave runup are computed using the guidelines included in Subsection 3.3.2.

The emergency spillway may become operative only in the unlikely situation when the entire surcharge capacity has been filled up due to a highly unlikely sequence of extreme events, such as a PMF series plus 100-year flood, and a pump breakdown in conjunction with spring freshet in the critical month of May. The probability of an additional full PMF discharge through the spillway concurrent with the above extraordinary conditions is extremely low.

Therefore, the dam crest was set to pass the full PMF without the water surface rising above it. A parapet berm will be required at the upstream edge of the dam crest as protection against wave overtopping during the unlikely event of a full PMF. The parapet berm would be built of coarse rockfill or riprap, and is considered to be adequate for this purpose and avoids the need to build a higher dam that is not needed.

4.4   ANALYSIS SEQUENCE

The hydrologic and hydraulic analyses associated with the preliminary spillway design for the Red Dog Tailings Pond are conducted in a sequence of four steps as follows:

- **Pond Surcharge Capacity** – The pond surcharge capacity starts at the estimated tailings surface level of El. 975 at closure plus a two-foot water cover to prevent acid generation (El. 977.0) (SRK, 2007a). The surcharge capacity includes capacity for the surface runoff resulting from average spring freshet and May precipitation, a PMF series and a 100-year flood in the catchment area of the tailings pond.

- **Spillway Discharge Capacity -** The spillway discharge capacity will be above the surcharge capacity, and will start at the spillway crest. It will consist of the height produced by routing an IDF of a 0.5 PMF through the spillway and the freeboard required to contain wind setup and wave runup associated with coincident wind speeds. The spillway must pass the PMF without overtopping the dam, and freeboard during the PMF is required.

- **Preliminary Spillway Design -** The spillway will be located along the west hillside above the dam and not in the dam embankment. The design is determined on the basis of the site hydrology and hydraulics, and soil and rock conditions along the alignment. The design includes the spillway entrance, broad-crested spillway, outlet channel, erosion protection and stilling basin for long-term operability and creek protection.

- **Future Spillway Options -** Future spillway options can be considered for the possible future condition when the tailings pond reaches a "clean pond" status and meets requirements to discharge from the tailings impoundment into Red Dog Creek and the environment.

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The analyses that were completed for these four steps, and the results obtained from the analyses, and the design criteria that were developed, are described in the following sections of this report.

5.0 POND SURCHARGE CAPACITY

5.1   CAPACITY COMPONENTS

This section describes the surcharge capacity to be provided in the Red Dog tailings impoundment below the crest of the spillway for the ultimate closure configuration of the tailings main dam. The surcharge capacity includes the following four components:

- Water Cover
- Spring freshet
- Probable maximum flood series
- 100-year flood

The assumptions for design criteria (tailings level/water cover) and calculations of the surcharge components above are detailed in the following subsections.

5.2   WATER COVER

According to the current mine plan (SRK, 2007a), it is estimated that 88,000,000 tons (79,800,000 tonnes) of tailings will be produced by the end of ore processing in 2031. Using the lowest tailings density estimate of 94.3 pounds per cubic feet (pcf), these tailings will result in a final struck-level tailings surface of approximately El. 975.0.

To prevent the oxidation of the underlying tailings, a minimum water cover of two feet will be maintained over the tailings surface in perpetuity after closure. This will raise the pool elevation to El. 977.0.

5.3   SPRING FRESHET

5.3.1   Introduction

The pumping, treatment and discharge from the tailings impoundment after closure is designed to remove runoff entering the tailings pond to maintain the pond surface at El. 977.0 (SRK, 2007a).

Established dam operations practices and FEMA guidelines recommend the consideration of possible spillway and outlet works malfunctions when determining outflows and storage for sizing spillways. URS assumed that the water treatment plant (WTP) will break down for one month. May was conservatively chosen as that month because it is the largest runoff month. The surface runoff in May includes the spring freshet which is precipitation accumulated from November to April, and the precipitation in May itself.

The tailings impoundment is typically frozen between November and April, and no pumping out of the impoundment occurs during this time. The spring freshet is assumed to be pumped out along with the May runoff. The WTP is assumed to return to normal operations in June and will maintain the pond level at or below the level produced from the spring freshet and May runoff.

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The 2007 national pollutant discharge elimination system (NPDES) permit for Red Dog Mine allows a cumulative flow into the Red Dog Creek of 2,418 billion gallons per year (SRK, 2007a), which will not inhibit the pumping rates required to maintain the tailings pond at or below the level produced from the spring freshet and May runoff. The required monthly discharge rates to maintain the tailings pond below the level of spring freshet and May runoff will be well below the historic monthly discharges from the WTP into Red Dog Creek.

Since the discharge from the tailings impoundment will cease to be contaminated any further after closure of mine operations, it is assumed that comparable discharge rates will continue to be permitted and that the mine will be able to maintain the water level in the tailings impoundment below the level required for the spring freshet and the May runoff.

5.3.2   Spring Freshet and Monthly Precipitation

The average monthly precipitation values for the months of May and June used in this calculation are taken from the "*Red Dog Mine Closure and Reclamation Plan, Supporting Document E1, Water and Load Balance Spreadsheet*"(SRK, 2007b).

The average monthly precipitation values were calculated by multiplying the precipitation in the months of October to April by a factor of 1.4 to account for underestimated precipitation due to snow measurement methods. The 1.4 factor was chosen because it reportedly produces annual precipitation values consistent with the average annual runoff depths measured at Red Dog Mine Gage Station 140 (TCAK, 2008b).

Precipitation values were calculated assuming spring freshet release in May. By using the precipitation data from 1992 to 2007, the average May (with spring freshet) and June precipitation were calculated to be 6.6 and 1.5 inches respectively (Appendix A-2).

5.3.3   Catchment Area Contributing to Pond

The catchment area contributing to the tailings impoundment at final closure was calculated to be 2.52 square miles as a result of a water and load balance analysis by SRK (SRK, 2007b) and is shown in Figure 5-1. The catchment area for the tailings main dam was verified using the ArcGIS Spatial Analyst Watershed Delineation Tool (ESRI, 2006).

URS verified the catchment areas by using a mine site topographic map dated July 2, 2007 (Aero-Metric Anchorage, 2007). The topographic map projection used was "ALMOST" Alaska State Plane, Zone 7, North America Datum (NAD) 27 expressed in U.S. Survey feet, with vertical referenced to mean sea level (MSL). The map is based on photography taken on July 2, 2007 and produced at a scale of 1-inch to 200-feet and contour interval of 5 feet.

URS converted the map contours to a raster image file, mosaiced them, and analyzed them on a 5-foot by 5-foot cell size to determine flow directions and catchment areas. The URS calculation of the total catchment area was within 5 percent of the SRK calculation of 2.52 square miles. So the SRK number was used for consistency with the closure plan report.

In calculating the catchment areas, it was assumed that the diversion ditches in place on the artic tundra portion of the mine site (SRK, 2007a) will eventually fail after closure. Therefore, the catchments that are currently diverted away from the pond were included in the overall catchment at final closure. It was also

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assumed that seepage diversion of the main waste stockpile runoff from the pond to the Aqqaluk Pit would fail sometime after closure.

5.3.4 Catchment Curve Number

Curve number (CN) is an empirically derived value which is dependent on location, soil-type, land use, and antecedent moisture conditions. The CN provides a measure of how much of the precipitation falling on the catchment will end up as runoff. A higher CN means that a higher proportion of rainfall that falls on a catchment area becomes runoff as opposed to being absorbed into the ground.

Theoretically, the CN varies between 0 and 100. A CN of 100 represents that all the precipitation that falls on the catchment area becomes runoff. The catchment CN was conservatively estimated in this design using a CN of 98 as specified in NRCS guidance on hydrologic soil-cover complexes: *"Impervious and water surfaces, which are not listed, are always assigned a CN of 98"* (NRCS, 2004b).

5.3.5 Initial Abstraction

Initial abstraction (Ia) is the measure of the rainfall which falls prior to the beginning of runoff. It includes infiltration into the subsurface and surface storage in the watershed such as lakes and low areas. Initial abstraction is inversely related to the CN. If the CN is assumed to be 98, as is the assumed case for the pond catchment, the initial abstraction is 0.04. The initial abstraction for the mine site was calculated using the following formula:

$$Ia = 0.2S, \text{ where S is maximum retention}$$

$$S = \frac{1000}{CN} - 10 = \frac{1000}{98} - 10 = .2$$

$$I_a = 0.2S = 0.2 \times 0.2 = 0.04$$

5.3.6 Stage-Storage-Table

The stage-storage table for the tailings impoundment was constructed from contours based off the Red Dog Mine Topographic Map (Aero-Metric Anchorage, 2007). One-foot contours were interpolated from the 5-foot contours in ArcGIS to create surface areas at 1-foot intervals. The volume capacity at each stage was then determined by multiplying the average area of the preceding and current contour stage surface areas by the 1-foot depth between the stages (Appendices A-4, 5).

Linear interpolation was used to estimate the elevations of storage capacities in between the 1-foot contour intervals. The surface areas measured at each 1-foot interval were confined by the specific contour for that interval, a 600-foot tailings beach that is planned alongside the tailings main dam and a 600-foot wide tailings beach that is assumed for conservatism to be alongside the tailings back dam.

It should be noted that according to the Red Dog Mine Closure and Reclamation Plan (SRK, 2007a), it is unsure whether a beach will be built alongside the back dam as part of its ultimate closure configuration. However, discussions with the back dam designers confirm that no beach is planned at closure (Anderson, 2008). However, a back dam beach was assumed in this stage-storage analysis for conservatism. Conversely, a back dam beach was not assumed in the fetch analysis for calculating wave heights, also for conservatism.

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5.3.7 Spring Freshet and Monthly Runoff

The average monthly runoff into the tailings impoundment for May, including the spring freshet, was determined from the total average May runoff minus the average volume of evaporation and seepage that leaves the impoundment during May.

This volume was found to be equivalent to a pond water depth of 1.35 feet above the tailings impoundment water surface level of El. 977.0 feet. This would raise the pond stage water level to El. 978.35 (Appendix A-6).

5.3.8 WTP2 Monthly Discharge Capacity

The monthly discharge capacity of Water Treatment Plant No. 2 (WTP2) at closure was assumed to be a portion of the predicted annual treatment capacity according to the ratio of average June discharge to the average annual discharge for WTP2 for the years of 1999 to 2007, as detailed in the water and load balance spreadsheet located in Appendix A-7 (SRK, 2007a; Weakley, 2008). The ratio of June discharge to annual discharge was determined to be 0.29.

The expected annual discharge of WTP2 from the tailings impoundment at closure is 471 million gallons (Mgal) from the water and load balance spreadsheet (SRK, 2007b). The discharge capacity of WTP2 greatly exceeds that required to handle the average June precipitation. Assuming that the WTP2 discharge capacity is equal to the monthly ratios times annual discharge, it is determined that WTP2 has the capacity to maintain the water surface in the impoundment below the water storage level for the combined average May runoff and spring freshet (Appendices A-7-10).

5.4 PROBABLE MAXIMUM FLOOD SERIES

5.4.1 Introduction

The PMF series was calculated by estimating a PMP event from U.S. Weather Bureau data and local precipitation data at Red Dog Mine and Kotzebue, AK. A catchment area of 2.52 square miles and a CN of 98 were used to determine the runoff from the PMP to enter the pond as the PMF. The PMF was then multiplied by 40% to obtain the PMF series.

5.4.2 Probable Maximum Precipitation Duration Selection

The occurrence of a sustained PMP for a relatively long duration of time, such as greater than 6 hours, over a relatively small drainage area of 2.52 square miles is a very low probability event. Therefore, to be conservative, a PMP of 24-hour duration was adopted.

5.4.3 Statistical Analysis of Precipitation Data

Long-term precipitation data are available at the Weather Bureau gage at Kotzebue Airport (Latitude 66^0 52' N, Longitude 162^0 38'W, El. 10) (National Climatic Data Center, 2008). Also, daily precipitation values are available for 16 complete years (1992 to 2007) for the Red Dog Mine gage, except for missing values for January 1997 and January 1998 (SRK, 2007b; Teck Cominco Alaska, 2008). January is a very low precipitation month, so the missing data is not expected to affect the annual maximum of daily values used in the PMP analysis. In addition, daily precipitation values are available for six months (July to December) in 1991, but were not used in this analysis.

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There is usually a difference in the observed daily (observational-day) precipitation and precipitation measured during a sequential period of 24 hours (or 1440-minutes) containing the maximum amounts. Based on the reported average ratio between the two values, the 24-hour precipitation depth was assumed to be 1.13 times the corresponding daily value at each of the two stations (Weather Bureau, 1961).

Statistical parameters have been estimated for the maximum daily values for the precipitation data at Kotzebue and Red Dog Mine for the same time period of 16 years (1992 to 2007). The estimated statistical parameters are included in Table 5-1.

Table 5-1 Statistical Parameters of Daily Precipitation Data (1992-2007)

Parameter	Precipitation depth (in)		Relevant ratio
	Red Dog Mine Gage	Kotzebue Gage	Red Dog Mine / Kotzebue
No. of years of record (n)	16	16	N.A.
Mean of daily maximum values in each year	1.28	0.74	1.7
Standard deviation using n-1as divisor	0.5306	0.1975	N.A.
Standard deviation using n as divisor	0.5137	0.1912	N.A.
Coefficient of skewness	1.265	1.524	N.A.

N.A. = Not applicable

5.4.4 Probable Maximum Precipitation Estimation

The PMP is defined by the National Oceanic & Atmospheric Administration (NOAA) as the theoretically greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographic location at a certain time of the year (NOAA, 1994).

One of the most commonly used methods to estimate PMP for a given drainage basin is to follow the procedures and charts in the NOAA Hydrometeorological Report or Technical Paper for the region where the basin is located (National Research Council, 1985; Bureau of Reclamation, 1987; Ponce, 1994). These procedures and charts reduce the need for detailed site-specific analyses for storm transposition, moisture maximization and orographic effects for each site.

The U.S. Weather Bureau document that is applicable to the Red Dog mine is Technical Paper No. 47 (TP-47) (Weather Bureau, 1963). TP-47 includes a generalized chart (Figure 2-12) for 24-hour PMP in Alaska. The 24-hour PMP for the Red Dog Mine site interpolated from this chart is approximately 5.9 inches. However, as discussed below, it is prudent to adjust this value to properly fit the site conditions.

The gage density that was used to develop the charts in TP- 47 was one gage per 3,200 square miles. The average density in the United States is one gage per 250 square miles. In particular, the gage density north of the 65^0 latitude in Alaska was 8,300 square miles per gage. This unusually sparse gage density, relatively short periods of records, rugged topography, and extreme arctic climatic regimes made it difficult to make an accurate estimation of the PMP for this region.

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Consequently, the results in TP-47 are deemed to have a lower degree of accuracy than those in NOAA reports for other parts of the United States. In particular, the isopluvials in Figure 2-12 of TP-47 seem to indicate a lower PMP at Red Dog Mine (i.e., in the mountains north of Kotzebue) than at Kotzebue. In contrast, the observed precipitation depths at the mine have been much greater than at Kotzebue (see Table 5-1). It appears that the orographic and elevation effects affecting the precipitation at the mine are not adequately accounted for in Figure 2-12 of TP-47.

One of the stations for which precipitation data was used in developing the charts of TP-47 was in Kotzebue. Therefore, it is concluded that the estimated PMP at Kotzebue is based on analysis of actual site-specific data and is reasonable. A procedure suggested to estimate PMP for locations similar to Red Dog Mine includes adjusting the PMP for the nearest nonorographic location (e.g., Kotzebue) for topographic effects using comparison of extreme rainfalls of various categories at the two locations (National Research Council, 1985).

From Figure 2-12 of TP-47, the 24-hour point PMP for Kotzebue is approximately 7.8 inches. Concurrent precipitation depths at the gages at Kotzebue and Red Dog Mine were compared to develop a relationship between the two stations. The resulting multiplying factor is deemed to represent the orographic and meteorologic effects applicable to the Red Dog Mine area.

The ratio of the mean of daily maximum values observed in each year during the period of concurrent data (1992 to 2007) for the Red Dog Mine and Kotzebue gages is 1.7 (see Table 5-1). This ratio was used as a multiplying factor to estimate the 24-hour PMP at Red Dog Mine. Thus, the estimated 24-hour PMP at Red Dog Mine is 13.26 inches (equal to 1.7 x 7.8 inches).

Approximate methods used to verify the reasonableness of the estimated PMP are described in Appendices A-11, 12.

5.4.5   Probable Maximum Flood Series

The same catchment area (2.52 square miles) and CN (98) that were used for the May runoff calculations were used along with the calculated PMP to determine the PMF series contribution to the tailings impoundment storage capacity of 3,951,341 cubic yards.

This volume is equivalent to a pond depth of 4.25 feet above the water surface level at El. 978.35 that was obtained after storing the runoff during the critical month of May (Subsection 5.3.7). The storage of this volume in the tailings impoundment raises the water level to El. 982.60 (Appendix A-14).

5.5   100-YEAR FLOOD

The 100-year flood contribution to the pond surcharge capacity was determined using the same methodology as used in determining the PMF. The 24-hour, 100-year precipitation for Kotzebue was determined from Figure 3-59 of Technical Paper 47 (Weather Bureau, 1963) to be 2.5 inches. Adjusting this value by the previously used Red Dog Mine to Kotzebue site ratio of 1.7, yields a 24-hour, 100-year precipitation of 4.25 inches.

The same catchment area (2.52 square miles) and CN (98) which were used for the May runoff calculations were used along with the calculated 24-hour, 100-year precipitation to determine the 100-year flood contribution to the pond storage capacity of 817,428 cubic yards.

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This volume is equivalent to a pond depth of 0.92 feet above the water surface level of El. that was computed after adding the PMF series (Subsection 5.4.5). The storage of this volume in the tailings impoundment raises the water level to El. 983.52 (Appendices A-14, 15).

5.6   SURCHARGE CAPACITY AVAILABLE FOR ANNUAL RUNOFF

Most of the surcharge capacity is designed to hold the runoff from the PMF Series. Since the probabilistic PMF is an extremely rare event and has a smaller return period than 1 in 1,000,000 years (Appendix A-13) it is highly unlikely that it would occur simultaneously with another extreme event.

Should unforeseen circumstances, such as a prolonged WTP failure or an institutional failure occur, it is highly probable that the PMF series surcharge capacity will be available for annual runoff. The PMF series storage capacity could store approximately 1.6 years of runoff, given the catchment area and precipitation data used in this design (Appendix A-15).

In another hypothetical and almost improbable scenario, if pumping was interrupted with no repairs for a full year, only 2.77 feet of the total surcharge capacity depth of 6.52 feet (Subsection 5.7) would be used to store the one-year surface runoff (Appendix A-16). Then hypothesizing that a full PMF also occurs on top of a one-year pump break-down with no repairs, a storage depth of only 5.79 feet would be used (Appendix A-16). This leaves an additional storage space of about 0.71 feet below the spillway crest. Thus, even during this extremely severe joint event, there would be no flow in the spillway. The joint probability of a one-year long pump break-down and a PMP during the same time is extremely low.

5.7   CONCLUSIONS

The surcharge capacity of the tailings impoundment at closure is summarized in Table 5.2.

Table 5-2 Surcharge Capacity

	Water Surfaces	Incremental height (ft)	Design elevation (ft)
Design Criteria	Top of Tailings		975.0
	Water Cover	2.00	977.0
Surcharge Capacity	Spring Freshet	1.35	978.4
	PMF Series	4.25	982.6
	100-Year Flood	0.92	983.5
	Spillway Crest Elevation	**-**	**983.5**

As shown in Table 5-2, the total surcharge depth below the spillway crest available to store surface runoff under abnormal conditions is approximately 6.5 feet (1.35 + 4.25 + 0.92 feet).

6.0 SPILLWAY CAPACITY

6.1   INTRODUCTION

This section describes the preliminary design level capacity of the spillway for the tailings main dam at its ultimate closure configuration. The design concept is an open-channel, side-hill spillway aligned along the west hillside above the tailings impoundment and outside the west abutment of the dam with discharge into Red Dog Creek downstream of the seepage collection system. This same concept was adopted at the freshwater dam and has operated successfully there since it was constructed in 1989.

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The capacity of the spillway at this preliminary design level was developed on the basis of two components:

- Routed inflow design flood
- Freeboard criteria.

The calculations for these components of the spillway design are described in the following subsections.

6.2   ROUTED INFLOW DESIGN FLOOD

The Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) (USACE, 2001) model was used to estimate the peak flow through the spillway and the elevation of the water in the tailings impoundment when routing the IDF and the extreme event of a PMF. The HEC-HMS model input parameters that were used in the modeling include the catchment area, CN and initial abstraction as described in subsections 5.3.3, 5.3.4 and 5.3.5, respectively, as well as the following:

- Inflow design flood
- Lag time
- Stage storage discharge curve.

These additional parameters are described in the following subsections.

6.2.1   Inflow Design Flood

The IDF that was used to size the spillway capacity is equal to one half of the PMF. This IDF was selected because the spillway crest is above the surcharge capacity requirement as described in Section 5.0, and the spillway is a fail safe measure for extreme events with extremely low probability of occurrence. In addition, the spillway is tested to safely pass the full PMF as an extreme event.

The extreme event of another storm occurring in addition to the PMF series and the 100-year flood represents a recurrence interval of far less than 1 in 1,000,000 years. Therefore, based upon the conservative surcharge capacity that has been provided and the proposed operational pumping, the spillway is not expected to be used at all during the life of the dam.

URS accounted for one half of the PMF by using one half of the PMP of 6.63 inches in the HEC-HMS model. Although the tailings main dam is classified as a Class II (significant) hazard dam in accordance with Title 11 of Alaska Administrative Code 93.157 (URS, 2007), the preliminary spillway design was selected for conservatism to achieve the minimum IDF requirements of the NRCS (2005) criteria for the design hydrograph to size an emergency spillway for a "high hazard" dam. These criteria are as follows:

$$P_{100} + 0.12 \ (PMP\text{-}P_{100}) = 4.25 + 0.26 \ (13.26 - 4.25) = 6.59 \text{ in,}$$

Where:

P_{100} = 24-hour, 100-year storm which is equal to 4.25 inches (Subsection 5.5)
PMP = 24-hour PMP = 13.26 inches (Subsection 5.4)

The spillway was also sized to handle a PMF, in case of extreme events.

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6.2.2 Lag Time

Lag time (L_t) is a weighted time of concentration (T_c) which is defined as the time duration from the center of mass of the rainfall to the center of mass of the outflow. Some factors which affect lag time include the ground slope, ground permeability and vegetation interception.

L_t is usually estimated to be 60% of T_c (Kent, 1972). T_c is the travel time from the hydraulic furthermost point in a watershed to the outlet of the watershed. Kirpich's Equation (Chow, 1964) was used to calculate a T_c of 10.8 minutes. This assumed that the T_c was from the farthest part of the watershed to any edge of the tailings impoundment, because the tailings impoundment has a level water surface to the spillway inlet (Figure 6-1) (Washington State Department of Ecology, 2001). Input parameters for Kirpich's equation are below (Appendix B-3):

$$T_c = 0.0078 * (L^{0.77} / S^{0.385}) = 0.0078 * (4,763^{0.77} / 0.16^{0.385}) = 10.8 \text{ min}$$

Where:

T_c = time of concentration (min)

L = length of travel (ft) = 4,763 ft

S = slope (ft/ft) = 743 ft / 4,763 ft = 0.16 ft/ft (Determined from ArcGIS profile tool)

$$L_t = T_c * 0.6 = 10.8 \text{ min} * 0.6 = 6.5 \text{ min}$$

Where:

L_t = lag time (min)

T_c and L_t were also estimated using the procedure by NRCS, Technical Release 55 (1986), which is commonly used in the United States. A travel time (t_i) was estimated for each type of water movement. T_c and L_t were then calculated using the following formulas:

$$T_c = t_s + t_{sc} + t_c$$

$$L_t = (T_c) * (0.6)$$

Where:

t_s = travel time for sheet flow

t_{sc} = travel time for shallow concentrated flow

t_c = travel time for channel flow

The three types of flow from this formula are sequential and are described as follows:

- **Sheet flow** – This is a flow of water over the plane surfaces. It usually occurs in the headwater of streams, and typically in less than 300 feet it becomes concentrated flow.

- **Shallow concentrated flow** – This flow occurs for a maximum of 300 feet after sheet flow and typically becomes concentrated in very shallow channels.

- **Channel flow** – This flow begins after shallow concentrated flow either when survey cross section information is available, or where channels are visible on air photos, or where blue lines (indicating streams) appear on USGS quadrangle sheets.

The overall lag time for the tailings pond catchment using the NRCS method was estimated to be 16.1 minutes (Appendix B-2-4). However, a Kirpich lag time of 6.5 minutes was used as the input into the HEC-HMS model in order to take a conservative approach.

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6.2.3 Stage-Storage-Discharge Curve

The discharge rating curve of a reservoir is a function of the type of spillway used, inflow control structures, and water elevation versus storage. The spillway of the tailings main dam was represented as a broad-crested weir with a base width of 40 feet and side slopes of 2.5:1, which are similar to the spillway at the freshwater dam. The following equation was used to estimate the spillway discharge:

$$Q = CL\ H^{3/2} \text{ (Brater and King, 1976)}$$

Where:

        Q = Discharge, cfs
        C = Discharge coefficient (2.64)
        L = Width of Weir, feet (bed width = 40 ft, side slope 2.5H:1V)
        H = height of water above weir, ft

According to Brater and King (1976), Table 5-3, a broad-crested weir with approximately 1.2 feet of measured head and a 15-foot breadth of crest of weir will have a C = 2.64 (table maximum is 15-foot weir breadth, the best approximation of the designed 30-foot breadth).

The stage-storage-discharge-table is presented in Appendix B-5.

6.2.4 Hydrologic Model Settings

The following HEC-HMS Model settings calculated in the previous subsections were entered in the respective fields below to route the IDF and PMF through the spillway.

- **Reservoir model (1 catchment area)**
  - Method = Outflow Curve
  - Storage Method = Elevation-Storage-Discharge
  - Stor-Dis Function = Storage Discharge
  - Elev-Stor Function = Elevation Storage
  - Initial Condition = Elevation
  - Initial Elevation (feet) = 983.5
- **Basin model (1 catchment area)**
  - 3 components of the catchment area were defined
    - Loss rate method – SCS CN
    - Transform method – SCS
    - Base flow method – no base flow
    - Curve number= 98, Ia = 0.04, Impervious % = 0
- **Meteorological model**
  - Method – SCS Storm Type 1 for Alaska (NRCS, 1986)
  - Depth 1 – One half the 24-hour, PMP event equal to 0.5 *13.26 inches = 6.63 inches
  - Depth 2 – 24-hour, PMP event = 13.26 inches
- **Control Specification**
  - Time interval = 1 minute

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- Start and end date and time of the storm event are specified arbitrarily for the duration of over 24 hours

6.2.5 Hydrologic Model Results

The results of the HEC-HMS models are presented in Appendices B-6, 7. The peak flow and depth of flow from routing one half of the PMF through the spillway were calculated to be 154.7 cubic feet per second (cfs) and 1.2 feet, respectively.

An additional 0.22 feet of water height will be required in the tailings impoundment to provide the head needed to overcome frictional and convergence losses from pushing the water up the inlet channel to the spillway outlet. The required head was determined using Hydrologic Engineering Center-River Analysis System (HEC-RAS) software (USACE, 2004). The HEC-RAS output files are in Appendices B-8 and 9.

Therefore, the maximum water surface level in the tailings impoundment during the half PMF event is estimated to be El. 984.9, which is 1.42 feet above the spillway crest at El. 983.5.

6.2.6 Hydrologic Model Extreme Case Check

A full PMF, rather than one half of the PMF, was routed through the spillway as an extreme-case condition check for spillway capacity. The peak flow and depth through the spillway that would develop from routing a PMF were calculated to be 404.8 cfs and 2.2 feet, respectively.

An additional 0.28 feet of water height will be required in the tailings impoundment to provide the head needed to overcome frictional and convergence losses from pushing the water up the inlet channel to the spillway outlet. The required head was determined using HEC-RAS software. The HEC-RAS output files are in Appendices B-10 and 11.

Therefore, the maximum water surface level in the tailings impoundment dam during the full PMF event is estimated to be El. 986.0, which is 2.48 feet above the spillway crest at El. 983.5, and just over one foot more that was calculated for one half of the PMF as described in subsection 6.2.5.

6.3 FREEBOARD CRITERIA

For the preliminary design of the spillway, the required freeboard on top of the IDF (one half PMF) was calculated in order to prevent the tailings main dam from being overtopped. This required freeboard must accommodate the wind setup and wave runup as described in subsections 6.3.1 and 6.3.2.

Additionally, the required freeboard on top of a PMF was calculated to size whatever parapet berm might be needed to prevent overtopping of the tailings main dam as a result of wind-generated waves in a very extreme precipitation event. The parapet berm is adequate for this purpose, and avoids the need to build a higher dam that is not necessary.

The impacts of a potential decrease of the dam crest by settlement due to consolidation of the dam embankment and foundation materials and by deformation caused by earthquakes need to be addressed in the design by overbuilding the dam by at least the estimated settlement and deformation amounts. This is addressed in the preliminary design report for the ultimate closure configuration of the tailings main dam, which is currently in preparation.

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6.3.1 Wind Setup

Wind setup is the horizontal stress that the wind exerts on water as it blows across the water surface. This stress causes the piling up of water on the leeward end of body of water, which in this case was assumed to be the north end of the tailings impoundment at the tailings main dam and spillway inlet area. The parameters used to calculate wind setup are listed below and detailed in the following subsections.

- Fetch length
- Water depth
- Design wind velocity
- Wind setup equation

6.3.1.1 Fetch Length

The fetch length was determined in ArcGIS (ESRI, 2006) using URS design drawings for the tailings main dam and beach at the ultimate closure configuration (URS, 2007) and an SRK layout figure showing the planned back dam at closure (SRK, 2008a).

It should be noted that a 600-foot wide beach is part of the preliminary design of the ultimate closure configuration of the tailings main dam. However, at present, a beach is not planned as part of the ultimate closure configuration of the back dam (SRK, 2007a). Therefore, the fetch length is calculated from the upstream edge of the crest of the tailings back dam, in order to provide the longest fetch and a conservative condition for the preliminary spillway design at the tailings main dam.

The fetch length was measured as the longest "straight-line" distance (USACE, 2006) between the tailings back dam and 110 feet upstream of the tailings main dam where the still-water pond level reaches at an elevation 984.9 feet from routing the IDF through the spillway. The fetch length was calculated to be 8,699 feet or 1.65 miles (2.65 kilometers).

6.3.1.2 Design Water Depth

The design water depth was calculated to be 9.9 feet measured from the maximum water surface level in the tailings impoundment based on the HEC-HMS model of the IDF routed through the spillway, minus the struck tailings elevation (El. 984.9 – El. 975 feet = 9.9 feet).

6.3.1.3 Design Wind Velocity

The design wind velocity was determined using USACE (2003), NRC (American Nuclear Society, 1992) and NRCS (1983) guidance. The calculations from these three techniques produce similar design wind velocities. Maximum wind velocity data is available for the Bons Creek Weather Station in the vicinity of the Red Dog Mine in 1-hour increments for the period of 2000 to 2007 (TCAK, 2008a).

Wind velocities are frequently observed and reported as the fastest mile or extreme velocity, which are considered to be synonymous. Usually, the fastest mile wind speeds are obtained for a short time period of generally less than 2 minutes, such as during thunderstorms, and may be available in 1-hour increments.

The maximum values in the Bons Creek data represent the maximum wind velocity readings taken every 2 seconds for each hour of the day (Diehl, 2008a). These maximum wind velocity readings were

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considered fastest mile wind velocities for design wind velocity calculations. The design wind velocity was calculated in the following five steps listed below and detailed in the wind calculations in Appendices B12-16.

- Wind direction determination
- Fetch determination
- Design wind velocity determination: USACE Coastal Engineering Manual
- Design wind velocity determination: NRC Guidance
- Design wind velocity determination: NRCS Guidance

A design wind velocity of 72.0 miles per hour (32.20 meters per second) was selected as a conservative calculation of the design wind velocity. The comparable results for design wind velocity from all three methods provide justification for this design velocity (Appendices B-12-16).

6.3.1.4 Wind Setup Result

The wind setup was calculated to be 0.62 feet for the IDF of one half of the PMF, and 0.56 for the extreme event of a full PMF. This wind setup was calculated from the following equation (IDF values shown in example) (USACE, 1997):

$$S = \frac{U^2 \times F}{1400 \times D} = \frac{72.0^2 \times 1.65}{1400 \times 9.9} = 0.62 \text{ ft}$$

Where:
- S = Wind setup (feet)
- U = Design wind velocity (72.0 miles per hour)
- F = Wind fetch (1.65 miles)
- D = Average water depth (IDF = 9.9 feet, PMF = 11.0 feet)

The design wind velocity was used to calculate the wind setup occurring at the time of wave runup. Wind setup is normally calculated using average wind velocities. However, when the wind setup is being added to wave runup for a cumulative effect, the same wind velocity is assumed in both calculations.

Wind setup associated with the routing of the extreme event of a PMF was calculated to be 0.56 feet using the same equation above, assuming an average water depth of 11.0 feet (El. 986.0 - El. 975.0) and the same wind velocity [assuming negligible effects to the design wind velocity due to the slight extension in fetch length (~ 30 feet) during PMF].

6.3.2 Wave Runup

Wave runup is the movement of water up a structure or beach upon the breaking of a wave. Wave run-up was estimated using the USACE Coastal Engineering Manual (2006) and United States Bureau of Reclamation (USBR) guidance (1992). The parameters used to calculate wave runup are listed below and detailed in the following subsections:

- Fetch-limited wave period
- Duration-limited wave period
- Wave runup

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6.3.2.1 Fetch-Limited Wave Period

The fetch length is 8,699 feet (2.65 kilometers) and the design wind speed is 72.0 miles per hour (32.20 meters per second), giving a fetch limited wave period of 2.3 seconds from Figure II-2-24 of the USACE Coastal Engineering Manual.

6.3.2.2 Duration-Limited Wave Period

The duration-limited wave period is estimated from Equation II-2-39 of the USACE Coastal Engineering Manual as follows:

$$T_p = 9.89 \left[\frac{d}{g} \right]^{1/2} = 9.89 \times \left(\frac{9.9\ ft}{32.174\ ft/\sec^2} \right)^{0.5} = 5.5 \sec$$

Where:

d = Design water depth = 9.9 feet
g = 32.174 feet/second squared

6.3.3 Significant Wave Height

The wave period is clearly fetch-limited (2.3 sec < 5.5 sec) and therefore, the significant wave height was determined from Equation II-2-36 from the Coastal Engineering Manual for fetch-limited waves to be 1.03 meters (3.38 feet) as shown below:

$$C_D = 0.001 \times (1.1 + 0.035\ U_{10}) = 0.001 \times (1.1 + 0.035 \times 32.20\ m/s) = 0.002227$$

$$u_* = \sqrt{C_D \times U_{10}^2} = \sqrt{0.002227 \times (32.20\ m/s)^2} = 1.52\ m/s$$

$$H_{m0} = \frac{4.13 \times 10^{-2} \times \left(\frac{gX}{u_*^2} \right)^{\frac{1}{2}} \times u_*^2}{g} = \frac{4.13 \times 10^{-2} \times \left(\frac{9.81\ m/s \times 2{,}650\ m}{(1.52\ m/s)^2} \right)^{\frac{1}{2}} \times (1.52\ m/s)^2}{9.81\ m/s} = 1.03\ m$$

Where:

U_{10} = Design wind speed at 10 meters (32.8 feet) = 32.20 meters per second (105.64 feet per second)
X = Fetch length = 2,650 meters (8,699 feet)
g = Acceleration of gravity = 9.81 meters per second per second (32.17 feet per second per second)
C_D = Drag coefficient
u_* = Friction velocity (meters per second)
H_{m0} = Energy-based significant wave height (meters)

6.3.4 Wave Runup for Inflow Design Flood

URS calculated the wave runup on the tailings beach upstream of the tailings main dam at the ultimate closure configuration for the IDF by using predictive equations out of the USACE Coastal Engineering

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Manual. The expression for maximum wave runup was calculated according to the following equation (Appendix B-18):

$$R_{max} = H_0 \times 2.32 \times \xi_0{}^{0.77}$$

Where:
R_{max} = maximum wave runup
H_0 = Significant deep-water wave height (3.38 feet)
ξ_0 = Surf similarity parameter (0.028)

The wave runup was calculated to be 0.50 feet for the waves against the beach assuming a beach slope of one percent down away from the dam crest to the berm that will confine the beach 600 feet away from the dam.

6.3.5   Wave Runup for Extreme Event of Probable Maximum Flood

In the extreme event of a PMF, wave runup will occur against a parapet berm that is constructed of coarse rock or riprap along the upstream side of the tailings main dam. The following equation from USBR guidance (1992) was used for calculating wave run-up against a parapet berm:

$$R_s = \frac{H_s}{0.4 + (H_s/L)^{0.5} \cot(x)} = \frac{3.38\,ft}{0.4 + (3.38\,ft/26.8\,ft)^{0.5} \cot(21.8°)} = 2.62\,ft$$

Where:
R_s = Wave run-up (feet)
L = Wave length (feet) = 26.8 feet (assuming depth of 11.0 feet)
H_s = Significant wave height = 3.38 feet
x = Angle of upstream face of riprap berm with horizon (2.5:1) = 21.8°

6.3.6   Freeboard Estimate

The total freeboard for the IDF, based on the wind setup and wave runup, required for the preliminary spillway design is estimated to be 1.1 feet, which is determined by adding the wind setup estimate of 0.62 feet and the wave runup estimate of 0.50 feet and then rounding off. This 1.1 feet of freeboard above the maximum water surface elevation during the IDF is far greater than the routing of the NRCS minimum freeboard hydrograph requirements for a "significant" class dam (NRCS, 2005) shown below:

$$P_{100} + 0.4\ (PMP\text{-}P_{100}) = 4.25 + 0.4\ (13.26 - 4.25) = 7.9 \text{ inch}$$

Where:

P_{100} = 24-hour, 100-year storm which is equal to 4.25 inches (Subsection 5.5)
PMP = 24-hour PMP = 13.26 inches (Subsection 5.4)

It is also equivalent to the routing of a 13.26-inch (PMP), which is the minimum freeboard hydrograph required to be routed by NRCS criteria for a "high hazard" class dam. The NRCS freeboard hydrograph is used to evaluate the total spillway flow capacity of the dam and, consequently, establish the minimum settled elevation of the top of the dam.

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The total freeboard, based on wind setup and wave runup, required for the extreme event of a PMF is estimated to be 3.2 feet, which is determined by adding the wind setup estimate of 0.56 feet and the wave runup estimate of 2.62 feet.

A 3.2-foot-high parapet berm aligned along the upstream edge of the crest of the tailings main dam will accommodate waves associated with the extreme event of a PMF. The berm could be constructed from either coarse rockfill or riprap. It will not be necessary to raise the entire dam, since only waves are expected to occur above El. 986.0.

6.3.7   Dissipation of Wind Setup and Wave Runup at Spillway Inlet

The spillway entrance will not be sheltered from the effects of wind setup and wave runup as the rest of the main dam will be from the beach. To minimize the potential for blockage of parts of the spillway or its entrance due to floating debris or ice, a boom can be provided across the spillway entrance. This boom will also reduce wind setup and wave runup effects at the spillway entrance.

This approach is supported by URS (2005) field observations of estimated four-foot waves that were dampened to almost no waves by a pipeline on the surface of the pond as described below:

> *"On October 19, a tailings discharge pipe in the western side of the impoundment, aligned perpendicular to the wind and wave direction, was observed to act as an effective wave break. While four-foot high waves crashed into the windward south side of the pipe, the water on the leeward north side of the pipe was relatively calm for several hundred feet north of the pipe."*

6.4   VERIFICATION

6.4.1   Field Observation

URS (2005) field observations of approximately four-foot high waves on the tailings pond were made during the 2004 Periodic Safety Inspection (PSI) of the tailings main dam at a time when TCAK recorded some of the strongest winds ever at the mine. The 2004 PSI contained the following field observations:

> *"The wind blew very strongly during both days of the PSI, but changed direction by about 90 degrees overnight. The wind on October 18 blew east to west, at recorded speeds of 45 to 56 miles per hour (mph) during the afternoon and a maximum speed of 67 mph at 7 p.m. The wind on October 19 blew south-south-east to north-north-west at recorded speeds of 44 to 54.3 mph during the afternoon and a maximum speed of 54.3 mph at 2p.m.*
>
> *Higher gusts were observed between the recorded speeds. On October 18, with the truck parked parallel to the wind direction, a gust swung the door beyond its opening range. On October 19, on the west side of the tailings pond, the windward door could not be opened while the truck was parked across the wind, and the wind generated waves on the tailings pond about four feet high.*
>
> *The observed four-foot high waves are consistent with the significant wind-generated wave height of 3.94 feet calculated as part of the tailings dam freeboard analysis (Geomatrix, 2003). The calculation was performed by determining the effective fetch*

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length of the tailings impoundment and estimating the fetch-limited wave period from a coastal engineering manual.

However, similar 4-foot-high waves will be greatly dissipated by the one percent slope, 600-foot-wide beach to be placed in front of the main dam, down to a height of 0.50 feet.

6.5   CONCLUSIONS AND RECOMMENDATIONS

URS recommends the preliminary dam crest level, based on the spillway capacity, to be El. 986.0 as shown in Table 6-1. This accounts for the maximum flow predicted from routing an IDF of one half the PMF and 1.1 feet of freeboard to accommodate wind setup and wave runup on the dam. The calculated spillway capacity and corresponding water surface heights are shown in Table 6-1.

Additionally, a 3.2-foot high parapet berm will need to be installed along the upstream edge of the dam crest to contain freeboard associated with routing a PMF through the emergency spillway over and above the surcharge capacity. The parapet berm will only be necessary to mitigate wind-generated waves.

Table 6-1 Spillway Capacity

	Water Surfaces	Incremental height (ft)	Design elevation (ft)
Spillway Capacity	Spillway Crest Elevation	-	983.5
	Inflow Design Flood	1.42	984.9
	Freeboard	1.12	986.0
	Preliminary Dam Crest	**-**	**986.0**

7.0 PRELIMINARY SPILLWAY DESIGN

7.1   INTRODUCTION

URS has completed a preliminary spillway design for the tailings main dam at the ultimate closure configuration consistent with the original design intent (Dames & Moore, 1986) for the dam at closure. The design concept is an open-channel, side-hill spillway aligned along the west hillside of the tailings impoundment and outside the west abutment of the dam so that it will discharge to Red Dog Creek downstream of the seepage collection system. The spillway will not be located in the dam embankment.

The preliminary spillway design was based on geotechnical conditions assessed from a 2005 geotechnical investigation conducted along the conceptual spillway alignment and a general knowledge of the geotechnical conditions in the right abutment area of the tailings main dam. These conditions were transposed and used to set the preliminary alignment of the spillway and spillway channel profiles.

A general methodology of spillway design with four basic considerations was used from the "*Hydraulic Design of Spillways*" (USACE, 1992):

- Subcritical flow is required in the spillway approach
- Critical flow as the water passes over the spillway crest
- Supercritical flow in the chute below the crest

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- Transitional flow at or near the terminus of the chute where the flow must transition back to velocities which minimize scour potential in the receiving creek

The geotechnical conditions, preliminary spillway alignment and associated cross-sections and components are detailed in the following subsections.

7.2   GEOTECHNICAL CONDITIONS

A geotechnical investigation was conducted in 2005 to establish the soil and rock conditions along the general alignment of the conceptual spillway for purposes of confirming the conceptual design, and specifically to determine the amounts of rock excavation and riprap placement that would be required. This investigation was part of a larger investigation for the conceptual design of the tailings main dam future raises to closure. The geotechnical information obtained from the investigation was used in this preliminary spillway design, since its alignment was close to the alignment of the conceptual spillway.

The subsurface conditions along the spillway alignment were investigated by drilling four borings and one test pit at locations determined by URS and TCAK as shown in the attached drawing TCP-15. These locations were determined by URS in coordination with TCAK, and were approved by ADNR.

The field exploration program was conducted between November 20 and 22, 2005. Four borings (SS-02-05, SS-03-05, SS-21-05, and SS-22-05) were drilled to depths of 14.1, 20.1, 18.6 and 15.1 feet, respectively, below the ground surface. One test pit (TP-01-05) was excavated to a depth of 10 feet. The boring and the test pit locations within the generalized layout of the spillway alignment are shown in the Drawing TCP-15. The boring locations were surveyed by TCAK.

The field exploration procedures and logs of the borings and test pit are described in a geotechnical data report (URS, 2006a). The general soil profile from top to bottom that was interpolated from the geotechnical investigation along the alignment of the spillway and consists of the following soil and rock types with approximate thickness:

- 1 foot of tundra organic root mat
- 2.5 to 6 feet of brown clayey silty sand with gravel
- 1 to 4 feet of highly weathered black shale
- Weak to medium strong laminated shale to the bottom of the borings and test pit

Drawing TCP-16 shows the cross sections including the spillway structure. The conceptual design of the spillway sections shown on Drawing TCP-16 has assumed that the organic root mat, clayey silty sand with gravel, and the highly weathered black shale will be over-excavated and replaced with riprap to provide the required protection for the spillway. Soil profiles along the centerline of the spillway reaches are shown in drawings TCP-17 through TCP-19.

7.3   ALIGNMENT

The alignment of the spillway channel for preliminary design purposes was split into five reaches based on channel slope and stationing shown on Drawing TCP-15. The reaches are labeled as Reach 1 to 5 from bottom to top, or from downstream to upstream. The reaches were numbered this way because it is the way that the HEC-RAS model works. However, for reader ease, the reaches in this preliminary spillway design report are described from upstream to downstream as described below.

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Reach 5 extends from station 19+05 at the spillway entrance along the northwest corner edge of the tailings impoundment until station 14+60 where the flat spillway outlet control section begins. Reach 5 is a relatively long, wide, deep and negatively sloped spillway entrance that is designed to maintain a large surface area of subcritical flow to minimize head losses from driving water through the spillway. Reach 5 transitions to Reach 4 where the channel flattens for the spillway outlet control section. The base width is 40 feet to reduce the water level in the impoundment needed to pass a PMF through the spillway.

Reach 4 extends from the entrance of the flat control section at station 14+60 to a broad-crested weir at the spillway exit station 14+30. The spillway control section is designed to transition the approaching subcritical flow from the spillway entrance to a critical depth at the spillway outlet and to supercritical flow down Reach 3. Reach 4 transitions to Reach 3 at station 14+30 where the channel changes to a 3.7 percent slope down the hillside. The base width is 40 feet to reduce the water level in the impoundment needed to pass a PMF through the spillway and maintain uniform flow approaching the weir.

Reach 3 extends from the spillway outlet at station 14+30 to station 9+29 where the channel steepens to a 25 percent slope. The purpose of Reach 3 is to convey water down the hillside at supercritical flow while conforming to the topography as much as possible to reduce required excavation. Reach 3 transitions to Reach 2 where the topography of the hillside steepens sharply in its descent to Red Dog Creek. It has a bottom width of 40 feet to maintain uniform flow down the initial outlet channel reach.

Reach 2 runs from the end of the 3.7 percent slope at station 9+29 to station 2+32 where the inlet to the stilling basin begins. The purpose of Reach 2 is also to convey water down the hillside at supercritical flow while conforming to the topography as much as possible to reduce required excavation. It has a bottom width that transitions from 40 to 12 feet based on minimizing the excavation quantities and low depth of flow at the 25 percent slope.

Reach 1 runs from the end of the 25 percent slope at station 2+32 to the edge of Red Dog Creek at station 0+28. The purpose of Reach 1 is to transition the supercritical flow from the spillway outlet channel back to subcritical flow to enter the creek at a non-scouring velocity. The transition in Reach 1 is accomplished through a hydraulic jump induced with a USBR Type I/ V stilling basin. In addition, a riprapped apron widening from 12 to 28 feet follows the exit of the stilling basin to reduce channel velocity.

Table 7-1 provides details on the locations, slopes and lengths of the reaches, along with the estimated flow depths and flow velocities.

Table 7-1 Spillway Channel Design Details for Full PMF Routing

Reach number	Stationing	Approximate Slope (%)	Base Width (feet)	Length (feet)	Flow Depth (feet)	Flow velocity (feet/sec.)
5	19+05 to 14+60	-1.8	297 to 40	445	6.98 to 2.48	0.15 to 4.78
4	14+60 to 14+30	0.0	40	30	2.48	4.78 to 6.70
3	14+30 to 9+29	3.7	40	501	1.13	8.56
2	9+29 to 2+32	25.1	40 to 12	697	1.25	21.41
1 (stilling basin inlet)	2+32 to 1+94	50.0	12	38	1.03	27.03
1 (stilling basin apron)	1+46 to 0+28	0.0	12 to 28	118	2.77 (max)	9.03 (max)

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The outlet channel downstream of the spillway control point will be relatively steep, ranging in slope from 3.7 percent at the top to 25 percent at the bottom. This slope is dictated by the natural ground slope which gets steeper from top to bottom. As part of the requirement for designing erosion protection for steep slopes, the channel alignment will be straight until it is beyond the toe of the tailings embankment.

The channel side slopes will be constructed to slopes of 2.5:1 at the spillway control section and riprapped areas of the outlet channel. The parts of the outlet channel in laminated shale bedrock will have a 1.5:1 side slope. Transitions between different side slopes will be made gradually over the course of 50 feet.

The alignment and reaches of the spillway channel are shown on Drawing TCP-15. Typical sections and details at representative stations are shown in Drawing TCP-16.

7.4   SPILLWAY ENTRANCE

The spillway entrance was designed to have an approach velocity of less than 2 feet per second for over 200 feet into the inlet channel in order to prevent scouring of tailings at the spillway entrance. A negative 1.8 percent slope, depth and wide opening of the spillway entrance help reduce the water surface level in the tailings impoundment required to force water through the spillway channel. Therefore, the spillway entrance is located approximately 270 feet upstream of the dam crest as shown on Drawing TCP-15.

The tailings pond area at the spillway entrance will have a reduced tailings beach and berm at El. 979.0 to prevent seepage at a normal pond operating level of El. 977.0. The reduced tailings beach and berm will be of similar design to the overall tailings beach and berm, but built to a lower elevation to allow a deep, wide spillway entrance channel leading up to the spillway outlet. The reduced beach and berm area prior to the spillway entrance can be seen in Drawing TCP-15.

A road will be required around the perimeter of the tailings impoundment at the ultimate closure configuration. The section of the perimeter road between the tailings main dam to the western side of the tailings impoundment will cross through the emergency spillway, similar to the design of the road access through the spillway at the freshwater dam. A road is shown on Drawing TCP-15. However, the final road location should be field-fitted and integrated with the spillway during its excavation and construction.

7.5   OUTLET CHANNEL SECTIONS

7.5.1   Channel Depths and Velocities

The spillway outlet channel reach slopes were aligned and sloped to ensure supercritical flow from the spillway outlet to the stilling basin. Flowmaster Software (Bentley Systems, 2005) and NRCS Guidelines were used to determine the depths and velocities for each of the outlet channel reaches as shown in Table 7.1. The normal depths and velocities in the spillway outlet channels were estimated using the Manning's formula option in the Flowmaster Software as presented in the equation below:

$$Q = \frac{1.486 \times A \times R^{3/2} \times S^{1/2}}{n}$$

Where:

Q – Flow in cubic feet per second (ft^3/s) (404.8 cfs)

A – Area of flow in square feet (ft^2)

n – Manning coefficient (0.035)

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R – Hydraulic radius (feet)

S – Channel slope (feet/foot) (see Table 7-1)

The outlet channel maintained supercritical flow from the spillway outlet to the stilling basin. The results from both methods of determining depths and velocities are in Appendices C-2-4.

7.5.2   Channel Riprap Requirements

Riprap sizing for the spillway outlet channels was completed in accordance with the Federal Highway Administration (FHWA), "*Design of Roadside Channels with Flexible Linings*" (2005) and USACE "*Hydraulic Design of Flood Control Channels*" (1994).

On the basis of the current geotechnical knowledge of the spillway alignment, some segments of the reaches will need to be protected with riprap because they are exposed in soil and not in laminated shale. The only segments that need to be protected with riprap are those that are excavated in soil and not into laminated shale. These to be protected are shown on Table 7.2.

Table 7-2 Riprap Design for Spillway Outlet Channel

Reach Parts to be Riprapped	S = Slope = $\Delta El/\Delta L$	D_{50}	Spillway Channel Freeboard
(ft)	(ft/ft)	(ft)	(ft)
11+00 to 9+29	0.04	0.58	2.22
9+29 to 6+00	0.25	3.75	2.92
4+40 to 2+32	0.25	3.75	2.58

The areas that require riprap were initially sized for riprap based on USACE guidance and then checked and refined using FHWA guidance (Appendices C-5-7). The USACE riprap calculations were used as the final values since the methods produced larger, more conservative riprap sizes.

The freeboard (requiring riprap) associated with each spillway outlet channel section is shown in Table 7.2 and was calculated using the following equation from USACE guidance (1992):

$$Freeboard = 2.0 + 0.025 \times V \times d_n^{1/3}$$

Additional freeboard was added to the bend area in the spillway outlet channel from stations 4+63 to 8+70, according to FHWA guidance (Appendix C-7), to account for superelevation effects. The spillway outlet channel was designed to be 5 feet deep to conservatively provide protection for both water depth and freeboard, as well as for construction purposes.

A filter blanket is needed between the riprap and soil along the spillway channel to protect the soil from erosion and piping. The filter blanket gradations should be specified using soil information from geotechnical explorations that will be needed for the final design.

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The thickness of riprap is recommended to be 1.5 x the D_{50} for each channel from guidance (USACE, 1994; FHWA, 2005), but will be 2.0 x the D_{50} to account for the steep slopes and high sulfate loss rate of the Okpikruak shale that will likely be used as riprap. The other sections of the outlet channel will be cut into laminated shale bedrock and will not require riprap.

Okpikruak shale at the mine site that could be used for riprap has a specific weight of 165 pounds per cubic foot, but has a 61-percent sulfate loss, which is higher than the USACE and USBR limits as will be explained in the rock durability section of the preliminary dam design report for the tailings main dam that is currently being completed.

Further testing of rock durability will be required for purposes of final design, when selecting and sizing the final riprap to be placed in the spillway channel. If a high sulfate loss percentage persists in the available rock on site for riprap other options such as gabions and concrete blocks will need to be considered.

7.6   STILLING BASIN FOR ENERGY DISSIPATION

URS designed stilling basin dimensions at the outfall of the outlet channel to prevent scour and damage to Red Dog Creek. The stilling basin will be located upstream of the point of entrance of the outlet channel to the creek. However, the proposed spillway, outlet channel, and stilling basin are located and designed to never to be used, and if they are used, it would only be under extremely rare circumstances and events.

The spillway channel at this preliminary design level of the project is planned to discharge into the North Fork of Red Dog Creek, approximately 500 feet downstream of the toe of the seepage collection dam, as shown on Drawing TCP-15. This discharge outlet is at the start of Reach 1 at station 0+28.

At the proposed outfall location, the creek has an estimated bed slope of 1.5 percent. Borings drilled in the dam vicinity show that the soils along the channel bed consist mostly of loose silty and sandy gravel which are granular and not cohesive. Additional geotechnical investigations will be required to further characterize the creek bed material for its gradation and variation with depth. For preliminary design, it is assumed that the soil in the creek bed and surrounding floodplain will withstand a velocity of 7.0 feet per second without significant scour (Appendices C-9 and C-10).

Also at the proposed outfall location there are potential wetland areas that need to be considered and addressed as part of the final design and permit approval process.

The stilling basin is planned to in a remote area downstream of the dam and somewhat out of sight of the dam. Therefore, it should be designed to function in perpetuity with minimal maintenance and not be subject to damage from the annual freeze-thaw cycles. Thus, a stilling basin protected by a flexible structure such rock riprap is preferable to a rigid structure built of concrete or grouted riprap.

Depending on construction costs, rock could be used as loose riprap or filled in gabions made of weather-proof synthetic wires such as is currently used at the fish weir. During the final design, consideration can be given to the technical and economic feasibility of using concrete blocks cast in shapes such as rectangular, cubical, tetrapod and quadripod in high velocity zones.

There is a possibility that the stilling basin may be located on bedrock. Field investigations may be conducted to verify the existence of competent bedrock. This may eliminate the use of stones or concrete blocks and warrant a change in design.

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In principle, the hydraulic criteria for the design of riprap-protected stilling basins are similar to concrete structures. Guidelines for riprap-protected stilling basins at culvert outlets are in FHWA (2006) and Barfield et. al. (1981). However, preliminary URS computations indicate that the guidelines for riprap-protected stilling basins for culvert outlets may not be appropriate for the outlet channel because the estimated riprap sizing from the FHWA design procedure yielded D_{50} estimates far too small to handle the extreme incoming velocities from the steep spillway outlet channel.

A plunge basin energy dissipater (USBR, 1987) was also considered but was found to not be suitable because flow from the Red Dog outlet channel may not discharge into the air and then plunge downward into the stilling basin.

The feasibility of an outfall structure at the creek bank with energy dissipation arrangement in the creek bed was considered. It may require a sheet pile or concrete cutoff at or near the creek bank and construction of a stilling pool within the bed or floodplain of the creek. It may be hard to assure the long-term structural integrity of a concrete or steel pile cutoff in the site environment.

Therefore, a conventional stilling basin was designed for the outlet channel of the spillway at the ultimate closure configuration of the tailings main dam. This type of stilling basin utilizes a hydraulic jump for energy dissipation and is commonly referred to as a Type I or V stilling basin (USBR, 1984) or a stilling basin with free hydraulic jump (FHWA, 2006). A schematic of a stilling basin with a similar rock apron is provided publications by Mills (1976) and Barfield (1981).

The stilling basin will be constructed at the end of the spillway outlet channel, starting at station 1+94, to control erosion at the outlet area in the creek. The stilling basin will dissipate the energy from the flow prior to discharging into the creek. The stilling basin is designed to handle the inflow from the maximum design discharge of the spillway of 404.8 cubic feet per second and release water into the creek at roughly the same velocity as the creek itself.

The stilling basin will be 86 feet long and will consist of a 38-foot inlet apron sloping at 2:1 into the basin, a 40-foot horizontal floor, and 8 feet of downstream apron at a slope of 2:1. A 118-foot long downstream apron will lead into the creek. The first 61 feet of the apron will be riprapped and the remainder will be natural ground, since the velocity will be less than 7 feet per second after station 0+85.

Water will exit the stilling basin at approximately 9 feet per second and depth of 2.8 feet. The stilling basin and spillway channel, were designed so that sufficient tailwater depth is available even when the water depth in the creek is relatively low at about 2.0 feet. The stilling basin was designed using the FHWA, "Hydraulic Design of Energy Dissipators for Culverts and Channels" (2006) (Appendix C-8-13).

7.7   CONCLUSIONS

The spillway has been designed at this preliminary level of design to handle a PMF from the tailings impoundment into Red Dog Creek to prevent overtopping of the tailings main dam and to protect the creek at the outfall and surrounding area from scour or other damage.

The spillway approach channel and entrance configuration have been designed at this preliminary level to minimize the required head to push water through the spillway.

The spillway outlet channel slopes are designed to maintain supercritical flow from the crest to the stilling basin. Large riprap will protect the spillway from thee high velocities on the steep slopes of the outlet

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channel, but may require excavation to bedrock or the use of concrete blocks or gabions if large riprap is not available.

Further geotechnical investigations of the preliminary spillway alignment will be necessary to complete the final design of the spillway outlet channel.

The stilling basin was designed to reduce the flow velocities at the entrance to the Red Dog Creek and minimize the potential for channel bed scour. The stilling basin will be constructed out of gabions, riprap or concrete blocks to handle the entrance velocities.

8.0 FUTURE SPILLWAY CONSIDERATIONS

8.1 INTRODUCTION

The preliminary spillway design for the tailings main dam at the ultimate closure configuration has considered future spillway options for the possibility of the "clean pond" closure condition achieving a level of water quality that complies with clean water standards for discharges without treatment to the surface water bodies such as Red Dog Creek.

If water quality permits discharge is achieved, the tailings impoundment will be operated under passive conditions. Therefore, the hydrologic design basis for the impoundment and spillway will be dam safety and minimization of abnormal peak flows into the downstream channel which may be attributable to the tailings impoundment.

The preliminary spillway design considerations took into account the following two options to meet dam safety requirements and minimize abnormal peak flows for "clean pond" conditions at the ultimate closure configuration:

- Future spillway crest at El. 977.0
- Future spillway crest at level required for dam safety.

These two options are discussed in the following subsections of this report.

8.2 FUTURE SPILLWAY CREST AT EL. 977.0

The option of a future spillway crest at El. 977.0 would require the excavation of the spillway crest from the current design of El. 983.5 to 977.0 for a total excavation depth of 6.5 feet. This could be completed because the surcharge capacity from El 977.0 to 983.5 will no longer be required.

A spillway crest at El. 977.0 will provide the hydrologic safety of the tailings main dam in accordance with standard dam safety practices as per state and federal design standards (NRC, 1977; FEMA, 2004; ADNR, 2005). With the storage capacity available above the proposed spillway crest at El 977.0, the peaks of the flood hydrographs entering the impoundment are expected to be attenuated.

Lowering of the spillway crest from the preliminary spillway design crest at El. 983.5 to 977.0 will involve substantial earthwork and extension of some channel walls through the spillway alignment. The earthwork would consist of deepening the channel bottom by 6.5 feet, plus widening the channel by moving its side walls to achieve this additional depth.

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8.3 FUTURE SPILLWAY CREST FOR DAM SAFETY

This option of a future spillway crest as required for dam safety involves lowering the spillway crest below the current elevation of 983.5 feet only to the extent that sufficient freeboard is available to pass the full PMF without a parapet.

A spillway crest at an elevation such that there is sufficient freeboard between the dam and spillway crest to pass the PMF hydrograph with coincident wind wave activity will also ensure hydrologic safety of the tailings dam as per state and federal design standards (NRC, 1977; FEMA, 2004; ADNR, 2005).

With the storage capacity available above the proposed spillway crest, the peaks of the flood hydrographs entering the tailings impoundment are expected to be attenuated. Lowering of the spillway crest from the preliminary spillway design crest at El. 983.5 to a spillway crest elevation meeting freeboard requirements during the PMF will involve significantly less earthwork and extension of channel walls through the spillway alignment than lowering the spillway crest to El. 977.0.

However, a spillway crest above El. 977.0 will involve the storage of additional water behind the dam, which under extreme circumstances could represent more of a safety risk if released to Red Dog Creek.

9.0 CONCLUSIONS AND RECOMMENDATIONS

The preliminary spillway design will be an open-channel, side-hill spillway that will be aligned along the west hillside of the tailings impoundment and outside of the west abutment of the tailings main dam so it will discharge downstream of the seepage collection system. The spillway will not be located within the embankment of the tailings main dam.

URS recommends a spillway crest elevation and preliminary dam crest elevation of El.983.5 and El. 986.0, respectively, for the preliminary spillway design of the tailings main dam at ultimate closure configuration. The recommended spillway and dam crest elevations are based on the calculations summarized in Table 9-1.

Table 9-1 Tailings Impoundment Capacity

	Water Surfaces	Incremental height (ft)	Design elevation (ft)
Design Criteria	Top of Tailings	-	975.0
	Water Cover	2.00	977.0
Surcharge Capacity	Spring Freshet	1.35	978.4
	PMF Series	4.25	982.6
	100-year Flood	0.92	983.5
	Spillway Crest	**-**	**983.5**
Spillway Capacity	Inflow Design Flood	1.42	984.9
	Freeboard	1.12	986.0
	Preliminary Dam Crest	**-**	**986.0**

URS has recommended considering two options to meet dam safety requirements for passive operation of the tailings main dam at "clean pond" conditions. The options considered were lowering the preliminary spillway design crest at El. 983.5 to El. 977.0 or to a minimum spillway elevation meeting freeboard requirements during the PMF.

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Design calculations may change as the closure plan is being developed for the tailings impoundment, as tailings production and storage area projections are updated, and as the U.S. Weather Bureau and TCAK produce new precipitation information for Alaska and for the Red Dog Mine area.

10.0 REFERENCES

ADNR (2005). Guidelines for Cooperation with the Alaska Dam Safety Program 30 June, 2005.

Aero-Metric Anchorage (2007). Red Dog Mine - Mine Site (CAD File). Anchorage, Alaska.

American Nuclear Society (1992). 2.8. Determining Design Basis Flooding at Power Reactor Sites.

Anderson, Steve (2008), Personal Communications on Back Dam Design, Golder Associates, Anchorage, Alaska

Barfield, Billy J., R. C. Warner, et al. (1981). Applied hydrology and sedimentology for disturbed areas. Stillwater, Okla. (815 Hillcrest, Stillwater 74074), Oklahoma Technical Press.

Bentley Systems, Inc. 2005 "Flowmaster". Watertown, CT Watertown, CT

Brater, Ernest F. and Horace Williams King (1976). Handbook of Hydraulics, McGraw-Hill, Inc.

Buehler, Bob (1984). "Realistic Assessment of Maximum Flood Potentials." Journal of Hydraulic Engineering 110(8)

Bureau of Reclamation (1987). U.S. Department of the Interior. Design of Small Dams. Third

Bureau of Reclamation (1992). U.S. Department of the Interior. ACER Technical Memorandum 2. Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams.

Chow, Ven Te (1959). Open-channel hydraulics. New York, McGraw-Hill.

Chow, Ven Te (1964). Handbook of Applied Hydrology: A Compendium of Water-Resources Technology. New York, NY, McGraw-Hill, Inc.

Dames & Moore (1986). Proposed Tailing Dam Red Dog Mine Development for Cominco Alaska, Inc. Anchorage, AK June 6, 1986.

Diehl, Joe (2008a), Maximum Wind Speed Verification, Red Dog Mine, AK

Diehl, Joe (2008b), Wind Measurement Information, Red Dog Mine, AK

ESRI. 2006 "Arc Map". 9.2.ESRI.

FEMA (2004). U.S. Department of Homeland Security. Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams.

C:\Documents and Settings\skang\Local Settings\Temporary Internet Files\Content.Outlook\3DNQ4DT1\Report on Preliminary Spillway Design Red Dog Tailings Main Dam Ultimate Closure Configuration-111408.doc

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URS

FHA (2005). U.S. Department of Transportation. Hydraulic Engineering Circular 15. Design of Roadside Channels with Flexible Linings. 3rd

FHA (2006). U.S. Department of Transportation. Hydraulic Design of Energy Dissipators for Culverts and Channels.

FHWA (2005). U.S. Department of Transportation. Hydraulic Engineering Circular No. 15 Design of Roadside Channels with Flexible Linings. Third Edition

FHWA (2006). U.S. Department of Transportation. Hydraulic Engineering Circular No. 14 Hydraulic Design of Energy Dissipators for Culverts and Channels. Third Edition

Haan, C.T. (1977). Statistical Methods in Hydrology. Ames, Iowa, The Iowa State University Press.

Hershfield, D.M. (1961). "Estimating the Probable Maximum Precipitation." Journal of Hydraulics Division, ASCE 87(HY 5)

Kent, Kenneth M. (1972). U.S. Department of Agriculture. National Engineering Handbook Travel Time, Time of Concentration and Lag. Volume 15

Kite, G.W. (1977). Frequency and Risk Analysis in Hydrology. Fort Collins, CO, Water Resources Publications.

Learned, Cheryl (2008), Red Dog Creek Characteristics, Seattle, WA

Lettenmaier, Dennis P. and Stephen J. Burges (1982). "Gumbel's Extreme Value I Distribution: A New Look." Journal of the Hydraulics Division 108(4)

Mills, Thomas R., Michael L. Clar, et al. (1976). Erosion and sediment control : surface mining in the eastern U.S. [Washington, D.C.], The Agency.

National Climatic Data Center (2008). Average Daily Precipitation Data for Kotzebue Ralph Wein Airport, COOPID - 505076, WBANID - 26616, U.S. Department of Commerce.

National Research Council (1985). Safety of Dams: Flood and Earthquake Criteria. National Academic Press.

NOAA (1994). Probable Maximum Precipitation, Pacific Northwest States, Hydrometeorological Report No. 57.

NRC (1977). Design, Construction, and Inspection of Embankment Retention Systems for Uranium Mills.

NRCS (1951). Hydraulics: Uniform Depths and Discharges in Trapezoidal and Rectangular Channels. ES-55, U.S. Department of Agriculture.

NRCS (1956). U.S. Department of Agriculture. Technical Release 2 No. 2. Earth Spillways.

NRCS (1977). U.S. Department of Agriculture. Technical Release No. 25 Design of Open Channels.

C:\Documents and Settings\skang\Local Settings\Temporary Internet Files\Content.Outlook\3DNQ4DT1\Report on Preliminary Spillway Design Red Dog Tailings Main Dam Ultimate Closure Configuration-111408.doc

URS

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NRCS (1983). U.S. Department of Agriculture. Technical Release No. 69 - Riprap for Slope Protection Against Wave Action.

NRCS (1997). U.S. Department of Agriculture. National Engineering Handbook Hydraulics.

NRCS (2004a). U.S. Department of Agriculture. National Engineering Handbook, Part 630, Chapter 10 Estimation of Direct Runoff From Storm Rainfall.

NRCS (2004b). U.S. Department of Agriculture. National Engineering Handbook, Part 630, Chapter 9 Hydrologic Soil-Cover Complexes.

NRCS (2005). Technical Release 60 Earth Dams and Reservoirs. U.S. Department of Agriculture.

NRCS, Conservation Engineering Division (1986). United States Department of Agriculture. Urban Hydrology for Small Watersheds, Technical Release 55.

Ponce, Victor Miguel (1994). Engineering Hydrology: Principles and Practices. Englewood Cliffs, NJ, Prentice-Hall, Inc.

SRK (2007a). Red Dog Mine Closure and Reclamation Plan - Final Draft for State Review. Vancouver, B.C. September 2007.

SRK (2007b). Red Dog Water and Load Balance. Anchorage, AK March 15, 2007.

SRK (2008a). BackDam_Layout_Golder.dwg. Anchorage, AK.

SRK (2008b), Outflow 001 Discharge Monthly Summaries, Monthly Oufall 001 Ratios, SRK, Anchorage, AK

TCAK (2008a). Bons Creek Wind Speed Data - 1996-2007. Bons Creek Wind Speed Data - 1996-2007. Red Dog Mine, AK.

TCAK (2008b). Red Dog Hydrograph: Station 140 - 1996-2007. Hydrological Monitoring Data-Sta 140-Attachment 2K.xls. Red Dog Mine, AK.

Teck Cominco Alaska, Inc (2008). Precipitation Data. Red Dog Precip Records 1991 - 2008. Red Dog Mine, AK.

URS (2005). Periodic Safety Inspection, Report No. 3, Tailings Dam, NID ID# AK 00201, Red Dog Mine, Alaska. Seattle, WA August 5, 2005.

URS (2006a). Geotechnical Data Report, Tailings Main Dam Future Raises to Closure" for Teck Cominco Alaska Inc. February 3, 2006.

URS (2006b). Geotechnical Investigation Report Red Dog Tailings Main Dam Future Raises to Closure Red Dog Mine, Alaska. Seattle, WA July 28, 2008.

C:\Documents and Settings\skang\Local Settings\Temporary Internet Files\Content.Outlook\3DNQ4DT1\Report on Preliminary Spillway Design Red Dog Tailings Main Dam Ultimate Closure Configuration-111408.doc

URS

36

URS (2007). Preliminary Conceptual Design Report Red Dog Tailings Main Dam Future Raises to Closure, Red Dog Mine, Alaska. Seattle, WA November 26, 2007.

URS (2008a). Hydrology and Hydraulics Mine Water Diversion System, Red Dog Mine, Alaska. Seattle, WA February 28, 2008.

URS (2008b). Preliminary Design Report, Ultimate Closure Configuration, Red Dog Tailings Main Dam, Red Dog Mine, Alaska. Seattle, WA (in preparation).

USACE (1992). U.S. Department of Army. Electronic Manual 1110-2-1603. Hydraulic Design of Spillways.

USACE (1993). EM 1110-2-1415. Engineering Design - Hydrologic Frequency Analysis.

USACE (1994). EM 1110-2-1601. Hydraulic Design of Flood Control Channels.

USACE (1997). Department of the Army. EM 1110-2-1420. Hydrologic Engineering Requirements for Reservoirs.

USACE. 2001 "Hydrologic Modeling System HEC-HMS". Hydrologic Engineering Center.

USACE (2003). U.S. Department of Army. EM 1110-2-1100. Coastal Engineering Manual.

USACE. 2004 "Hydrologic Engineering Center – River Analysis System (HEC-RAS)". Davis, CA Davis, CA

USACE (2006). EM 1110-2-1100. Coastal Engineering Manual.

USBR (1984). U.S. Department of the Interior. A Water Resources Technical Publication, Engineering Monograph No. 25 Hydraulic Design of Stilling Basins and Energy Dissipators.

USBR (1987). U.S. Department of the Interior. Design of Small Dams. Third Edition

USBR (1992). U.S. Department of the Interior. ACER Technical Memorandum 2. Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams.

Wanielista, Martin, Robert Kersten, et al. (1997). Hydrology: Water Quantity and Quality Control. Toronto, Canada, John Wiley & Sons, Inc.

Washington State Department of Ecology (2001). Stormwater Management Manual for Western Washington, Hydrologic Analysis and Flow Control Design/BMPs.

Weakley, Jason (2008), Verification of Water Treatment Plant Flowrates, TCAK, Red Dog Mine, AK

Weather Bureau (1961). U.S. Department of Commerce. Technical Report 40. Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years. U.S Government Printing Office.

C:\Documents and Settings\skang\Local Settings\Temporary Internet Files\Content.Outlook\3DNQ4DT1\Report on Preliminary Spillway Design Red Dog Tailings Main Dam Ultimate Closure Configuration-111408.doc

URS

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Weather Bureau (1963). U.S. Department of Commerce. Technical Paper 47. Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska for Areas to 400 Square Miles, Duration to 24 Hours, and Return Periods from 1 to 100 Years. U.S. Government Printing Office.

West Consultants Inc. 2005 "RIPRAP Design Systems". Version 3.0.San Diego, CA San Diego, CA

World Meteorological Organization (WMO) (1973). Manual for Estimation of Probable Maximum Precipitation. Geneva, Switzerland. WMO-No. 332

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FIGURES

Catchments at Closure	mi²
Arctic Tundra Natural Ground	0.48
Tailings Impoundment	1.18
Main Waste Stockpile	0.86
Total	2.52

Tailings Impoundment

Mill and Concentrate Facility

Open Pit Mine
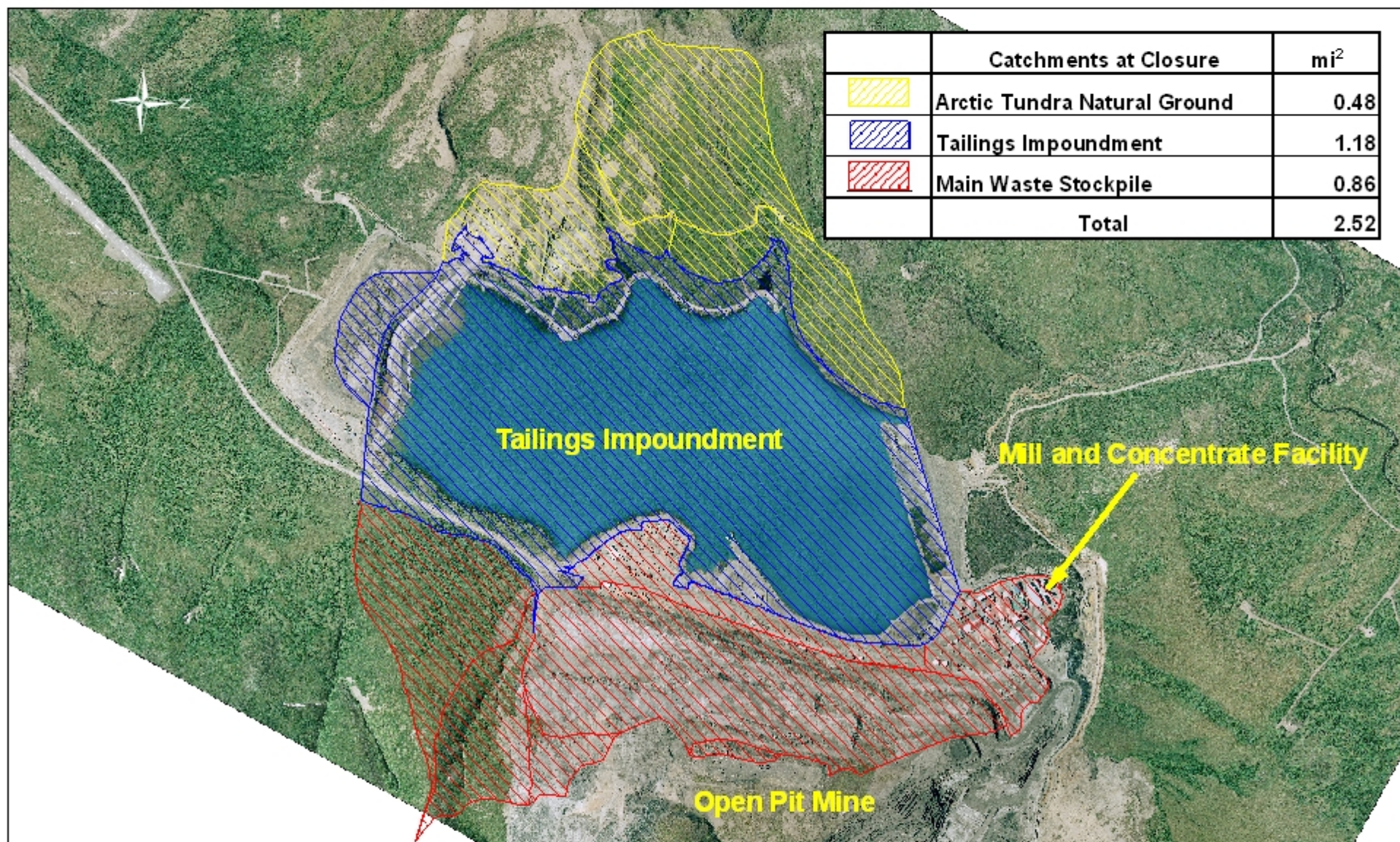
Figure 5-1
Tailings Pond Catchment Areas

Job No. 33760401.00001

Preliminary Spillway Design
Red Dog Tailings Main Dam,
Ultimate Closure Configuration
Red Dog Mine, AK

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0   950 1,900   3,800   5,700   7,600
Feet

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Legend

— Open Channel Flow

— Shallow Concentrated Flow

— Sheet Flow

0	550	1,100	2,200	3,300	4,400

Feet

Figure 6-1
Time of Concentration Path

Preliminary Spillway Design
Red Dog Tailings Main Dam
Ultimate Closure Configuration
Red Dog Mine, AK

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DRAWINGS

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PRELIMINARY DESIGN DRAWINGS
FOR
TECK COMINCO ALASKA INC.
RED DOG MINE TAILINGS MAIN DAM, ULTIMATE CLOSURE CONFIGURATION
RED DOG MINE, ALASKA

DRAWING INDEX

DRAWING NO.	REV. NO.	TITLE
TCP−1	0	TITLE SHEET, INDEX, MAP, REFERENCES, SPECIFICATIONS AND NOTES
TCP−2	0	LEGEND AND ABBREVIATIONS
TCP−3	0	EXISTING STAGE VIII DAM PLAN AND PROFILE
TCP−4	0	EXISTING STAGE VIII DAM HIGHEST CROSS SECTION
TCP−5	0	DAM AND SPILLWAY PLAN AT CLOSURE
TCP−6	0	CLOSURE EMBANKMENT PLAN AND PROFILE
TCP−7	0	SUBSURFACE PROFILE EMBANKMENT
TCP−8	0	CROSS SECTIONS−TAILINGS MAIN DAM
TCP−9	0	CROSS SECTIONS−TAILINGS MAIN DAM
TCP−10	0	WING WALL PLAN
TCP−11	0	CROSS SECTIONS−EMBANKMENT TO WING WALL BEND
TCP−12	0	CROSS SECTIONS−WING WALL
TCP−13	0	CURTAIN WALL OPTION 6 AND 6A−PLANS
TCP−14	0	CURTAIN WALL OPTION 6 AND 6A−PROFILES
TCP−15	0	SPILLWAY PLAN AND PROFILE
TCP−16	0	SPILLWAY SECTIONS AND DETAILS
TCP−17	0	SPILLWAY STILLING BASIN AND REACH 1 PROFILE
TCP−18	0	SPILLWAY REACH 2 PROFILE
TCP−19	0	SPILLWAY REACH 3, 4 AND 5 PROFILES
TCP−20	0	SEEPAGE CONTROL BEACH AND BERM

VICINITY MAP

RED DOG MINE
KIVALINA
PORT SITE
CHUKCHI SEA
KOTZEBUE SOUND
KOTZEBUE

REFERENCE DRAWINGS

CONSTRUCTION DRAWINGS, STAGE VIII-RAISE, RED DOG TAILINGS MAIN DAM, BY URS FOR TECK COMINCO ALASKA, INC., RED DOG MINE, 2008. SEE NOTE 12.

DRAWING NO.	REV. NO.	TITLE
T8R−1	2	TITLE SHEET, INDEX AND VICINITY MAP
T8R−2	0	EXISTING TAILINGS MAIN DAM PLAN AND PROFILE
T8R−3	0	LAYOUT−TAILINGS MAIN DAM
T8R−4	1	PLAN AND PROFILE−EMBANKMENT
T8R−5	0	PLAN AND PROFILE−WING WALL
T8R−6	0	SUBSURFACE PROFILE−EMBANKMENT
T8R−7	0	SUBSURFACE PROFILE−WING WALL
T8R−8	1	CROSS SECTIONS−TAILINGS MAIN DAM
T8R−9	1	CROSS SECTIONS−TAILINGS MAIN DAM
T8R−10	1	CROSS SECTIONS−EMBANKMENT TO WING WALL BEND
T8R−11	1	CROSS SECTIONS−WING WALL
T8R−12	1	LEFT ABUTMENT CUTOFF WALL DETAILS
T8R−13	0	CURTAIN WALL OPTIONS
T8R−14	1	MEMBRANE WALL EXTENSION SECTIONS AND DETAILS
T8R−15	1	SLURRY WALL SECTIONS AND DETAILS
T8R−16	2	LINER SECTIONS AND DETAILS
T8R−17	0	EMBANKMENT TO WING WALL 3D VIEW
T8R−18	1	BEND LINER SECTIONS AND DETAILS

PRELIMINARY SPECIFICATIONS

URS "TECHNICAL SPECIFICATIONS, ULTIMATE PRELIMINARY CLOSURE, TAILINGS MAIN DAM, RED DOG MINE, ALASKA" FOR TECK COMINCO ALASKA, INC., AUG. 22, 2008.

GENERAL NOTES

1. THESE DRAWINGS ARE PRELIMINARY DESIGN DRAWINGS THAT ARE PART OF A PRELIMINARY DAM DESIGN REPORT AND A PRELIMINARY SPILLWAY DESIGN REPORT FOR THE TAILINGS MAIN DAM ULTIMATE CLOSURE CONFIGURATION. THESE DRAWINGS ARE NOT FOR CONSTRUCTION AND ARE TO BE USED IN THE CONTEXT OF THE PRELIMINARY DESIGN REPORTS. FINAL DESIGN DRAWINGS WILL BE PREPARED FOR THE APPROVAL TO CONSTRUCT EACH FUTURE RAISE.

2. THE TOPOGRAPHIC MAP WAS COMPILED TO MEET HORIZONTAL AND VERTICAL ACCURACY IN ACCORDANCE WITH AMERICAN SOCIETY FOR PHOTOGRAMMETRY AND REMOTE SENSING (ASPRS) CLASS I STANDARDS. AREAS DENOTING VEGETATION COVER ON THE GROUND SHOULD BE CONSIDERED LESS ACCURATE AND NOT USED FOR CONSTRUCTION UNTIL FIELD CHECKED IN ACCORDANCE WITH ASPRS STANDARDS. THIS MAP PROJECTION IS BASED UPON NORTH AMERICAN DATUM OF 1927 (NAD27), ALASKA STATE PLANE ZONE 7 AS EXPRESSED IN U.S. SURVEY FEET.

3. FOR THE PRELIMINARY DESIGN, THE FUTURE RAISES OF THE TAILINGS MAIN DAM TO ULTIMATE CLOSURE WILL CONSIST OF SEVERAL INTERIM RAISES TO THE ULTIMATE CREST ELEVATION. THE NUMBERS AND HEIGHTS OF INTERIM RAISES WILL DEPEND ON MINE OPERATION NEEDS. THE NUMBER OF INTERIM RAISES AND THEIR HEIGHTS HAVE NO IMPACT ON THE PRELIMINARY DESIGN INTENT FOR THE ULTIMATE CLOSURE CONFIGURATION AND MINIMAL IMPACT ON THE PRELIMINARY DESIGN DETAILS.

4. THE FUTURE RAISES AND FINAL DAM CONFIGURATION WILL INCLUDE THE EMBANKMENT AND A WING WALL OUT FROM THE RIGHT ABUTMENT OF THE EMBANKMENT. THE EMBANKMENT AND WALL WILL BE OF SIMILAR CONSTRUCTION AND WILL COMPRISE ONE DAM WITH CONTINUOUS SEEPAGE AND DRAINAGE CONTROL SYSTEMS AND A BEND AT THE EMBANKMENT TO WALL CONNECTION. THERE ARE SEVERAL POSSIBLE BEND AND WALL ALIGNMENTS AND CONFIGURATIONS THAT WILL BE RESOLVED IN THE FINAL DESIGNS OF EACH RAISE.

5. EACH RAISE WILL REQUIRE THE FOLLOWING WORK:

   a) WIDEN THE EXISTING DAM TO PROVIDE A FOOTPRINT FOR THE RAISE, CONSTRUCT THE RAISE TO THE CREST ELEVATION AND EXTEND THE LINER SYSTEM ON THE UPSTREAM PART OF THE RAISE.
   b) EXTEND THE PRIMARY SEEPAGE CONTROL SYSTEM BY EXTENDING THE EXISTING SEEPAGE CUTOFF SYSTEM ALONG THE WEST ABUTMENT OF THE MAIN DAM EMBANKMENT AND THE CURTAIN WALL ALONG THE EAST EXTENSION OF THE WING WALL.

6. ALL CONSTRUCTION MATERIAL SOIL TYPES ON THE DOWNSTREAM SIDE OF THE DAM AND ABOVE THE TOP ELEVATION OF THE LINER SHALL BE NON−"MINERALIZED". "MINERALIZED" MATERIAL CAN BE USED FOR CONSTRUCTION ON THE UPSTREAM SIDE OF THE LINER BELOW THE TOP ELEVATION OF THE LINER.

7. THE CURTAIN WALL CONSTRUCTION WILL BE IN A VERTICAL CUT AND WILL CONSIST OF EITHER A MEMBRANE WALL WITH THE BOTTOM EMBEDDED IN CONTROL DENSITY FILL, OR A SLURRY WALL.

8. ALL CONSTRUCTION MATERIAL SOIL TYPES ON THE DOWNSTREAM SIDE OF THE DAM SHALL BE NON−"MINERALIZED".

9. THE SAME SOIL, ROCK AND GEOSYNTHETIC MATERIALS WERE USED IN THE DAM DURING ALL PREVIOUS STAGES OF CONSTRUCTION AND WERE ASSUMED FOR THE FUTURE RAISES IN THE CONCEPTUAL DESIGN. HOWEVER, THE FINAL DESIGN OF EACH RAISE WILL INVESTIGATE IF OTHER MATERIALS ARE MORE APPROPRIATE, ESPECIALLY FOR GEOSYNTHETICS WHERE TECHNOLOGY HAS ADVANCED AND NEW PRODUCTS MAY BE MORE SUITABLE.

10. THE LINER SYSTEM HAS BEEN MODIFIED FROM THE PREVIOUS DESIGNS UP TO STAGE VII−B BY CHANGING THE GRADATIONS OF SOIL TYPE 3 AND 4 TO MEET FILTER CRITERIA AND ELIMINATING THE GEOTEXTILES. THE GEOTEXTILES ARE NOT NEEDED BECAUSE OF THE GRADATION CHANGES.

11. HOWEVER, THE GEOTEXTILES ARE SHOWN ON THE DRAWING AND SPECIFICATIONS IN THE EVENT THAT THEY MAY BE NEEDED IF THE FILTER CRITERIA CANNOT BE MET. THIS DETERMINATION WILL BE MADE BY THE CQA PROJECT DIRECTOR, CQA FIELD MANAGER, AND CONSTRUCTION MANAGER.

12. FOR PURPOSES OF THESE ULTIMATE CLOSURE CONFIGURATION DRAWINGS, THE STAGE VIII RAISE IS THE "EXISTING" CONDITION EVEN THOUGH STAGE VIII CONSTRUCTION IS CURRENTLY UNDERWAY AND IS PLANNED TO BE COMPLETED IN 2009. THE STAGE VIII AS−BUILT DRAWINGS WILL BE PROVIDED IN THE STAGE VIII CONSTRUCTION COMPLETION REPORT THAT WILL BE PREPARED AFTER STAGE VIII IS CONSTRUCTED.

REFERENCES TITLE	REFERENCES TITLE	REVISIONS			REVISIONS		
KR/AIC STAGE VII−B CONSTRUCTION SURVEY 2007		NO. BY DATE DESCRIPTION			NO. BY DATE DESCRIPTION		
AERO−METRIC ANCHORAGE SURVEY 2007							
DAMES & MOORE, 1987 (TOPOGRAPHY)							
DAMES & MOORE, 1984 (TOPOGRAPHY)							

STATE OF ALASKA 49TH
CECIL M. URLICH
CE−11095
REGISTERED PROFESSIONAL ENGINEER

SCALE:	AS NOTED	INIT DATE
DESIGNED BY:	C. LEARNED	08−22−08
DRAWN BY:	T. PHAM	08−22−08
CHECKED BY:	DENHERDER C. LEARNED	08−22−08
APPROVED BY:	C. URLICH	08−22−08
CLIENT APPROVAL BY:		

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TECK COMINCO ALASKA INC.
RED DOG MINE TAILINGS MAIN DAM CLOSURE
TITLE SHEET, INDEX, MAP, REFERENCES, SPECIFICATIONS AND NOTES

JOB NO.	33760401
DRAWING NO.	TCP−1
REV.	0

LEGEND

PLAN

- CONTOURS (EXISTING)
  - MAJOR
  - MINOR — *350*
- CONTOURS (NEW)
  - MAJOR
  - MINOR — *350*
- LINER (EXISTING)
- LINER (NEW)
- LAKE OR WATER LINE
- CREST OF DAM
- PUMPBACK PIPELINE/ PIPELINE (EXISTING)
- ROADWAY
- DAM CREST
- DOWNSTREAM SLOPE EXTENT
- HISTORIC DRAINAGE
- FLOW PATH
- CUTOFF WALL/CURTAIN WALL
- UPSTREAM SLOPE EXTENT

SECTION

- ORIGINAL GROUND SURFACE (1984)
- GROUND SURFACE 1987
- GROUND SURFACE 2007
- GOUGE OR CLAY ZONE
- REPRESENTATIVE BEDDING
- ESTIMATED PROFILE
- EXISTING LINER SYSTEM
- PROPOSED LINER SYSTEM
- GROUND (EXISTING)
- HIGHLY WEATHERED SHALE
- SLIGHTLY TO MODERATELY WEATHERED SHALE
- GRAVEL FILL
- SILTY SAND OR CLAYEY SILT WITH GRAVEL
- SILTY GRAVEL, COBBLES AND BOULDERS (ALLUVIUM)
- ROCK DRAIN
- RIP RAP
  - TYPE I
  - TYPE II
- ROCK FILL/UNDER DRAIN
- PERMAFROST IN 2005
- TUNDRA ORGANIC ROOTMAT
- NEW DAM RAISE
- CONCRETE BLOCKS
- POSSIBLE WASTE FILL
- MODERATELY WEATHERED SHALE LENS EMBEDDED IN RESIDUAL TO HIGHLY WEATHERED SHALE

LOCATION OF BORINGS & TEST PITS

- DAMES & MOORE FOR TAILINGS DAM (1982)
- DAMES & MOORE FOR TAILINGS DAM (1984)
- DAMES AND MOORE FOR WASTE STORAGE (1984 & 1987)
- EBA ENGINEERING FOR MILL FACILITIES (2000)
- PND FOR RIGHT ABUTMENT WING WALL (2004)
- DAMES & MOORE FOR MILL FACILITIES (1982, 1985)
- URS WITH PIEZOMETERS (2005)
- THERMISTORS (PRE-2005)
- URS WITH PIEZOMETERS & THERMISTORS (2005)
- URS WITH PIEZOMETERS (2005)
- URS WITH NO INSTRUMENTATION (2005-2006)
- URS BORING (2007)
- URS BORING (2008)
- URS WASTE FILL INVESTIGATION TEST PIT 2008

INSTRUMENTATION

KEY:

- GROUND SURFACE
- SOIL
- HIGHLY WEATHERED BEDROCK
- BEDROCK

- SURFACE ELEVATION
- TIP ELEVATION

BORING   PIEZOMETER/THERMISTOR

SECTION AND DETAIL DESIGNATION

POINT AT WHICH SECTION/PROFILE IS TAKEN

LETTER OR NUMBER INDICATES DESIGNATION OF SECTION/PROFILE

C   TCP-9

C'   TCP-9

INDICATES DRAWING WHERE SECTION/PROFILE IS SHOWN

REFERENCE TO DETAILS AND SECTIONS

POINT AT WHICH DETAIL IS TAKEN AND SECTION OR DETAIL IS SHOWN

LETTER OR NUMBER INDICATES DESIGNATION OF DETAIL OR SECTION

SCALE   **NAME**   1
        SCALE: 1"=30'   T8R-5

INDICATES DRAWING WHERE DETAIL OR SECTION IS REFERRED

ARE ALSO DESIGNATED BY X/A-Y, WHERE X IS A NUMBER FOR DETAILS AND A LETTER FOR SECTIONS, AND A-Y IS THE DRAWING ON WHICH THE SECTION OR DETAIL APPEARS

EMBANKMENT CONFIGURATION

STAGE	CREST ELEVATION
I	865'
II	890'
III	910'
IV	925'
V	940'
VI	950'
VII-A	955'
VII-B	960'
VIII-W	960'
VIII-R	970'
CLOSURE	986'

SOIL TYPE	MATERIAL
①	RANDOM ROCKFILL (24 INCH)
③	PROCESSED SELECT (3 INCH)
④	PROCESSED SELECT (3/8 INCH)
⑦	SELECT (12 INCH)
⑨	RANDOM MINEWASTE (12 INCH MAX)

GEOSYNTHETICS
GEOMEMBRANE - 100 MIL HDPE
GEOTEXTILE - 10 OZ AND 16 OZ NONWOVEN (IF NEEDED)

ABBREVIATIONS

DRAWINGS

T7B	TAILINGS MAIN DAM STAGE VII-B
T8W	TAILINGS MAIN DAM STAGE VIII WIDENING
T8R	TAILINGS MAIN DAM STAGE VIII RAISE
TCP	TAILINGS MAIN DAM CLOSURE PRELIMINARY

DIRECTIONS

N	NORTH
S	SOUTH
E	EAST
W	WEST
NW	NORTHWEST
NE	NORTHEAST
SW	SOUTHWEST
SE	SOUTHEAST

DRAWING LABELS

TDAM	TAILINGS MAIN DAM
P	PIEZOMETER
T	THERMISTOR
TP	TEST PIT
BH	BORE HOLE
D_{50}	GRAIN DIAMETER (IN MM) CORRESPONDING TO 50% PASSING BY WEIGHT (OR MASS)

SHORTENED WORDS

APPROX.	APPROXIMATE(LY)
CDF	CONTROLLED DENSITY FILL
EL	ELEVATION
EST	ESTIMATED
EXIST	EXISTING
FT	FOOT (FEET)
HDPE	HIGH DENSITY POLYETHYLENE
HOR (H)	HORIZON
INC	INCORPORATED
LTD	LIMITED
MAX	MAXIMUM
MIN	MINIMUM
NO	NUMBER
NTS	NOT TO SCALE
OZ	OUNCE
REV	REVISION
STA	STATION
VER (V)	VERTICAL

TEXT SYMBOLS

#	NUMBER
%	PERCENT
Ⅷ	STAGE OF DESIGN

PLAN

SCALE

0   150   300

FEET

REFERENCES			REVISIONS					REVISIONS									
TITLE				NO.	BY	DATE	DESCRIPTION		NO.	BY	DATE	DESCRIPTION					
KR/AIC STAGE VII-B CONSTRUCTION SURVEY 2007																	
AERO-METRIC ANCHORAGE SURVEY 2007																	
DAMES & MOORE, 1987 (TOPOGRAPHY)																	
DAMES & MOORE, 1984 (TOPOGRAPHY)																	

TECK COMINCO ALASKA INC.

RED DOG MINE TAILINGS MAIN DAM CLOSURE

DAM AND SPILLWAY PLAN AT CLOSURE

	INIT	DATE
SCALE: AS NOTED		
DESIGNED BY: J.R. SUGALSKI		08-22-08
DRAWN BY: T. PHAM		08-22-08
CHECKED BY: DENHERDER C. LEARNED		08-22-08
APPROVED BY: C. URLICH		08-22-08
CLIENT APPROVAL BY:		

URS
1501 4th Avenue, Ste. 1400
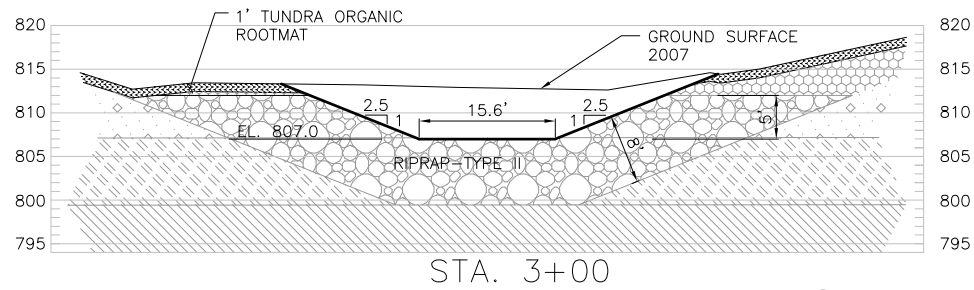Seattle, WA 98101-1616
Telephone (206) 438-2700

JOB NO. 33760401

DRAWING NO. TCP-5   REV. 0

STATE OF ALASKA
49TH
CECIL M. URLICH
CE-11095
REGISTERED PROFESSIONAL ENGINEER

Plan view labels:

- FIELD-FITTED ROAD TO ACCESS WEST SIDE OF TAILINGS IMPOUNDMENT
- SLOPE TOE 2.5H: 1V (FROM STA. 13+50 TO 19+05)
- SLOPE TOE 1.5H: 1V (FROM STA. 10+49 TO 13+50)
- SLOPE TOE 2.5H: 1V (FROM STA. 0+00 TO 10+49)
- FLAT AREA EL. 979.0
- D TCP-16
- C TCP-16
- B TCP-16
- A TCP-16
- SPILLWAY CONTROL SECTION EL. 983.5
- ALP-97-4
- SS-02-05
- SS-21-05
- CLOSURE CUTOFF WALL
- SS-22-05
- TP-01-05
- SS-03-05
- SPILLWAY ENTRANCE EL. 979
- EL. 979
- SS-01-05
- SPILLWAY OUTLET TO RED DOG CREEK
- SPILLWAY BERM EL. 979
- ALP-97-2
- SS-6-84
- CLOSURE DAM CREST EL. 986
- SS-5-84
- P-96-004 T-95-004
- EXISTING CUTOFF WALL
- CLOSURE DOWNSTREAM SLOPE TOE 2.5H:1V
- ALP-97-3
- TAILINGS BERM EL. 983.5
- FLOW
- W E N S

SCALE
0 100 200
FEET

Profile view labels:

- EL. 979 @19+05
- EL. 979 @17+09
- EL. 983.5 @14+60
- EL. 983.5 @14+30
- EL. 965 @9+29
- PT @8+70
- PC @4+64
- EL. 790 @2+32
- EL. 775 @0+28
- RED DOG CREEK
- REACH 5
- REACH 4
- REACH 3
- REACH 2
- REACH 1
- LEVEL CONTROL SECTION
- SS-21-05
- SS-02-05
- SS-22-05
- GROUND SURFACE 2007
- 1' TUNDRA ORGANIC ROOTMAT
- SLOPE 0.0%
- SLOPE 1.8%
- BOTTOM OF OUTLET CHANNEL
- SLOPE 3.7%
- SILTY SAND OR CLAYEY SILT WITH GRAVEL
- TP-01-05
- HIGHLY WEATHERED SHALE
- SS-03-05
- SLOPE 25.14%
- SLIGHTLY TO MODERATELY WEATHERED SHALE
- STILLING BASIN, FOR DETAIL SEE F TCP-17

HORIZ. & VER. SCALE:
0 100 200
FEET

STA. 3+00
SPILLWAY OUTLET
25% SLOPE CHANNEL CROSS SECTION
(A) TCP-15

Labels: 1' TUNDRA ORGANIC ROOTMAT; GROUND SURFACE 2007; EL. 807.0; RIPRAP-TYPE II; 2.5 / 1; 15.6'; 2.5 / 1
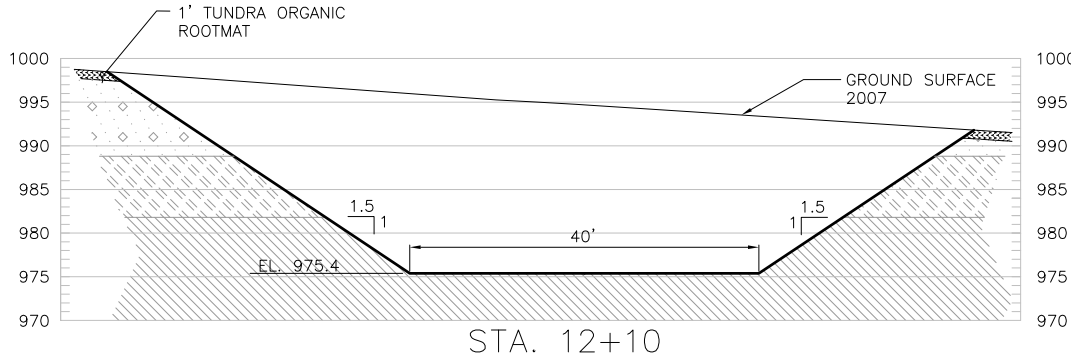
STA. 12+10
SPILLWAY OUTLET
3.7% SLOPE CHANNEL CROSS SECTION
(B) TCP-15

Labels: 1' TUNDRA ORGANIC ROOTMAT; GROUND SURFACE 2007; EL. 975.4; 40'; 1.5 / 1; 1.5 / 1
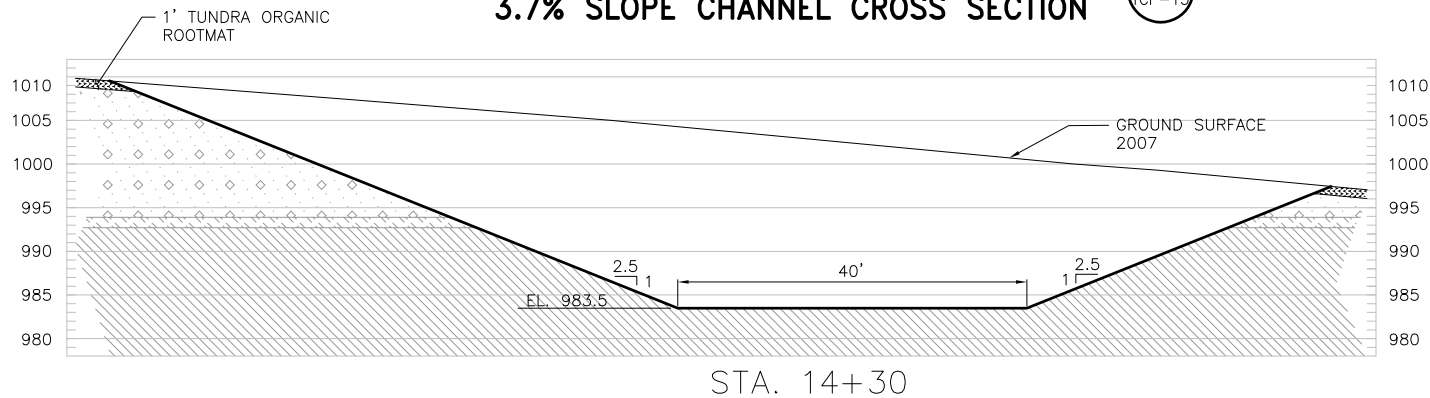
STA. 14+30
SPILLWAY OUTLET CHANNEL
BROAD-CRESTED WEIR CROSS SECTION
(C) TCP-15

Labels: 1' TUNDRA ORGANIC ROOTMAT; GROUND SURFACE 2007; EL. 983.5; 40'; 2.5 / 1; 2.5 / 1
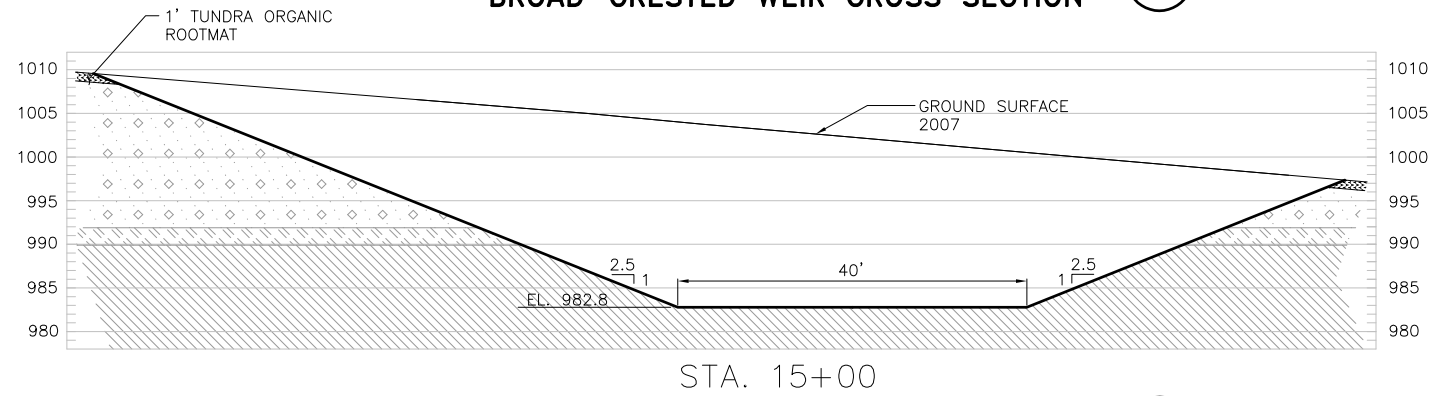
STA. 15+00
SPILLWAY UPSTREAM CHANNEL CROSS SECTION
(D) TCP-15

Labels: 1' TUNDRA ORGANIC ROOTMAT; GROUND SURFACE 2007; EL. 982.8; 40'; 2.5 / 1; 2.5 / 1

HOR. & VER. SCALE
0 ... 10 ... 20
FEET
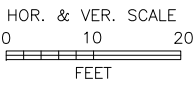
STA.	TOP OF OUTLET CHANNEL (NATURAL GROUND) EL. (FT)	BOTTOM OF OUTLET CHANNEL EL. (FT)	DIFFERENCE IN EL. (FT)	BREAK POINT
0+28	775.0	775.0	0.0	END OUTLET CHANNEL
0+85	778.8	775.0	3.8	END ROCK FILL
1+46	784.8	775.0	9.8	END SLOPE STILLING BASIN
1+54	785.8	771.0	14.8	BOTTOM STILLING BASIN EXIT
1+94	791.5	771.0	20.5	BOTTOM STILLING BASIN ENTRANCE
2+32	797.3	790.0	7.3	BEGINNING SLOPE STILLING BASIN
3+00	812.9	807.0	5.9	CROSS SECTION A
4+64	865.0	848.2	16.8	PC (POINT CURVE)
8+70	961.5	952.8	8.7	PT (POINT TANGENT)
12+10	994.7	975.4	19.3	CROSS SECTION B
14+30	1002.4	983.5	18.9	CROSS SECTION C
14+60	1002.6	983.5	19.1	PC (POINT CURVE)
15+00	1002.3	982.8	19.5	CROSS SECTION D
19+05	979.0	979.0	3.5	START OF INLET CHANNEL

NOTE:

1. EROSION PROTECTION IS NOT REQUIRED ABOVE FLOW DEPTH AND FREEBOARD.

SOIL TYPE	MATERIAL DESCRIPTION
RIPRAP	SELECT MATERIAL CONSISTING OF BLASTED ROCK. THE ROCK SHALL BE DURABLE, BLOCKY, ANGULAR, AND HAVE A MAXIMUM LENGTH TO WIDTH RATIO OF 2. FLAT OR ELONGATED ROCK SHALL NOT BE USED.

TYPE I — ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO TWO (2) FEET.

TYPE II — ROCK DIAMETERS SHALL BE WITH D_{50} EQUAL TO FOUR (4) FEET.

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PLAN

2.5:1

2.5:1

SLOPE TOE 2.5H:1V

40'
STILLING BASIN

2.5:1

12'

2.5:1

28'

FLOW

EXISTING RED DOG CREEK

2.5:1

2.5:1

2.5:1

RIPRAP–TYPE II

EL. 790 @2+32

GROUND SURFACE 2007

EL. 771 @1+94.4

CONCRETE BLOCKS

EL. 771 @1+54.4
EL. 775 @1+46.4

RIPRAP–TYPE I

EL. 775 @0+85

EL. 775 @0+28

RED DOG CREEK

— 800

— 790

3
4

2
1

40'
STILLING BASIN

2
1

1
2

S=0%

— 780

BOTTOM OF OUTLET CHANNEL

— 770

— 760

— 750

2+40 2+30 2+20 2+10 2+00 1+90 1+80 1+70 1+60 1+50 1+40 1+30 1+20 1+10 1+00 0+90 0+80 0+70 0+60 0+50 0+40 0+30 0+20 0+10 0+00

PROFILE

STILLING BASIN F / TCP-15

HORIZ. & VER. SCALE:

0   10   20
FEET

REFERENCES		REFERENCES			REVISIONS				REVISIONS	
TITLE		TITLE	NO.	BY	DATE	DESCRIPTION	NO.	BY	DATE	DESCRIPTION
KR/AIC STAGE VII–B CONSTRUCTION SURVEY 2007										
AERO–METRIC ANCHORAGE SURVEY 2007										
DAMES & MOORE, 1987 (TOPOGRAPHY)										
DAMES & MOORE, 1984 (TOPOGRAPHY)										

STATE OF ALASKA
49TH
CECIL M. URLICH
CE–11095
REGISTERED PROFESSIONAL ENGINEER

	SCALE:	AS NOTED	INIT	DATE
	DESIGNED BY:	T. CLARKIN		08–22–08
	DRAWN BY:	T. PHAM		08–22–08
	CHECKED BY:	DENHERDER C. LEARNED		08–22–08
	APPROVED BY:	C. URLICH		08–22–08
	CLIENT APPROVAL BY:			

URS
1501 4th Avenue, Ste. 1400
Seattle, WA 98101–1616
Telephone (206) 438–2700

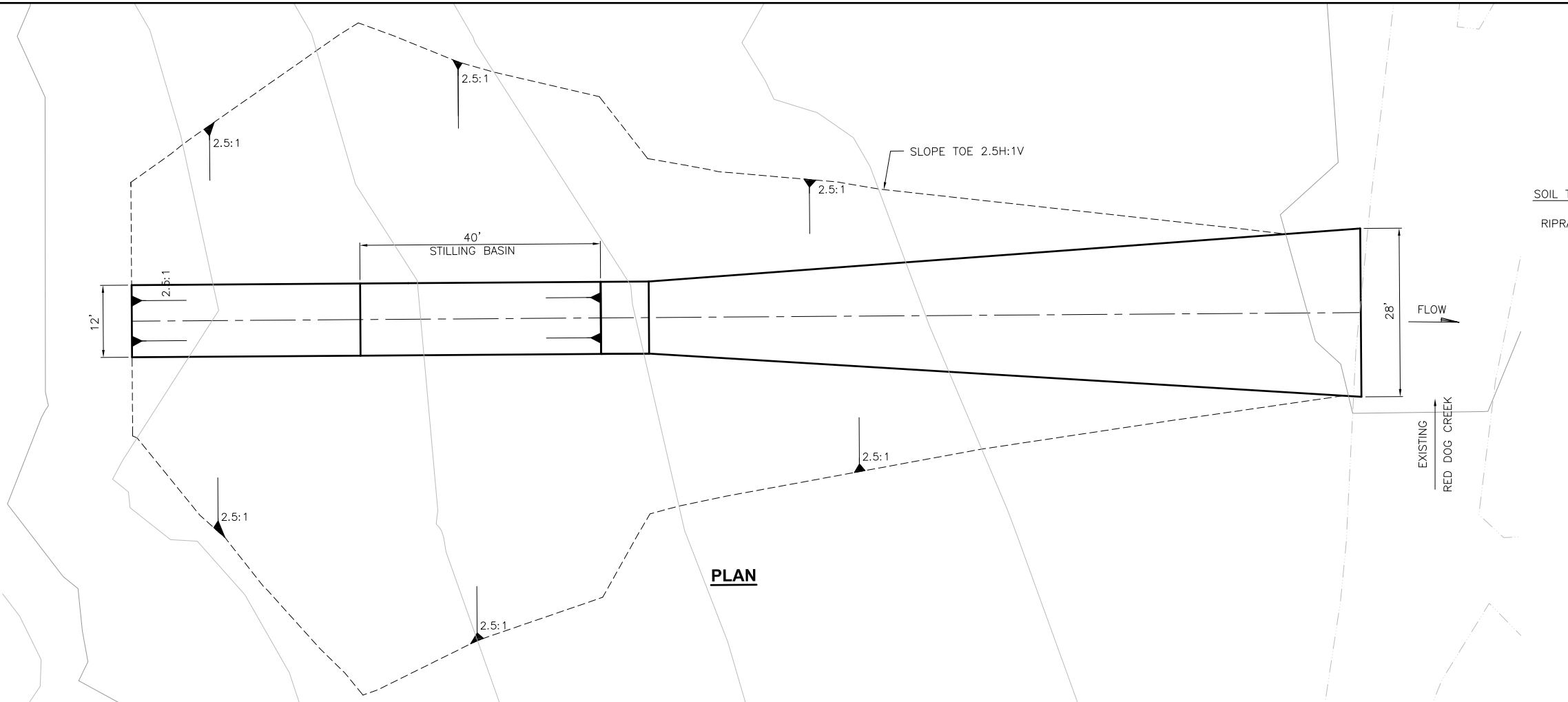## TECK COMINCO ALASKA INC.

RED DOG MINE TAILINGS MAIN DAM CLOSURE

SPILLWAY STILLING BASIN
AND REACH 1 PROFILE

JOB NO. 33760401

DRAWING NO. TCP–17   REV. 0

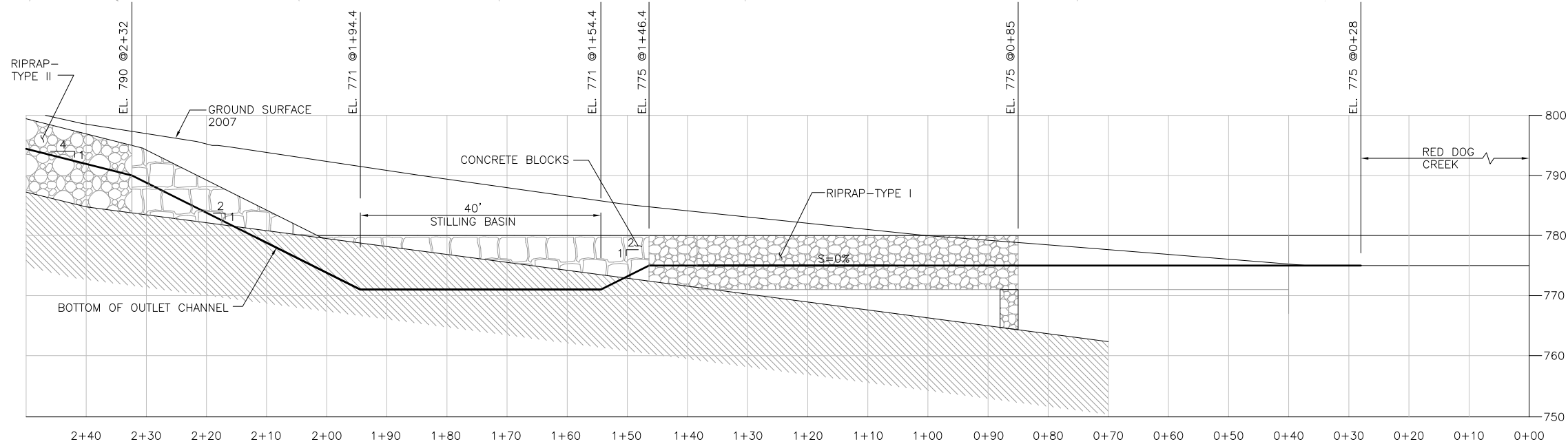P:\ACAD\PROJECT\TeckComincoAlaska\Red_Dog_Tailings_Dam_Closure_Design\R06\TCP–17.dwg Nov 14, 2008 – 2:26pm

Upper profile labels:
- REACH 2
- 920, 880, 840, 800, 760
- MATCH LINE
- SLOPE 25.14%
- TP-01-05
- PC @4+63.87
- GROUND SURFACE 2007
- 1' TUNDRA ORGANIC ROOTMAT
- BOTTOM OF OUTLET CHANNEL
- EL. 790 @2+32
- 6+00, 5+00, 4+00, 3+00

Lower profile labels:
- REACH 2
- 1000, 960, 920, 880
- EL. 965 @9+29
- PT @8+70.33
- 1' TUNDRA ORGANIC ROOTMAT
- GROUND SURFACE 2007
- BOTTOM OF OUTLET CHANNEL
- MATCH LINE
- 9+00, 8+00, 7+00, 6+00

PROFILE SPILLWAY – REACH 2
FROM STATION 2+32 TO 9+29
TCP-15

HORIZ. & VER. SCALE:
0   20   40
FEET

NOT FOR CONSTRUCTION

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PROFILE SPILLWAY – REACH 3, 4
FROM STA. 9+29 TO 14+60

REACH 4

REACH 3

LEVEL CONTROL SECTION 30'

SLOPE 0.0%

SS-02-05

GROUND SURFACE 2007

1' TUNDRA ORGANIC ROOTMAT

SS-22-05

BOTTOM OF OUTLET CHANNEL

SLOPE 3.7%

EL. 983.5 @14+60

EL. 983.5 @14+30

EL. 965 @9+29
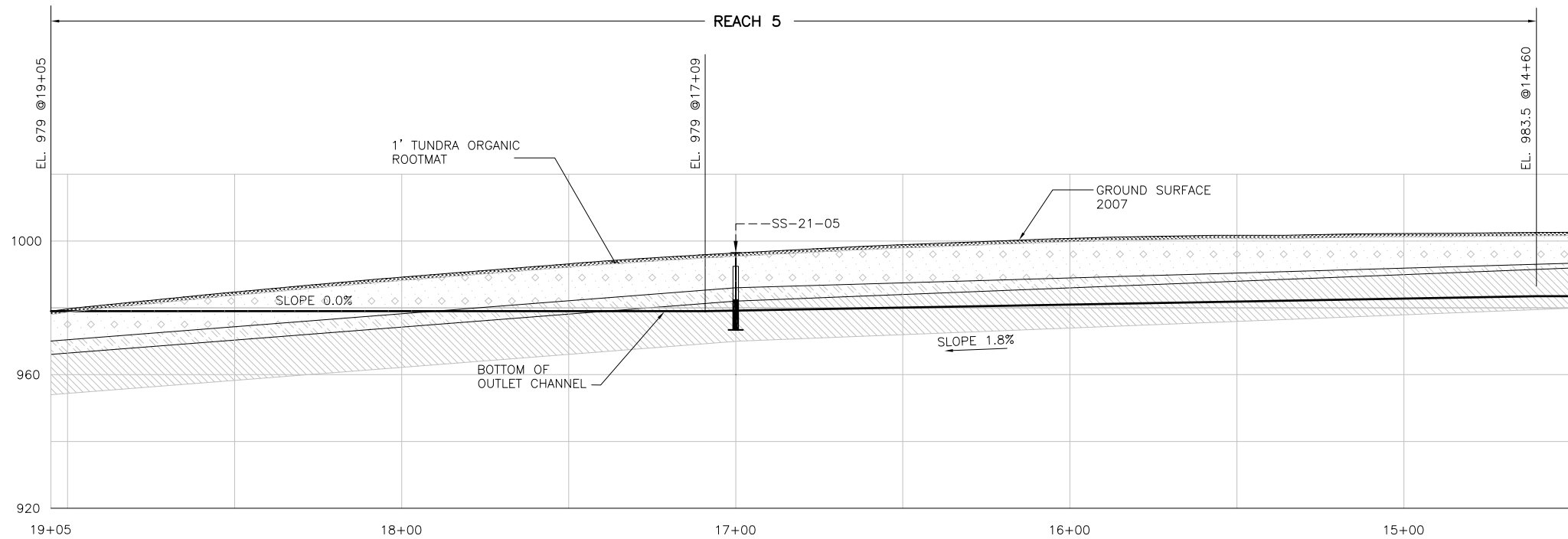
PROFILE SPILLWAY – REACH 5
FROM STA. 14+60 TO 19+31

REACH 5

1' TUNDRA ORGANIC ROOTMAT

GROUND SURFACE 2007

SS-21-05

SLOPE 0.0%

BOTTOM OF OUTLET CHANNEL

SLOPE 1.8%

EL. 979 @19+05

EL. 979 @17+09

EL. 983.5 @14+60

HORIZ. & VER. SCALE:
0   20   40
FEET

APPENDIX A
Surcharge Capacity Calculations

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URS

A-1

Precipitation data extracted from SRK water and load balance spreadsheet, Measured Precip Tab, (SRK, 2007b)

Table 4: Precipitation Released in inches (assumes 100% of snowpack released in May)																
	1992	1993	1994	1995	1996	1997	1998	**1999**	**2000**	2001	2002	2003	2004	2005	2006	2007
January	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
February	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
March	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
April	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
May	7.3	6.7	6.3	6.6	6.4	6.3	6.7	4.1	5.2	5.0	7.7	7.8	8.8	7.8	5.6	6.8
June	1.7	1.1	1.1	2.1	1.6	0.1	2.4	0.9	0.8	1.5	1.5	1.6	2.7	0.6	2.4	1.8
July	0.7	2.5	6.0	2.9	3.0	2.9	1.5	3.4	3.2	4.9	1.6	4.0	3.3	1.7	4.7	2.2
August	5.0	4.5	11.3	2.2	5.0	4.2	7.4	2.9	7.4	6.9	3.3	3.0	4.6	4.3	1.7	1.9
September	0.4	3.6	2.3	2.0	0.8	3.3	3.1	1.5	4.8	1.8	5.5	2.4	0.5	4.9	3.7	3.8
October	1.4	4.4	1.9	1.1	1.0	2.1	1.8	0.1	1.2	1.1	2.7	2.6	2.5	1.8	4.1	0.6
November	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
December	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	**16.4**	**22.7**	**28.8**	**17.0**	**17.7**	**18.8**	**22.7**	**12.8**	**22.5**	**21.1**	**22.4**	**21.5**	**22.4**	**21.1**	**22.2**	**17.1**

Average of May monthly precipitation from 1992-2007 = 6.6 inches
Average of June monthly precipitation from 1992-2007 = 1.5 inches
Average of July monthly precipitation from 1992-2007 = 3.1 inches
Average of August monthly precipitation from 1992-2007 = 4.9 inches
Average of September monthly precipitation from 1992-2007 = 2.7 inches
Average of October monthly precipitation from 1992-2007 = 2.0 inches
Average of annual precipitation from 1992-2007 = 20.5 inches

Averages determined using "AVERAGE" function in Microsoft Excel.

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A-2

URS

Evaporation data extracted from SRK water and load balance spreadsheet, Measured Evap Tab, (SRK, 2007b)

Table 3: Summary Table of Evaporation (inches of water)

Month	1992	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006
January	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
February	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
March	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
April	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
May	0.60	1.27	0.34	0.13		0.55	0.97	0.78	0.00	0.00	0.00	0.00	0.00	0.00	0.73
June	3.46	3.58	3.20	2.01		2.50	1.30	2.46	2.43	2.35	2.34	3.17	2.75	2.23	1.62
July	2.41	2.48	1.43	2.01		1.67	1.59	2.10	1.80	2.29	1.92	1.00	2.20	2.31	1.29
August	1.36	0.59	0.54	1.25		0.49	1.62	1.19	1.92	1.38	1.40	1.00	1.23	0.81	1.03
September	0.03	0.39	0.17	0.83		0.52	1.77	0.88	1.40	0.00	0.00	0.57	0.53	0.61	1.36
October	0.00	0.00	0.00	0.00		0.12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
November	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
December	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total	**7.84**	**8.29**	**5.68**	**6.22**		**5.84**	**7.25**	**7.41**	**7.54**	**6.02**	**5.66**	**5.73**	**6.71**	**5.95**	**6.02**

Average of May monthly evaporation 1992-2006 = 0.38 inches
Average of June monthly evaporation from 1992-2006 = 2.53 inches
Average of July monthly evaporation from 1992-2006 = 1.89 inches
Average of August monthly evaporation from 1992-2006 = 1.13 inches
Average of September monthly evaporation from 1992-2006 = 0.65 inches
Average of October monthly evaporation from 1992-2006 = 0.01 inches
Average of annual evaporation from 1992-2006 = 6.58 inches

Averages determined using "AVERAGE" function in Microsoft Excel.

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Stage-Storage Table, Formulas and Conversion Factors

	A	B	C	D	E	F
1	Stage (elev-ft)	Surface Area (ft²)	Depth (ft)	Storage (yd³)	Storage (Mgal)	Storage (Acre-ft)
2	986	26101020.9557	1	=D3+((B2+B3)/2)*C2/27	=D2*201.974026 /(10^6)	=D2*27/43560
3	985	25985239.7202	1	=D4+((B3+B4)/2)*C3/27	=D3*201.974026 /(10^6)	=D3*27/43560
4	984	25668995.0061	1	=D5+((B4+B5)/2)*C4/27	=D4*201.974026 /(10^6)	=D4*27/43560
5	983	25498012.6765	1	=D6+((B5+B6)/2)*C5/27	=D5*201.974026 /(10^6)	=D5*27/43560
6	982	25389329.9151	1	=D7+((B6+B7)/2)*C6/27	=D6*201.974026 /(10^6)	=D6*27/43560
7	981	25257935.1224	1	=D8+((B7+B8)/2)*C7/27	=D7*201.974026 /(10^6)	=D7*27/43560
8	980	24993705.6185	1	=D9+((B8+B9)/2)*C8/27	=D8*201.974026 /(10^6)	=D8*27/43560
9	979	24797413.9466	1	=D10+((B9+B10)/2)*C9/27	=D9*201.974026 /(10^6)	=D9*27/43560
10	978	24565106.5007	1	=D11+((B10+B11)/2)*C10/27	=D10*201.974026 /(10^6)	=D10*27/43560
11	977	24471430.2285	1	=D12+((B11+B12)/2)*C11/27	=D11*201.974026 /(10^6)	=D11*27/43560
12	976	24352285.8171	1	=D13+((B12+B13)/2)*C12/27	=D12*201.974026 /(10^6)	=D12*27/43560
13	975	24231135.9824	1	0	=D13*201.974026 /(10^6)	=D13*27/43560

$$\text{Storage (yd}^3) = \frac{\text{Previous Stage - Storage (yd}^3)}{} + \frac{\text{Stage Surface Area (ft}^2) + \text{Previous Stage Surface Area (ft}^2)}{2\ \text{Surface Areas}} \times \frac{\text{Depth (ft)}}{} \times \frac{1\ (\text{yd}^3)}{27\ (\text{ft}^3)}$$

$$\text{Storage (Mgal)} = \frac{\text{Storage (yd}^3)}{} \times \frac{201.974026\ (\text{gal})}{1\ (\text{yd}^3)} \times \frac{1\ (\text{Mgal})}{1,000,000\ (\text{gal})}$$

$$\text{Storage (acre - ft)} = \frac{\text{Storage (yd}^3)}{} \times \frac{27\ (\text{ft}^3)}{1\ (\text{yd}^3)} \times \frac{1\ (\text{acre})}{43,560\ (\text{ft}^2)}$$

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Stage-Storage Table, Values

Stage (elev-ft)	Surface Area (ft^2)	Depth (ft)	Storage (yd^3)	Storage (Mgal)	Storage (Acre-ft)
986	26,101,021	1	10,227,612	2,066	6,339
985	25,985,240	1	9,263,052	1,871	5,742
984	25,668,995	1	8,306,492	1,678	5,149
983	25,498,013	1	7,358,955	1,486	4,561
982	25,389,330	1	6,416,597	1,296	3,977
981	25,257,935	1	5,478,684	1,107	3,396
980	24,993,706	1	4,548,098	919	2,819
979	24,797,414	1	3,626,041	732	2,248
978	24,565,107	1	2,711,920	548	1,681
977	24,471,430	1	1,803,836	364	1,118
976	24,352,286	1	899,693	182	558
975	24,231,136	1	0	0	0

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Average Monthly Runoff in May (Including Spring Freshet)

The average monthly runoff for May was determined from total average May runoff minus the average volume of evaporation and seepage that leaves the pond during May. The total average monthly runoff during May was determined from the average monthly precipitation for May (P) and the maximum potential retention (S) as determined from the assigned curve number according to the NRCS runoff equation:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} = \frac{(6.6 - 0.2 \times 0.2)^2}{(6.6 + 0.8 \times 0.2)} = 6.37 \text{ inches}$$

Q = depth of runoff, in inches
P = depth of rainfall, in inches
S = maximum potential retention, in inches

The average monthly runoff was then multiplied by the catchment area and converted to cubic yards (yd³).

$$\text{Storage } (yd^3) = \frac{6.37 \text{ in}}{} \times \frac{1 \, ft}{12 \, in} \times \frac{70,234,514 \, ft^2}{} \times \frac{1 \, yd^3}{27 \, ft^3} = 1,380,845 \quad yd^3$$

The average depth of evaporation in the month of May from the pond is estimated to be 0.38 in from the SRK water and load balance (Appendix A-3). The depth of evaporation will be assumed across the surface area of the pond at the closure operating level of El. 977.0 (Appendix A-5) to determine average volume of evaporation according to the equation below:

$$\text{Evap Volume } (yd^3) = \frac{0.38 \text{ in}}{} \times \frac{1 \, ft}{12 \, in} \times \frac{24,471,430 \, ft^2}{} \times \frac{1 \, yd^3}{27 \, ft^3} = 28,701 \quad yd^3$$

The average monthly seepage from the pond in the summer with a 600' beach is calculated to be 573 gallons per minute (URS, 2007). The average monthly seepage is calculated from this flow rate according the following equation:

$$\text{Seepage Volume } (yd^3) = \frac{573 \text{ gpm}}{} \times \frac{60 \, min}{1 hr} \times \frac{24 \, hr}{day} \times \frac{31 \, day}{\text{Month of May}} \times \frac{1 \, yd^3}{201.97 \text{ gallons}} = 126,646 \quad yd^3$$

The total average May runoff volume is 1,225,498 cubic yards as calculated by subtracting the volume losses from evaporation and seepage from the total average May runoff below:

$$\text{Average Runoff in May } (yd^3) = 1,380,845 \quad yd^3 - 28,701 \, yd^3 - 126,646 \, yd^3 = 1,225,498 \quad yd^3$$

Adding this volume to the starting volume of storage at El. 977.0 gives the following volume to interpolate the stage elevation from the stage-storage table:

1,803,836 yd3 + 1,225,498 yd³ = 3,029,334 yd³

Using linear interpolation, this volume is equivalent to the following stage elevation from the stage-storage table (Appendix A-5):

$$\text{Stage (ft)} = \frac{(3,029,334 - 2,711,920) \times (979 - 978)}{(3,626,041 - 2,711,920)} + 978 = 978.35 \text{ yd}^3$$

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Water Treatment Plant 2 (WTP2) Discharge Capacity

The monthly discharge capacity of WTP2 at closure is assumed to be a portion of the predicted annual treatment capacity according to the ratio of average monthly discharge to the average annual discharge for WTP2 for the years 1999-2007, as detailed in the SRK water and load balance spreadsheet below and verified by TCAK personnel (SRK, 2008b; Weakley, 2008). The ratio of June discharge to annual discharge was determined to be 0.29. The expected annual discharge of WTP2 from the pond at closure is 471 Mgal from the SRK water and load balance (SRK, 2007b).

Evaporation data extracted from SRK water and load balance spreadsheet, Outflow 001 Tab, (SRK, 2007b)

Table 1: Outfall 001 Discharge Monthly Summaries (Mgals)

Year	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
1999	0.0	219.7	396.3	100.5	352.1	350.5	115.0	1,534.2
2000	0.0	24.0	245.8	184.8	307.2	323.8	55.0	1,140.6
2001	0.0	0.6	393.6	364.1	436.4	160.0	41.1	1,395.7
2002	0.0	42.4	289.9	246.8	107.3	268.9	37.4	992.8
2003	0.0	30.6	449.5	313.4	175.6	0.0	0.0	969.1
2004	0.0	241.6	379.5	155.9	205.6	60.8	0.0	1,043.3
2005	0.0	304.5	314.6	124.9	348.6	371.2	31.8	1,495.5
2006	0.0	216.7	392.1	353.3	250.2	241.1	0.4	1,453.8
2007	0.0	79.2	230.2	79.1	162.6	316.8	13.0	880.9
2008								
2009								
Average	0.0	128.8	343.5	213.6	260.6	232.6	32.6	1,211.8
Min	0.0	0.6	230.2	79.1	107.3	0.0	0.0	880.9
Max	0.0	304.5	449.5	364.1	436.4	371.2	115.0	1,534.2

Ratios	Apr	May	Jun	Jul	Aug	Sep	Oct
1999	0.00	0.14	0.26	0.07	0.23	0.23	0.07
2000	0.00	0.02	0.22	0.16	0.27	0.28	0.05
2001	0.00	0.00	0.28	0.26	0.31	0.11	0.03
2002	0.00	0.04	0.29	0.25	0.11	0.27	0.04
2003	0.00	0.03	0.46	0.32	0.18	0.00	0.00
2004	0.00	0.23	0.36	0.15	0.20	0.06	0.00
2005	0.00	0.20	0.21	0.08	0.23	0.25	0.02
2006	0.00	0.15	0.27	0.24	0.17	0.17	0.00
2007	0.00	0.09	0.26	0.09	0.18	0.36	0.01
2008							
2009							
Average	0.00	0.10	0.29*	0.18	0.21	0.19	0.03

The average June ratio of discharge was calculated from June discharge divided by annual discharge for 1999-2007. The average expected discharge at closure for the month of June is calculated in the following manner:

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$$\text{Avg June Discharge} \ (\text{yd}^3) = \frac{0.29 \ \text{June Discharge}}{\text{Annual Discharge}} \times \frac{471 \ \text{Mgal Annual Disharge at Closure}}{} \times$$

$$\frac{1,000,000 \ gal}{1Mgal} \times \frac{1 \ \text{yd}^3}{201.97 \ \text{gal}} = 676,289 \ \text{yd}^3$$

The average expected runoff into the pond in the month of June using the same calculations and methodology as used for May above is the following:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} = \frac{(1.5 - 0.2 \times 0.2)^2}{(1.5 + 0.8 \times 0.2)} = 1.28 \ \text{in}$$

$$\text{Storage} \ (\text{yd}^3) = \frac{\text{June Runoff}}{} \times \frac{\textit{Cathcment}}{} = \frac{1.28 \ \text{in}}{} \times \frac{1 \ ft}{12 \ in} \times \frac{70,234,514 \ ft^2}{} \times \frac{1 \ yd^3}{27 \ ft^3} = 277,469 \ yd^3$$

$$\text{June Evap Volume} \ (\text{yd}^3) = \frac{2.53 \ \text{in}}{} \times \frac{1 \ ft}{12 \ in} \times \frac{24,471,430 \ ft^2}{} \times \frac{1 \ yd^3}{27 \ ft^3} = 191,089 \ yd^3$$

$$\text{Seepage Volume} \ (\text{yd}^3) = \frac{573 \ \text{gpm}}{} \times \frac{60 \ \text{min}}{1hr} \times \frac{24 \ hr}{day} \times \frac{31 \ day}{\text{Month of May}} \times \frac{1 \ yd^3}{201.97 \ \text{gallons}} = 126,646 \ yd^3$$

$$\text{Average Runoff in May} \ (\text{yd}^3) = 277,469 \ yd^3 - 191,089 \ yd^3 - 126,646 \ yd^3 = -40,266 \ yd^3$$

The June discharge capacity of WTP2 greatly exceeds the calculated June runoff. Using the same equations above for May-Oct, it can clearly be seen that WTP2 will have the monthly capacity to maintain the runoff level below the depth from the volume of May runoff (1,225,498 cubic yards) in the pond, as seen in the table below:

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Balance of Runoff in Tailings Pond, Variables and Formulas

	A	B	C	D	E	F	G	H	I	J	K
1				Monthly Balance of Pond Level from Annual Average Runoff and WTP2 Discharging							
2	Month	Avg Monthly Precip (in)	Precip as Runoff (in)	Volume - Precip X Catchment - (yd³)	Avg Monthly Evaporation (in)	Evap Volume -Evap * Pond Surface Area - (yd³)	Avg Monthly Seepage (yd³)	Monthly Runoff (yd³)	Monthly to Annual Discharge Ratio	Monthly Discharge from WTP2 (yd³)	Cumulative Annual Runoff (yd³)
3	May	6.6	=ROUND((B3-0.2*0.2)^2/(B3+0.8*0.2),2)	=C3/12*70234514/27	0.38	=E3/12*24471430/27	126646	=D3-F3-G3	0.1	0	=H3-J3
4	June	1.5	=ROUND((B4-0.2*0.2)^2/(B4+0.8*0.2),2)	=C4/12*70234514/27	2.53	=E4/12*24471430/27	126646	=D4-F4-G4	0.29	=I4*471*1000000/201.97	=K3+H4-J4
5	July	3.1	=ROUND((B5-0.2*0.2)^2/(B5+0.8*0.2),2)	=C5/12*70234514/27	1.89	=E5/12*24471430/27	126646	=D5-F5-G5	0.18	=I5*471*1000000/201.97	=K4+H5-J5
6	August	4.9	=ROUND((B6-0.2*0.2)^2/(B6+0.8*0.2),2)	=C6/12*70234514/27	1.13	=E6/12*24471430/27	126646	=D6-F6-G6	0.21	=I6*471*1000000/201.97	=K5+H6-J6
7	September	2.7	=ROUND((B7-0.2*0.2)^2/(B7+0.8*0.2),2)	=C7/12*70234514/27	0.65	=E7/12*24471430/27	126646	=D7-F7-G7	0.19	=I7*471*1000000/201.97	=K6+H7-J7
8	October	2	=ROUND((B8-0.2*0.2)^2/(B8+0.8*0.2),2)	=C8/12*70234514/27	0.01	=E8/12*24471430/27	126646	=D8-F8-G8	0.03	=I8*471*1000000/201.97	=K7+H8-J8
9											

$$\text{Precip as Runoff (in)} = \frac{(\text{Avg Monthly Precip (in)} - 0.2 \times S)^2}{\text{Avg Monthly Precip (in)} + 0.8 \times S}, \; S = 0.2 \; (\text{p.6}), \; \text{Avg Monthly Precip}$$

$$\text{Volume (yd}^3) = \frac{\text{Precip as Runoff (in)} \times \text{Catchment Area (ft}^2)}{12\,in} \times \frac{1\,ft}{27\,ft^3} \times \frac{1\,yd^3}{27\,ft^3}$$

$$\text{Evap Volume (yd}^3) = \frac{\text{Avg Monthly Evap (in)} \times \text{Pond Surface Area at 977-ft (ft}^2)}{12\,in} \times \frac{1\,ft}{27\,ft^3} \times \frac{1\,yd^3}{27\,ft^3},$$

(Avg Monthly Evap from a Pond Surface Area at 977-ft)

Monthly Runoff (yd³) = Monthly Runoff (yd³) – Monthly Evap Volume (yd³) – Seepage (yd³), Seepage from calculation on p. 7

$$\text{Monthly Discharge (yd}^3) = \frac{\text{Monthly Annual Discharge Ratio} \times 471 \text{ MGal}}{1\,Mgal} \times \frac{1,000,000\,gal}{1\,Mgal} \times \frac{1\,yd^3}{201.97\,gal^3}$$

Cumulative Annual Runoff (yd³) = Previous Runoff (yd³) + Monthly Runoff (yd³) – Monthly Discharge (yd³)

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Balance of Runoff in Tailings Pond, Values

Month	Avg Monthly Precip (in)	Precip as Runoff (in)	Volume – Precip X Catchment – (yd^3)	Avg Monthly Evaporation (in)	Evap Volume – Evap * Pond Surface Area – (yd^3)	Avg Monthly Seepage (yd^3)	Monthly Runoff (yd^3)	Monthly to Annual Discharge Ratio	Monthly Discharge from WTP2 (yd^3)	Cumulative Runoff (yd^3)
					Balance of Runoff in Tailings Pond					
May	6.6	6.4	1,380,845	0.38	28,701	126,646	1,225,498	0.10	0	1,225,498
June	1.5	1.3	277,470	2.53	191,089	126,646	-40,265	0.29	676,289	508,945
July	3.1	2.9	622,139	1.89	142,750	126,646	352,743	0.18	419,765	441,922
August	4.9	4.7	1,012,331	1.13	85,348	126,646	800,337	0.21	489,726	752,533
September	2.7	2.5	535,430	0.65	49,094	126,646	359,690	0.19	443,086	669,137
October	2.0	1.8	385,856	0.01	755	126,646	258,455	0.03	69,961	857,631

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Approximation of Probable Maximum Precipitation using Probabilistic Methods

The PMP and PMF are deterministic events. Their estimation by probabilistic extrapolations based on records of relatively short periods is not considered accurate. However, probabilistic estimates may be used for verification of the reasonableness of deterministically estimated PMP and PMF peaks. The commonly adopted return period of the PMP is 10,000 to 1,000,000 years (Buehler, 1984; National Research Council, 1985; Bureau of Reclamation, 1992). For a conservative estimate, a return period of 1,000,000 years was used.

The probabilistic estimation of the PMP involves a frequency analysis of observed maximum daily precipitation depths for individual years, identification of statistical parameters of the data, and use of an appropriate probability distribution to obtain a probabilistic estimate of PMP. The Fisher-Tippett Type I probability distribution (or Gumbel distribution) has been used by the U.S. Weather Bureau for frequency analysis of precipitation data (Weather Bureau, 1961; Chow, 1964; Haan, 1977).

The theoretical skew coefficient of the Fisher-Tippett Type I (Gumbel) distribution is 1.14 (Chow, 1964). The skew coefficient of the data for the mine gage is 1.27, which is close to the constant, theoretical skew coefficient of the Fisher-Tippett Type I (Gumbel) distribution. This further suggests that the Fisher-Tippett Type I (Gumbel) distribution is applicable to precipitation data from the mine.

In view of the above, the Fisher-Tippett Type I (Gumbel) distribution was used to estimate the 1,000,000-year 24-hour precipitation depth at the mine based on a frequency analysis of annual daily maximum values during 1992 to 2007. The statistical parameters of the data are shown in Table 5-1. Several methods have been proposed to estimate the parameters of the Fisher-Tippett Type I (Gumbel) distribution (Chow, 1964; Kite, 1977; Lettenmaier and Burges, 1982; Ponce, 1994). PMP estimates using two commonly used methods are presented below.

The first method involves estimating the mean and standard deviation of the Gumbel variates as a function of record length (Kite, 1977; USACE, 1993; Ponce, 1994). Using this method, the 1,000,000-year daily precipitation depth is estimated to be 8.11 inches. With the adjustment factor of 1.13 to convert the estimated daily value to the 24-hour value, the 24-hour PMP is estimated to be 9.17 inches. The computations for this method are summarized in Appendix A-13.

The second method involves using the limiting values of the Gumbel variates, which are those corresponding to n equal to infinity. (Lettenmaier and Burges, 1982; Ponce, 1994; Wanielista et al., 1997). This method uses a standard deviation computed using n, instead of n-1, where n is the number of observations. Using this method, the 1,000,000-year daily precipitation depth is estimated to be 6.59 inches. With the adjustment factor of 1.13, the 24-hour PMP is estimated to be 7.44 inches. The computations for this method are summarized in Appendix A-13.

These estimates of the 24-hour PMP (i.e., 7.44 and 9.17 inches) demonstrate the conservatism of the adopted value in subsection 5.4.4.

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Approximation of PMP using Empirical Frequency Factors

This method is also based on a frequency analysis. It includes use of the mean and standard deviation of the data and extrapolation to obtain the PMP value using an empirical frequency factor, K (Hershfield, 1961; World Meteorological Organization (WMO), 1973; National Research Council, 1985; Ponce, 1994). The suggested frequency factor varies from 15 to 20. Thus the one-day PMP values at Red Dog Mine are estimated to be as follows:

$$X = X_{bar} + K\,S$$

Where:

X = estimated PMP value;

K = frequency factor;

X_{bar} = mean of observed data (i.e., 1.28 inches); and

S = standard deviation of data with n-1 used as the divisor (i.e., 0.5306)

The following results were obtained:

Upper limit PMP (daily value) = 1.28 +20* 0.5306 = 11.89 inches.

Lower limit PMP (daily value) = 1.28 +15* 0.5306 = 9.24 inches

Using the adjustment factor of 1.13:

The upper limit 24-hour PMP = 13.4 inches.

The lower limit 24-hour PMP = 1.13*9.23 = 10.4 inches.

These upper and lower limit values of the 24-hour PMP also demonstrate the reasonableness of the adopted PMP value from subsection 5.4.4.

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PMP Computations using Gumbel Method 1

According to Gumbel distribution (USACE, 1993; Ponce, 1994),

Y_{Tr} = -ln[-ln ((T-1)/T)] =-[0.83405 +2.30250 log log T/ (T -1)]

T (PMP) = 1,000,000 years
Y_{Tr} = 13.81

Number of years of observed data = n = 16
From Tables (Kite, 1977; Ponce, 1994), for n = 16, y_n = 0.5157 and s_n = 1.0316.
So, frequency factor = K (T = 1,000,000) = [(Y_{Tr} – y_n)/s_n)]= 12.8924

X = Xbar + K S

where X = estimated value for T-year return interval;
K = frequency factor;
Xbar = mean of observed data (i.e., 1.28 inches);
T = return period in years (i.e., 1,000,000 years); and
S = standard deviation of data with n-1 used as the divisor (i.e., 0.5306)

Thus, daily precipitation (T = 1,000,000 years) = 1.28+ 12.8924 *0.53 = 8.11 inches.
24-hour PMP = 1.13*8.11 = 9.17 inches.

Also, for T= 100 years and n =16, K = 3.9635.
So, daily precipitation (T = 100 years) = 1.28+3.9635*0.53 = 3.38 inches.
100-year 24-hour precipitation depth = 1.13 *3.38 = 3.82 inches.

PMP Computations using Gumbel Method 2

According to this method (Lettenmaier and Burges, 1982; Ponce, 1994; Wanielista et al., 1997),

X = Xbar – [0.7789 ln (ln (T/(T-1))+ 0.45] S

where X = estimated value for T-year return interval;
Xbar = mean of observed data (i.e., 1.28 inches);
T = return period in years (i.e., 1,000,000 years); and
S = standard deviation of data with n used as the divisor (i.e., 0.5137).

Using this method, 1,000,000-year daily value = 6.59 inches.
24-hour PMP = 7.44 inches.

Also, daily precipitation (T=100 years) = 2.89 inches.
100-year 24-hour precipitation depth = 1.13 *2.89 = 3.27 inches.

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URS

A-13

PMP Series

The same catchment area (2.52 square miles) and CN (98) that was determined from the annual average runoff calculations were used along with the calculated PMP (13.26 inches) to determine the PMF series contribution to the pond storage capacity of 3,951,341 cubic yards.

The PMF was determined from the calculated PMP precipitation (P) and the maximum potential retention (S) as determined from the assigned curve number according to the NRCS runoff equation (NRCS, 2004a):

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} = \frac{(13.26 - 0.2 \times 0.2)^2}{(13.26 + 0.8 \times 0.2)} = 13.02 \text{ inches}$$

Q = depth of runoff, in inches
P = depth of rainfall, in inches
S = maximum potential retention, in inches

The PMF is multiplied by the catchment area and the PMF series 1.4 multiplier per NRC 3.11 (NRC, 1977) and then converted to cubic yards to determine the stage raise in the pond due to the PMF series from the stage-storage table:

$$\text{Storage (yd}^3) = \frac{13.02 \text{ in}}{} \times \frac{1.4}{} \times \frac{1\,ft}{12\,in} \times \frac{70,234,514\,ft^2}{} \times \frac{1\,yd^3}{27\,ft^3} = 3,951,341 \text{ yd}^3$$

Adding this volume to the starting volume of storage at elevation 978.35-ft (annual average runoff level) gives the following volume to interpolate stage elevation from the stage-storage table:

3,029,334 yd³ + 3,951,341 yd³ = 6,980,675 yd³

Using linear interpolation, this volume is equivalent to the following stage elevation from the stage-storage table:

$$\text{Stage (ft)} = \frac{(x_2 - x_1) \times (y_3 - y_1)}{(x_3 - x_1)} + y_1 = \frac{(6,980,675 - 6,416,597) \times (983 - 982)}{(7,358,955 - 6,416,597)} + 982 = 982.60 \text{ yd}^3$$

100-yr Flood

The 100-yr flood was determined from the calculated 24-hr, 100-yr precipitation (P) and the maximum potential retention (S) as determined from the assigned curve number according to the NRCS runoff equation:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} = \frac{(4.25 - 0.2 \times 0.2)^2}{(4.25 + 0.8 \times 0.2)} = 4.02 \text{ inches}$$

Q = depth of runoff, in inches
P = depth of rainfall, in inches
S = maximum potential retention, in inches

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A-14

The 100-yr flood is multiplied by the catchment area and then converted to cubic yards to determine the stage raise in the pond from the stage-storage table:

$$\text{Storage} \ (\text{yd}^3) = \frac{4.02 \ in}{} \times \frac{1 \, ft}{12 \, in} \times \frac{70{,}234{,}514 \ ft^2}{} \times \frac{1 \, yd^3}{27 \, ft^3} = 871{,}428 \ \text{yd}^3$$

Adding this volume to the starting volume of storage at elevation 982.60-ft (PMF series elevation) gives the following volume to interpolate stage elevation from the stage-storage table:

6,980,675 yd³ + 871,428 yd³ = 7,852,103 yd³

Using linear interpolation, this volume is equivalent to the following stage elevation from the stage-storage table:

$$\text{Stage} \ (\text{ft}) = \frac{(x_2 - x_1) \times (y_3 - y_1)}{(x_3 - x_1)} + y_1 = \frac{(7{,}852{,}103 \ - 7{,}358{,}955) \times (984 - 983)}{(8{,}306{,}492 \ - 7{,}358{,}955)} + 983 = 983.52 \ yd^3$$

Annual Runoff

The annual runoff into the pond was calculated as shown below using the same methodology as used for the month of May. The average annual precipitation from 1992-2007 (Appendix A-2) and average annual evaporation from 1992-2006 excluding 1996 (Appendix A-3) were used in the calculations below. The average annual seepage calculations were based off of summer and winter seepage values from URS (URS, 2007).

$$Q = \frac{(\text{P - 0.2S})^2}{\text{P + 0.8S}} = \frac{(20.5 \ - 0.2 \times 0.2)^2}{(20.5 \ + 0.8 \times 0.2)} = 20.26 \ in$$

$$\text{Storage} \ (\text{yd}^3) = \frac{\text{Annual Runoff}}{} \times \frac{\text{Cathcment}}{} = \frac{20.26 \ in}{} \times \frac{1 \, ft}{12 \, in} \times \frac{70{,}234{,}514 \ ft^2}{} \times \frac{1 \, yd^3}{27 \, ft^3} = 4{,}391{,}825 \ yd^3$$

$$\text{Annual Evap Volume} \ (\text{yd}^3) = \frac{6.58 \ in}{} \times \frac{1 \, ft}{12 \, in} \times \frac{24{,}471{,}430 \ ft^2}{} \times \frac{1 \, yd^3}{27 \, ft^3} = 496{,}982 \ yd^3$$

$$\text{Annual Seepage Vol.} \ (\text{yd}^3) = \frac{573 \ \text{gpm} \ + 473 \ \text{gpm}}{2} \times \frac{60 \ \text{min}}{1 hr} \times \frac{24 \, hr}{day} \times \frac{365 \, day}{1 \ \text{yr}} \times \frac{1 \, yd^3}{201.97 \ \text{gal}} = 1{,}361{,}037 \ \text{yd}^3$$

$$\text{Average Annual Runoff} \ (\text{yd}^3) = 4{,}391{,}825 \ yd^3 - 496{,}982 \ yd^3 - 1{,}361{,}037 \ yd^3 = 2{,}533{,}806 \ yd^3$$

The volume of storage available from 1.4 x PMF is equal to 3,951,341 yd³, as calculated above on A-12. The annual runoff is equal to 2,533,806 yd³ of storage. There is enough surcharge capacity to store approximately 1.6 years of annual runoff.

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URS

A-15

Annual Runoff with Full PMF

The same catchment area (2.52 square miles) and CN (98) that was determined for the annual runoff calculations were used along with the calculated PMP (13.26 in) to determine the PMF and annual runoff contribution to the pond storage capacity of 5,356,192 cubic yards.

The PMF was determined from the calculated PMP (P) (Subsection 5.4.4.1) and the maximum potential retention (S) (Subsection 5.3.5) as determined from the assigned curve number according to the NRCS runoff equation (NRCS, 2004a):

$$Q = \frac{(P\text{ - }0.2S)^2}{P + 0.8S} = \frac{(13.26\ -0.2 \times 0.2)^2}{(13.26\ +0.8 \times 0.2)} = 13.02 \text{ inches}$$

Q = depth of runoff, in inches
P = depth of rainfall, in inches
S = maximum potential retention, in inches

The PMF is multiplied by the catchment area and then converted to cubic yards as follows:

$$\text{Storage } (\text{yd}^3) = \frac{13.02 \text{ in}}{12\,in} \times \frac{1\,ft}{12\,in} \times \frac{70,234,514\ ft^2}{} \times \frac{1\,yd^3}{27\ ft^3} = 2,822,386 \quad \text{yd}^3$$

The annual runoff was calculated with the same methods as the PMF, but with precipitation = 20.5 inches (Appendix A-2).

$$Q = \frac{(P\text{ - }0.2S)^2}{P + 0.8S} = \frac{(20.5\ -0.2 \times 0.2)^2}{(20.5\ +0.8 \times 0.2)} = 20.26 \text{ in}$$

The annual runoff is then adjusted for evaporation from the tailings pond surface at El. 977 (Appendix A-3) and seepage loss (URS, 2007) as shown below:

$$\text{Storage } (\text{yd}^3) = \frac{\text{Annual Runoff}}{} \times \textit{Cathcment} = \frac{20.26 \text{ in}}{} \times \frac{1\,ft}{12\,in} \times \frac{70,234,514\ ft^2}{} \times \frac{1\,yd^3}{27\ ft^3} = 4,391,825 \quad yd^3$$

$$\text{Annual Evap Volume } (\text{yd}^3) = \frac{6.58 \text{ in}}{} \times \frac{1\,ft}{12\,in} \times \frac{24,471,430\quad ft^2}{} \times \frac{1\,yd^3}{27\ ft^3} = 496,982 \quad yd^3$$

$$\text{Annual Seepage Vol. } (\text{yd}^3) = \frac{573 \text{ gpm} + 473 \text{ gpm}}{2} \times \frac{60 \text{ min}}{1hr} \times \frac{24\,hr}{day} \times \frac{365\,day}{1 \text{ yr}} \times \frac{1\,yd^3}{201.97 \text{ gal}} = 1,361,037 \quad \text{yd}^3$$

$$\text{Average Annual Runoff } (\text{yd}^3) = 4,391,825\ yd^3 - 496,982\ yd^3 - 1,361,037\ yd^3 = 2,533,806 \quad yd^3$$

Adding the annual runoff volume to the storage at El. 977.0 and interpolating from the stage-storage table (Appendix A-5) raises the level of the tailings pond to El. 979.77.

A full PMF and one year of runoff is equal to 5,356,192 cubic yards of storage (2,533,806 yd^3 + 2,533,806 yd^3 = 5,356,192 yd^3). Adding this volume to the starting volume of storage at El. 977.0 and interpolating from the stage-storage table (Appendix A-5) raises the level of the tailings pond to El. 982.79.

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A-16

APPENDIX B
Spillway Capacity Calculations

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URS

B-1

Time of Concentration Calculation Using TR-55 Method, Variable and Equation Table

	A	B	C	D	E	F	G	H	I	J	K
1							Time of Concentration Calculation from TR-55 - Urban Hydrology for Small Watersheds				
2	Flow Type	L (ft)	Start Elev (ft)	End Elev (ft)	n_s	k	S_0 (ft/ft)	V (fps)	P_2 (in/day)	T_t (min)	Lag Time (0.6 * T_t)
3	Sheet Flow	298	1729	1679	0.13		=(C3-D3)/B3		1.2	=0.42 * (E3*B3)^0.8/((I3^0.5)*(G3^0.4))	=J3*0.6
4	Shallow Concentrated Flow	853	1679	1464		11	=(C4-D4)/B4	=F4*SQRT(G4)		=B4/(60*H4)	=J4*0.6
5	Open Channel Flow	3612	1464	986		17	=(C5-D5)/B5	=F5*SQRT(G5)		=B5/(60*H5)	=J5*0.6
6											
7	Totals	=SUM(B3:B5)					=(C3-D5)/B7				=SUM(K3:K5)
8											

Equations:

Sheet Flow (NRCS, 1986):
$$T_t \,(min) = \frac{0.42 \times (n \times L\,(ft))^{0.8}}{P_2\,(in)^{0.5} \times s^{0.4}\,(ft/ft)}$$

Shallow Concentrated and Concentrated Flow ((Washington State Department of Ecology, 2001):
$$V\,(ft/s) = k\,(ft/s) \times s^{0.5}\,(ft/ft), \quad T_t\,(min) = \frac{L\,(ft)}{60 \times V\,(ft/s)}$$

Variables:

T_t = travel time (min)

L = length of travel (ft)

s_0 = slope of flow path (ft/ft) = (starting elevation – ending elevation) / length of travel

n_s = sheet flow Manning's effective roughness coefficient (NRCS, 1986)

k = time of concentration velocity factor (ft/s) (Washington State Department of Ecology, 2001)

P_2 = 2-year, 24-hour rainfall (in) (Appendix B-4)

V = velocity (ft/s)

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URS

B-2

Time of Concentration Calculation Using TR-55 Method, Value Table

Flow Type	L (ft)	Start Elev (ft)	End Elev (ft)	n_s	k	S_0 (ft/ft)	V (fps)	P_2 (in/day)	T_t (min)	Lag Time (0.6 * T_t)
Time of Concentration Calculation from TR-55 – Urban Hydrology for Small Watersheds										
Sheet Flow	298	1729	1679	0.13		0.17		1.2	14.60	8.76
Shallow Concentrated Flow	853	1679	1464		11	0.25	5.52		2.57	1.54
Open Channel Flow	3612	1464	986		17	0.13	6.18		9.73	5.84
Totals	**4763.0**					**0.16**				**16.14**

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B-3

URS

2-year, 24-Hour Rainfall Calculation Using Gumbel Type 1 Distribution

Given:		
Variables:	Values (in/day)	Reference
x_m =average precipitation =	1.28	Table 5-1
s = standard deviation of precipitation magnitudes =	0.51	Table 5-1
P = PMP Probability =	0.5	
Equations:		
$P = 1-e^{-e^{\wedge}-b}$		(Wanielista et al., 1997)
$P-1 = -e^{-e^{\wedge}-b}$		
$1-P = e^{-e^{\wedge}-b}$		
$\ln(1-P) = -e^{-b}$		
$-\ln(1-P) = e^{-b}$		
$\ln(-\ln(1-P)) = -b$		
$-\ln(-\ln(1-P))= b =$	0.37	
$b = (1/0.7797*s)*(x-x_m +0.45*s)$		(Wanielista et al., 1997)
$b*s*0.7797 = x-x_m +0.45*s$		
$b*s*0.7797+x_m-0.45*s = x =$	1.20	
x =	**1.20**	

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B-4

URS

Stage-Storage-Discharge Curve, Formulas

	A	B	C	D	E
57	Stage (elev-ft)	Storage (Acre-ft)	Top of Channel Width - L (ft)	Head on Weir Crest - H (ft)	Q = C*L*H$^{3/2}$ (cfs)
58	986	6339.42913270776	=((40+(A58-A61)*2*2.5)+40)/2	=A58-A61	=2.64*C58*D58^1.5
59	985	5741.56112678604	=((40+(A59-A61)*2*2.5)+40)/2	=A59-A61	=2.64*C59*D59^1.5
60	984	5148.6520964107	=((40+(A60-A61)*2*2.5)+40)/2	=A60-A61	=2.64*C60*D60^1.5
61	983.52	4866.74020835228	40	0	0

Equations

$$\text{Average Channel Width (ft)} = \frac{\text{Weir BaseWidth} + 2 \times \text{Weir Head} \times \text{Weir WallSlope} + \text{Weir BaseWidth}}{2}$$

Head on Crest (ft) = Stage Elevation - Spillway Crest Elevation

$Q = CLH^{3/2}$

Where:

Q = Discharge, cfs

C = Discharge coefficient

L = Width of Weir, ft

H = height of water above weir, ft

Stage-Storage-Discharge Curve, Values

Stage (elev-ft)	Storage (Acre-ft)	Average Channel Width – L (ft)	Head on Weir Crest – H (ft)	Q = C*L*H$^{3/2}$ (cfs)
986.0	6339	46.20	2.48	476.35
985.0	5742	43.70	1.48	207.72
984.0	5149	41.20	0.48	36.17
983.52	4867	40.00	0.00	0.00

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URS

HEC-HMS Model Output, One Half PMF (USACE, 2001)

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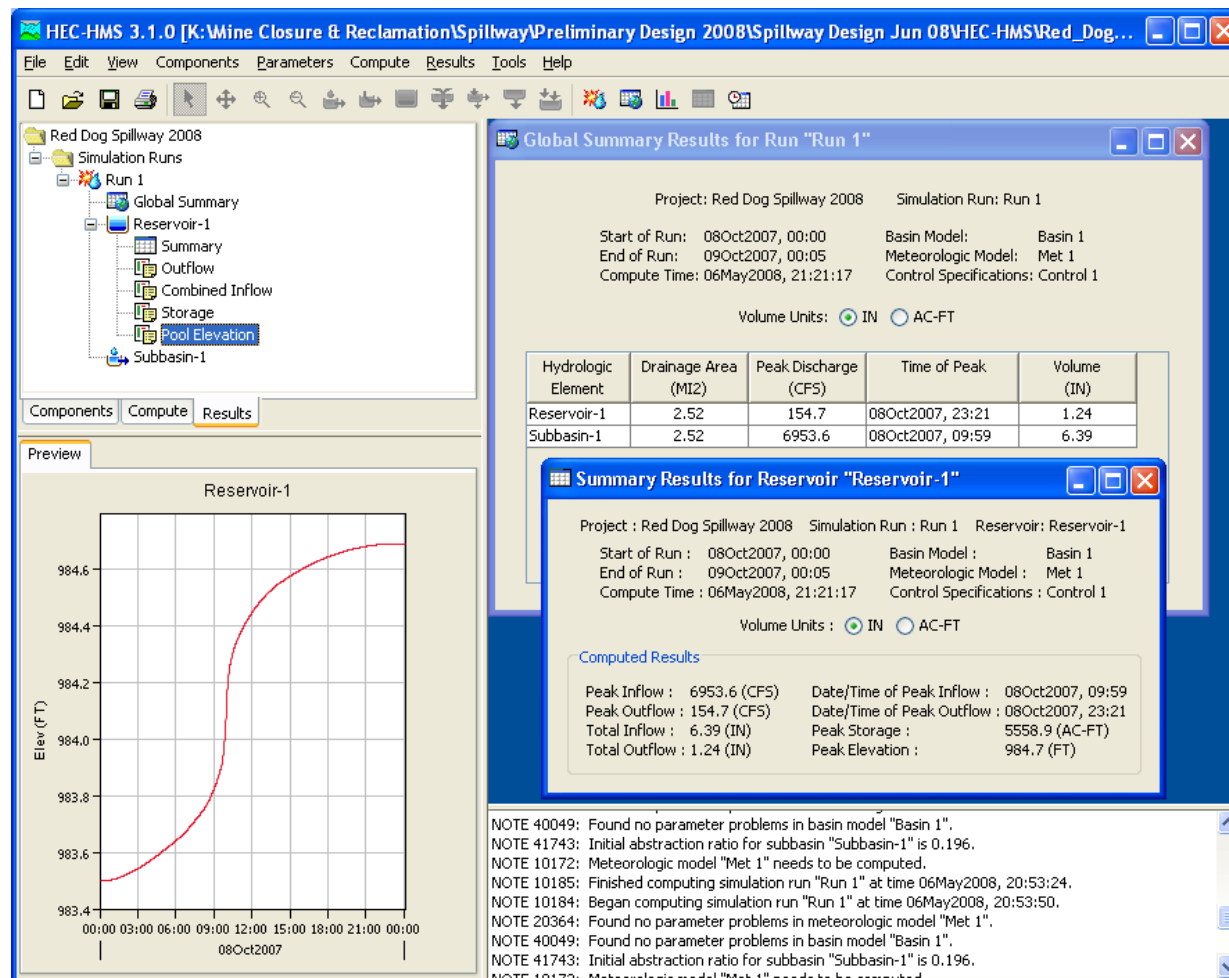
HEC-HMS Model Output, PMF (USACE, 2001)

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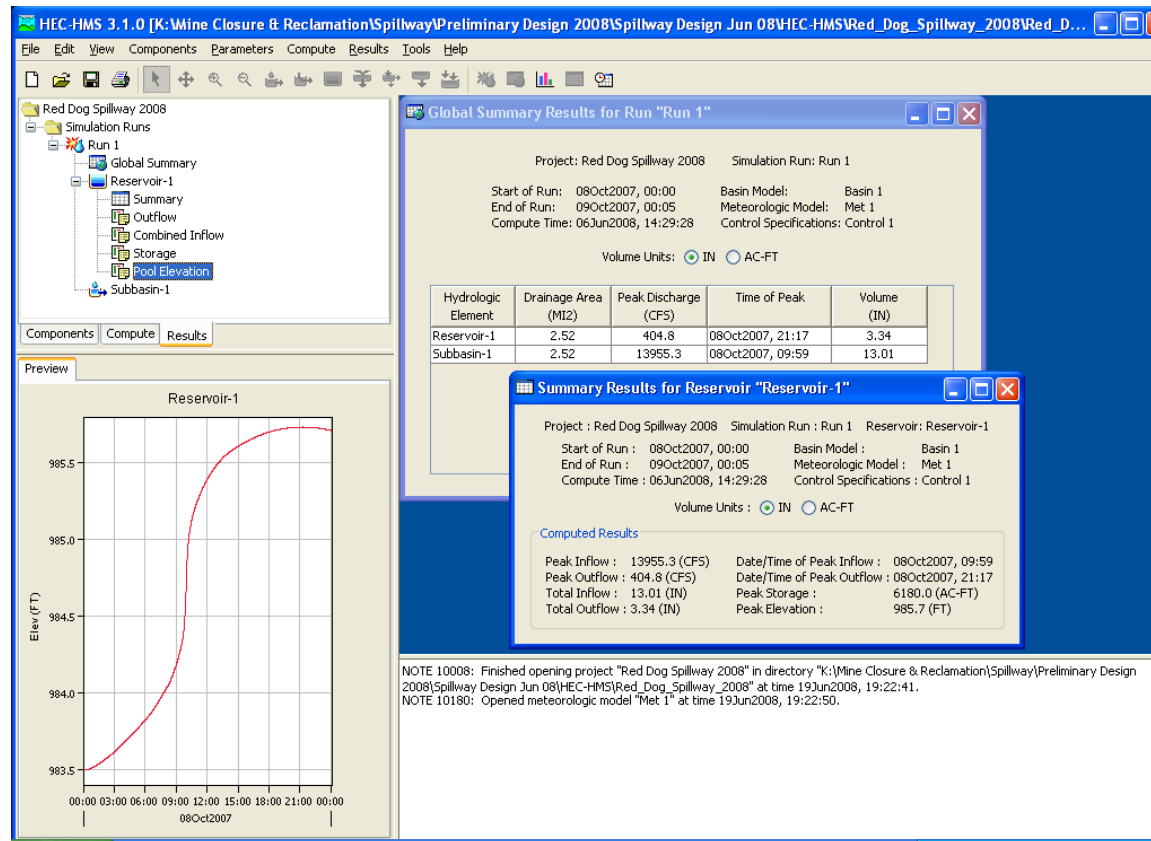
HEC-RAS Output Files, Profile Plot of Inlet Channel for One Half PMF, (USACE, 2004)

Prelim Spillway Design, Red Dog Mine, AK   Plan: Weir Outlet   7/8/2008

Preliminary Spil 4

Spillway control section outlet at critical depth

Tailing pond elevation required to push water through inlet channel over the spillway.

Inlet channel slope of -0.018 ft/ft

Legend

EG Full Spillway
WS Full Spillway
Crit Full Spillway
Ground

Elevation (ft)

Main Channel Distance (ft)

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Dog Tailings Main Dam Ultimate Closure Configuration-111408.doc

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URS

HEC-RAS Output Files for One Half PMF, Inlet Channel Parameters, (USACE, 2004)

Spillway Station	Q Total	Min Ch El	W.S. Elev	Crit W.S.	Vel Chnl	Flow Area	Top Width	Froude #
	(cfs)	(ft)	(ft)	(ft)	(ft/s)	(sq ft)	(ft)	
1860	154.7	976.3	984.92		0.06	2540.02	316.12	0
1800	154.7	977.4	984.92		0.09	1698.75	244.62	0.01
1760	154.7	978.1	984.92		0.13	1284.58	205.55	0.01
1700	154.7	979.2	984.92		0.2	791.44	152.61	0.02
1660	154.7	979.9	984.92		0.3	535.16	114.15	0.02
1650	154.7	980.1	984.92		0.33	486.61	108.16	0.03
1640	154.7	980.3	984.92		0.36	451.56	107.05	0.03
1630	154.7	980.4	984.92		0.39	407.18	96.86	0.03
1620	154.7	980.6	984.92		0.44	367.07	91.46	0.04
1610	154.7	980.8	984.92		0.49	330.22	86.36	0.04
1600	154.7	981	984.92		0.54	296.06	81.45	0.05
1590	154.7	981.1	984.92		0.59	272.96	77.25	0.05
1580	154.7	981.3	984.91		0.66	244.05	72.94	0.06
1570	154.7	981.5	984.91		0.74	217.75	68.94	0.07
1560	154.7	981.7	984.91		0.82	194.64	65.5	0.08
1545	154.7	982	984.9		0.97	165.66	61.41	0.1
1530	154.7	982.2	984.9		1.13	141.9	56.69	0.12
1515	154.7	982.5	984.88		1.39	114.47	50.46	0.16
1500	154.7	982.8	984.86		1.68	94.92	49.18	0.21
1490	154.7	983	984.84		1.95	81.42	47.11	0.25
1480	154.7	983.1	984.81		2.13	74.71	46.2	0.29
1470	154.7	983.3	984.76		2.57	61.7	44.34	0.37
1465	154.7	983.4	984.73		2.84	55.71	43.98	0.43
1460	154.7	983.5	984.68		3.13	50.76	45.91	0.51
1450	154.7	983.5	984.62		3.33	47.73	45.58	0.56
1440	154.7	983.5	984.52		3.65	43.45	45.1	0.64
1435	154.7	983.5	984.45		3.93	40.31	44.76	0.71
1432	154.7	983.5	984.39	984.26	4.21	37.54	44.45	0.79
1430	154.7	983.5	984.26	984.26	4.94	31.91	43.81	1

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HEC-RAS Output Files, Profile Plot of Inlet Channel for PMF, (USACE, 2004)

Prelim Spillway Design, Red Dog Mine, AK   Plan: Weir Outlet   7/7/2008

Preliminary Spil 4

Spillway control section outlet at critical depth

Tailing pond elevation required to push water through inlet channel over the spillway.

Inlet channel slope of -0.018 ft/ft

Legend

EG Full Spillway

WS Full Spillway

Crit Full Spillway

Ground

Elevation (ft)

Main Channel Distance (ft)

URS
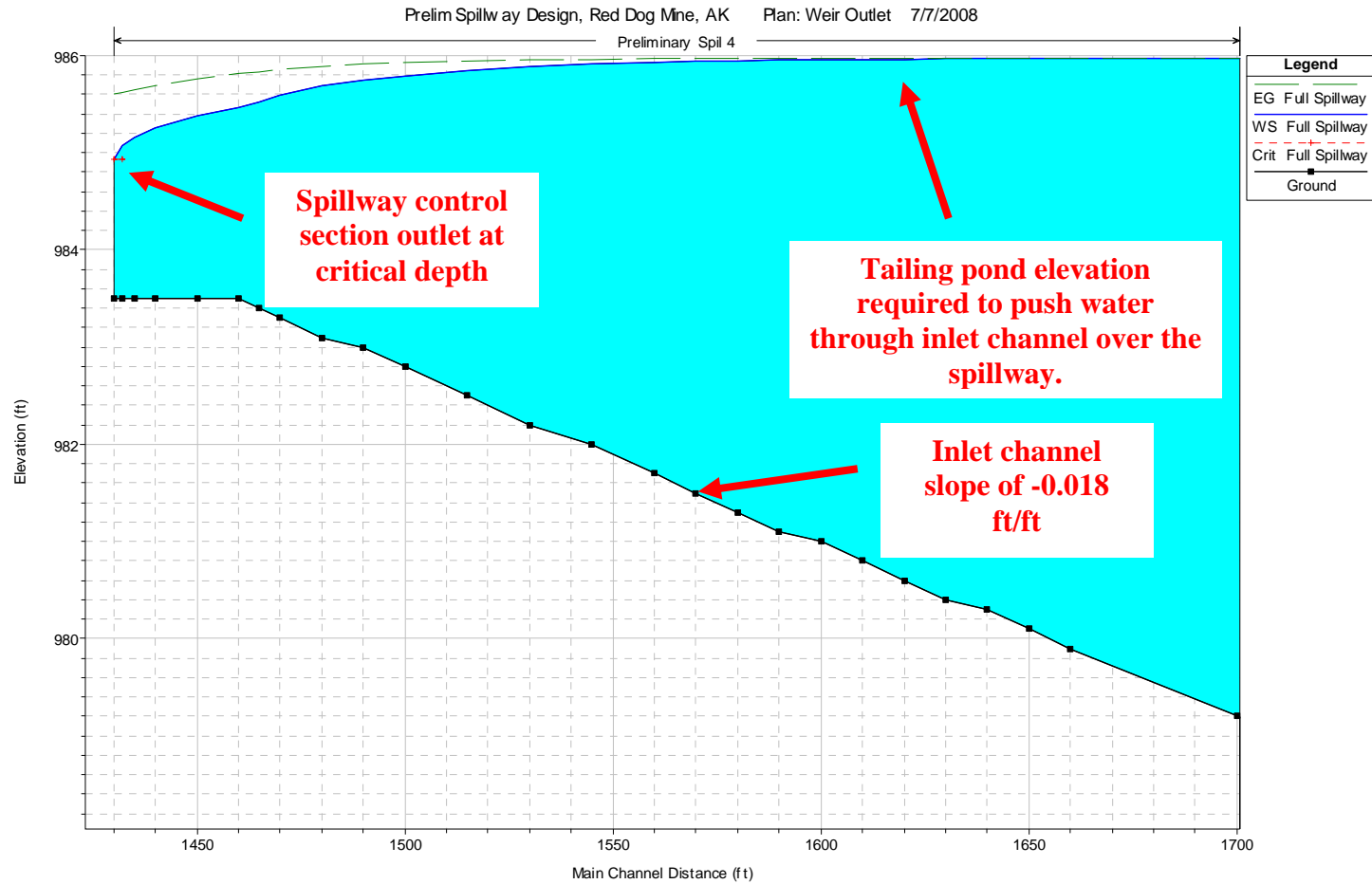
HEC-RAS Output Files for PMF, Inlet Channel Parameters, (USACE, 2004)

Spillway Station	Min Channel Elevation	W.S. Elevation	Velocity Channel	Flow Area	Froude # Channel
	(ft)	(ft)	(ft/s)	(sq ft)	
1860	976.3	985.98	0.15	2876.48	0.01
1800	977.4	985.98	0.21	1959.65	0.01
1760	978.1	985.98	0.28	1504.18	0.02
1700	979.2	985.98	0.45	954.91	0.03
1660	979.9	985.97	0.64	656.66	0.05
1650	980.1	985.97	0.7	601.69	0.05
1640	980.3	985.97	0.75	565.92	0.06
1630	980.4	985.97	0.83	510.12	0.06
1620	980.6	985.96	0.91	464.16	0.07
1610	980.8	985.96	1	421.77	0.08
1600	981	985.96	1.11	382.23	0.09
1590	981.1	985.95	1.19	354.52	0.1
1580	981.3	985.95	1.32	320.82	0.11
1570	981.5	985.94	1.46	290	0.12
1560	981.7	985.93	1.61	262.93	0.14
1545	982	985.91	1.85	229.03	0.16
1530	982.2	985.89	2.12	199.61	0.19
1515	982.5	985.84	2.55	164.05	0.25
1500	982.8	985.8	2.97	142.27	0.3
1490	983	985.74	3.37	125.27	0.36
1480	983.1	985.69	3.61	116.6	0.4
1470	983.3	985.59	4.22	99.35	0.49
1465	983.4	985.52	4.57	91.49	0.55
1460	983.5	985.47	4.78	88.62	0.6
1450	983.5	985.38	5.02	84.27	0.64
1440	983.5	985.25	5.41	77.83	0.72
1435	983.5	985.16	5.74	73.29	0.79
1432	983.5	985.08	6.05	69.38	0.85
1430	983.5	984.93	6.7	62.48	0.99

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URS

Wind Direction Determination

The wind direction was determined using NRCS method 2 of TR-69 (1983), which states, "*Wind direction can be obtained by determining the point on the shoreline over the longest stretch of open water from the dam*". Using this methodology, the wind direction was determined to be northeast towards the tailings main dam. This conservative wind direction was assumed in all three design wind velocity methods, since strong winds in varying directions were witnessed in past field visits (URS, 2005).

Fetch Determination

The fetch length was determined from the longest straight-line fetch across the tailings pond. An effective fetch was not used because the USACE (2003) Coastal Engineering Manual directs use of a straight-line fetch as a better measurement in wave calculations. The straight-line fetch of 1.65 miles was used to remain consistent with report calculations.

Design Wind Velocity: USACE Coastal Engineering Manual

The USACE Coastal Engineering Manual (2006) provides a step-by-step approach for estimating the design wind velocity. The approach involves the following steps:

- Determination of maximum wind velocity from site records
- Adjustment of wind velocity for anemometer height
- Determination of wind velocity duration
- Adjustment of wind velocity from overland to overwater

Determination of Maximum Wind Velocity from Site Records

The maximum wind velocity from Bons Creek Station data from 2000-2007 was 33.84 meters per second. The wind velocities reported at hourly increments for the period 2000-2007 are the maximum values observed during any two seconds in each hour. In view of the short length (i.e., 8 years) of data, the observed maximum value of 33.84 meters per second is multiplied by a factor of 1.17 to estimate the long-term maximum wind velocity. The reasonableness of this multiplying factor is verified using the statistical approach described below.

It is recognized that probabilistic extrapolation from a relatively short data set (i.e., eight yearly maximum values) to large return periods may not be accurate. To estimate the accuracy of such extrapolation, the upper limit of the 95 percent confidence level for the estimated value for a given return period is estimated. This estimated upper limit indicates that there is a 95 percent chance that the expected wind velocity for the given return period is less than this value.

The skew coefficient of the eight values (i.e., -0.41) is relatively small. The skew coefficients of the normal and extreme value Type I probability distributions are zero and 1.14, respectively (Chow, 1964). The normal distribution appears preferable for this set of data because the skew coefficient of the data is closer to zero than 1.14. Using the normal distribution, a 95 percent confidence interval was estimated for the 50-year recurrence interval to obtain a conservative, long-term maximum wind velocity estimate (Appendix B-17). No specific recurrence interval is specified by the Coastal Engineering Manual, so a 50-year recurrence interval was conservatively selected to remain consistent with NRCS guidance in TR-69 (NRCS, 1983).

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The results from the normal distribution analysis indicate that the wind velocity obtained by multiplying the maximum observed wind velocity of 33.84 m/s by a factor of 1.17 (i.e., 39.59 meters per second) is equal to the upper limit of the 95 percent confidence for the estimated 50-yr maximum wind. There is 95 percent confidence that the expected 50-yr maximum wind velocity (obtained from long-term data, if available) would be less than the estimated upper limit value of 39.59 meters per second (or maximum observed value x 1.17). The concurrence of a PMF with a 50-year maximum wind velocity is an extremely low probability event (American Nuclear Society, 1992).

In view of the low probability of a 50-year maximum wind velocity event concurrent with a PMF, the observed maximum value of 33.84 meters per second multiplied by a factor of 1.17, equaling 39.59 meters per second, was considered a conservative maximum wind velocity to estimate the design wind velocity for the USACE approach.

Adjustment of Wind Velocity for Anemometer Height

The anemometer at the Bons Creek station is located at a height of 30 meters above ground surface (Diehl, 2008b). The USACE guidance directs the adjustment for anemometer elevation to 10 meters above the ground surface. Using equation II-2-9 from the USACE Coastal Engineering Manual (2003), the maximum measured wind velocity at 30 meters above the ground surface (39.59 meters per second) was multiplied by a factor of 0.85 to obtain a maximum wind velocity at 10 meters above the ground surface of 33.84 meters per second.

$$U_{10} = U_z \times (\frac{10}{z})^{(1/7)} = 39.59\,m/s \times (\frac{10\,m}{30\,m})^{(1/7)} = 33.84\,m/s$$

Where:

U_{10} = Wind velocity measured at 10 meters above the ground surface (m/s)

U_z = Wind velocity measured at height z above the ground surface (m/s)

z = Height above ground surface (m)

Determination of Wind Velocity Duration and Associated Design Wind Velocity

The wind velocity duration is determined from the adjusted maximum wind velocity, adjusted maximum wind velocity duration and fetch length. The duration of the adjusted maximum wind velocity is determined by the following equation from Figure II-2-2 from the Coastal Engineering Manual at as shown below:

$$t = \frac{1609}{U_f} = \frac{1609}{33.84\,m/s} = 47.55\,s$$

Where:

U_f = Fastest mile velocity (maximum wind velocity)

t = time (s)

The average hourly wind velocity is then determined from the following equation in Figure II-2-1 from the Coastal Engineering Manual:

$$U_{3600} = \frac{U_t}{1.277 + 0.296 \times \tanh(0.9 \times \log_{10}(\frac{45}{t}))} = \frac{33.84\,m/s}{1.277 + 0.296 \times \tanh(0.9 \times \log_{10}(\frac{45}{47.55\,s}))} = 26.63\,m/s$$

Where:

U_t = Wind velocity for a duration of t

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$$U_{3600} = \text{Average hourly wind velocity}$$
$$t = \text{time (s)}$$

Next, the duration for the design wind velocity is derived from Figure II-2-3 and Equation II-2-35 of the Coastal Engineering Manual to be 35.97 minutes as shown below:

$$t_{x,u} = 77.23 \times \frac{X^{0.67}}{u^{0.34} \times g^{0.33}} = 77.23 \times \frac{2{,}650\, m^{0.67}}{33.84\, m/s^{0.34} \times 9.81\, m/s^{0.33}} \times \frac{1\, \text{min}}{60\, s} = 35.97\, \text{min}$$

Where:

$t_{x,u}$ = time required for waves crossing a fetch of length X under a wind of velocity u to become fetch limited

This is then used to determine the design wind velocity overland using the previously used equation from Figure II-2-1 from the Coastal Engineering Manual:

$$U_t = (1.277 + 0.296 \times \tanh(0.9 \times \log_{10}(\frac{45}{t}))) \times U_{3600} =$$

$$(1.277 + 0.296 \times \tanh(0.9 \times \log_{10}(\frac{45}{35.97\ \text{min} \times 60\ \text{sec}}))) \times 26.63\, m/s = 26.86\, m/s$$

Adjustment of Wind Velocity from Overland to Overwater

The Coastal Engineering Manual suggests an adjustment multiplier of 1.2 for converting overland wind velocities to overwater wind velocities. Multiplying the overland design wind velocity by a factor of 1.2 yields a final (overwater) design wind velocity of 32.23 meters per second (72.07 miles per hour) according to the following calculation:

$$U_{overwater} = U_{overland} \times 1.2 = 26.86\, m/s \times 1.2 = 32.23\, m/s =$$

$$32.23\, m/s \times \frac{3600\, s}{1hr} \times \frac{3.28\, ft}{1m} \times \frac{1mile}{5{,}280\ ft} = 72.07\, mph$$

Design Wind Velocity: Nuclear Regulatory Commission Guidance

Guidance for developing combined hydrologic and meteorologic events associated with spillway designs are provided in regulatory guides of the NRC. A commonly used criterion for combined events associated with PMP and PMF determination for the design of safety-related components of nuclear power plants (e.g., spillways) includes a 2-year maximum wind velocity combined with PMP or PMF (American Nuclear Society, 1992).

The technical reasoning for the abovementioned specification is that the joint probability of a wind velocity of return period higher than 2 years concurrent with PMP may be lower than the assumed probability of PMF (i.e., 10^{-6}, approximately). For instance, the probability of a 100-year wind may have to be combined with a 10,000-year flood to obtain a combined probability nearly equivalent to the PMF.

The 2-year maximum wind velocity is equal to the mean annual maximum wind velocity. The mean of the maximum wind velocities observed in the eight-year period from 2000-2007 was 30.11 meters per second with a standard deviation of 2.89 meters per second (See table below). This relatively low standard deviation indicates that the variation in the annual maximum wind velocities has been relatively small. Recognizing that the record length is relatively small (i.e., 8 years), the upper limit of the 95 percent confidence interval for the normal distribution for maximum wind velocity of 32.11 meters per second is considered to be a reasonably conservative estimate of the long-term average or 2-year maximum wind velocity.

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Bons Creek Station Maximum Wind Velocity Data	
Year	Maximum Wind Velocity (m/s)
2000	26.01
2001	30.77
2002	26.11
2003	33.84
2004	29.8
2005	31.49
2006	33.13
2007	29.8
Average	30.11
Stand Dev	2.89
SE	1.02

For comparison, the 95 percent confidence limit of the mean of the annual maximum wind velocities obtained from 8 years of data is computed. Using the record length of 8 years, and the standard deviation of 2.89, the standard error of the mean of the eight values = 1.02 as shown below:

$$S.E. = \frac{\sigma}{\sqrt{n}} = \frac{2.89}{\sqrt{8}} = 1.02$$

Where:

S.E. = Standard error
σ = Standard deviation
n = Number of years of record

The 95 percent confidence limits of the estimated mean of the 2-year maximum wind velocity are 32.11 and 28.11 meters per second as calculated below:

$$C.I. = u \pm t_{\alpha} \times S.E. = 30.11 \pm 1.96 \times 1.02 = 30.11 \pm 2.0$$

Where:

C.I. = Confidence interval
u = Mean
t = Standard normal t-value
α = Significance level

The maximum upper limit at 95 percent confidence level equals 32.11 meters per second. This velocity was selected as the design wind velocity, since there is 95 percent confidence that the expected 2-year maximum wind velocity (obtained from long-term data, if available) would be less than the estimated upper limit value of 32.11 meters per second or 72.07 miles per hour.

Design Wind Velocity: Natural Resources Conservation Service Method

The design wind velocity was determined using wind velocity data obtained from the Bons Creek Station (TCAK, 2008a) at the Red Dog Mine site and from methodology in TR-69 (NRCS, 1983). The

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maximum instantaneous (2-sec) wind velocity recorded at the Bons Creek Station from 2000-2007 was 33.84 meters per second (75.68 miles per hour).

TR-69 specifically references maximum basic wind velocities of a 50-year recurrence in Figure 4. The concurrence of a PMF with a 50 -year maximum wind velocity is a very low probability event. However, given the reference to 50-year velocities in the NRCS methodology, the maximum velocity recorded was multiplied by a factor of 1.5^{1} to account for the small period of recorded wind velocities. The factor of safety of 1.5 was selected in this method to attain a maximum wind velocity of over 100 miles per hour. This maximum wind velocity was comparable to extreme maximum basic wind velocities of 50-year recurrence shown for inland areas of the continental U.S. from Figure 4 of TR-69. Using a factor of safety of 1.5, a maximum wind velocity of 113.52 miles per hour was calculated as shown below:

$$\frac{33.84\,m}{s} \times \frac{3,600\,s}{1hr} \times \frac{3.28\,ft}{1m} \times \frac{1mile}{5280\,ft} = 75.68\,mph \times 1.5 = 113.52\,mph$$

The adjusted maximum velocity recorded was then used with TR-69 Figure 5 to plot overland wind velocity versus wind duration on TR-69 Worksheet D-4. The fetch length was then used with TR-69 Figure 2 to also plot overland wind velocity versus wind duration on TR-69 Worksheet D-4. The design overland wind velocity was then selected using TR-69 Worksheet D-4 where the two plotted curves intersected. This velocity predicts the largest overland wind velocity that will occur based on the time duration allowed for wave development by the fetch length. The design overland wind velocity obtained was 61.0 miles per hour.

The design overland wind velocity was then converted to the final design wind velocity using the fetch length with TR-69 Figure 7 to determine a ratio of overwater to overland wind velocities of 1.18. The final design wind velocity is calculated below:

$$61.0\,mph \times 1.18 = 72.0\,mph$$

NOTE 1: A statistically refined safety factor of 1.17 was later estimated to account for the short record of wind data. This safety factor, based on the 95% confidence interval of a 50-year recurrence interval wind, was deemed a more appropriate, site-specific estimate than the initial 1.5 safety factor based off of general continental US 50-year maximum wind speeds. The 1.17 safety factor was used in the USACE design wind velocity method as described in Appendix B-13. The NRCS wind velocity method was not revised with this safety factor, since a smaller safety factor would only yield a lower design wind velocity than the controlling design wind velocities from the USACE and NRC methods.

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Statistical Justification for Maximum Wind Velocity Multiplying Factor, Analysis Using Normal Distribution

The standard error of the estimates for different return periods obtained by normal distribution is given by,

$S_T = [(1+ K^2 /2)^{0.5}]*$ standard error of the mean of data (Kite, 1977).

Where:

K = frequency factor for the normal distribution.

Computations using this method are shown below:

Computations of Upper Confidence Limit at 95 percent Level using Normal Distribution

Normal distribution – $S_T = [(1+2.06^2/2)^{0.5}]*1.02 = 1.80$				
Return period (T) (yrs)	Frequency factor (K) (Ponce, 1989, Table A-5)[b]	Estimated wind velocity for return period of T = 100 yrs[a] (m/s)	Max. wind velocity at 95 percent confidence level (EM-1110-2-1415, 03/05/93, Page C-7)[c] (m/s)	Ratio of upper 95% confidence interval to maximum wind velocity
50	2.06	36.06	39.59	1.17
		(30.11 +2.89*2.06) 2.89 = standard deviation of data	(36.85+1.96*1.80)	39.59 m/s / 33.84 m/s

[a]Eq 5.1, EM-1110-2-1415, 03/05/93, Appendix C (USACE, 1993)
[b]Table A-5 (Ponce, 1994)
[c]Confidence Levels (USACE, 1993)

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Wave Runup Calculation for Tailings Pond

URS determined wave runup for the IDF on the tailings main dam at ultimate closure configuration using conservative predictive equations out of the USACE "Coastal Engineering Manual" (2003). The expression for maximum wave runup was calculated according to the following equation:

$$R_{max} = H_0 \times 2.32 \times \xi_0^{0.77}$$

Where:

R_{max} = maximum wave runup
H_0 = Significant deep-water wave height (ft) = 3.38
ξ_0 = Surf similarity parameter

GIVEN:

Beach slope of 1V:100H
Wave period of 2.3 sec

ASSUMPTIONS:

$1/30 < \tan \beta$ = embankment slope $< 1/5$; NOTE: The slope of the beach leading to the tailings main dam is gentler than slopes used in the development of these equations, however USACE manual examples for these equations have used slopes as gentle as 1V:80H, similar to the slope of the tailings beach.

$Ho/Lo > 0.007$, where Ho is the significant deepwater wave height and ξo is calculated from the deepwater significant wave height and length.

Field measurements of runup are consistently lower than predictions by the set of equations used here, by as much as a factor of two.

SOLUTION:

Calculation of runup requires determining deepwater wavelength (USACE, 2006):

$$L_0 = \frac{g \times T^2}{2 \times pi} \tanh(\frac{2 \times pi * d}{L}) = \frac{32.174 \ ft/s^2 \times (2.3s)^2}{2 \times pi} \tanh(\frac{2 \times pi * 9.9 \ ft}{L}) = 26.6 \ ft$$

and, from Equation II-4-1, the surf similarity parameter

$$\xi o = \tan \beta \ (Ho/Lo)^{-1/2} = (1/100) \ (3.38 \ ft \ /26.6 \ ft)^{-1/2} = 0.028$$

Maximum runup is calculated from Equation II-4-28

$$R_{max} = 2.32 \ H_o \ \xi_o^{0.77} = 2.32 \ (3.38 \ ft) \ (0.028)^{0.77} = 0.50 \ ft$$

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APPENDIX C
Spillway Design Calculations

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C-1

Flowmaster Output File, Spillway Outlet Channel Analysis Report, (Bentley Systems, 2005)

Spillway Reach	2	3	Units
Friction Method	Manning Formula	Manning Formula	
Solve For	Normal Depth	Normal Depth	
Roughness Coefficient	0.035	0.035	
Channel Slope	0.2514	0.037	ft/ft
Left Side Slope	2.5	2.5	ft/ft (H:V)
Right Side Slope	2.5	2.5	ft/ft (H:V)
Bottom Width	12	40	ft
Discharge	404.8	404.8	ft^3/s
Normal Depth	**1.25**	**1.12**	**ft**
Flow Area	18.9	48.13	ft^2
Wetted Perimeter	18.73	46.05	ft
Top Width	18.25	45.62	ft
Critical Depth	**2.7**	**1.43**	**ft**
Critical Slope	0.01498	0.01653	ft/ft
Velocity	**21.42**	**8.41**	**ft/s**
Velocity Head	7.13	1.1	ft
Specific Energy	8.38	2.22	ft
Froude Number	3.71	1.44	
Flow Type	Supercritical	Supercritical	

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NRCS Outlet Channel Analysis Method, Equations and Layout

EQUATIONS					
Mannings	**Q**	=	$1.486AR^{2/3}S^{1/2}/n$		
Area Flow	**A**	=	$bd+zd^2$		
Hydraulic Radius	**R**	=	A/P		
Wetted Perimeter	**P**	=	$b+2d(1+z^2)^{1/2}$		
Velocity	**V**	=	Q/A		

TYPICAL SECTION

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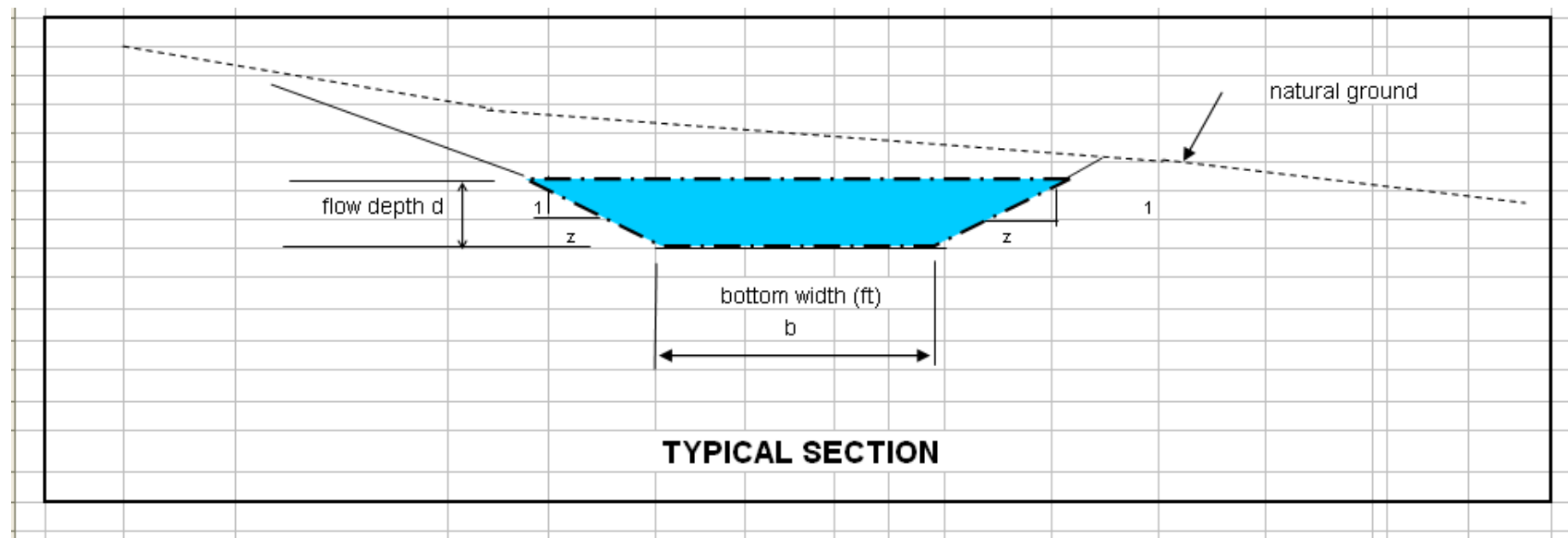
NRCS Outlet Channel Analysis Method, Critical and Normal Depth Calculations, (NRCS, 1951; NRCS, 1956; NRCS, 1997)

OPTION C: 12 FOOT BTM SECTION & DECREASE SLOPE AND VELOCITY

	Station	Elevation	Elevation	Diff	Distance	Slope	Mannings	Flow Q	Bottom Width	Side Slope	$nQ/(b^{8/3} \cdot s^{1/2})$	$\dfrac{1.486\,((1+z(d/b))(d/b))^{5/3}}{/(1+d(d/b)\sqrt{(z2+1)})^{2/3}}$	Normal Depth d_n	Qc	z/b	Critical Depth d_c
				ft	ft	ft/ft	"n"	cfs	ft	z:1			ft			ft
Inlet - start	1930	975							330	1.5						
			-8.5	470	-0.0181	0.035	404.8	varies	1.5						
Control Section	1460	983.5						40	2.5						
			0	30	0.0000	0.035	404.8	40	2.5						
Control Section	1430	983.5						40	2.5						
			18.5	501	0.0369	0.035	404.8	40	2.5	0.00394	0.0039	1.10	406.32	0.06	1.43
	929	965													
			190	756	0.2513	0.035	404.8	12	2.5	0.03744	0.0376	1.18	404.60	0.21	2.70
	173	775													
End Channel															
Start Impact															

NRCS ES-55 Sht 1 of 4   ES-24 sht 1 of 4

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Riprap Design for Spillway Outlet Channel Reaches

The following design procedure steps were followed in determining the riprap size from Chapter 6 of "Design of Roadside Channels with Flexible Linings" (FHWA, 2005):

Step 1. The channel slope and channel shape were determined from the spillway channel topography and alignment constraints. The design discharge was determined to be 404.8 cubic feet per second (Appendix B-7).

Step 2. The initial D_{50} sizes were determined as seen in Table 7.2 using USACE equations shown in Appendix C-6. The specific weight of the riprap was assumed to be 165 pounds per cubic foot based on testing of Okpikruak shale for the preliminary design (URS, 2008b). The actual rock to be used at final design should be tested for specific weight.

Step 3. The average flow depths in the channel reaches were determined using Flowmaster Software . The depths for each channel are in Appendix C-2.

Step 4. The Manning's n value was assumed to be 0.035 for jagged, irregular rock (Brater and King, 1976).

Step 5. The Reynold's number was calculated for each of the channel reaches using the appropriate equation at Appendix C-6. The Reynold's numbers for all reaches were equivalent to a Shield's Parameter of 0.15 and a Safety Factor (SF) of 1.5 according to Table 6.1 on p. 6-4.

Step 6. Equations 6.8 and 6.11 were used to determine the required D_{50} for the given channel reach, based on slope.

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Spillway Outlet Channel Riprap Sizing Calculations, Spreadsheet 1 of 2 (USACE, 1994; FHA, 2005)

Reaches to be Riprapped	S = Slope = $\Delta El/\Delta L$	Q_D = Design Flow Rate from HEC-HMS	B = Bottom Channel Width	T = Top Channel Width	d_n = Normal Depth from Flowmaster	z = Side Slope (V:H =1:z)	$'D_{30}$ = $1.95S^{0.555}(Q/B)^{2/3}/g^{1/3}$	$'D_{50}$ = $D_{30}(D_{85}/D_{15})^{1/3}$ 'D85/D15=2	b =effective roughness concentration = $1.14(D_{50}/T)^{0.453}(d_n/D_{50})^{0.814}$
(ft)	(ft/ft)	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	
1+38 to 2+32	0.50	404.80	12.00	17.15	1.03	2.50	4.36	5.49	0.17
2+32 to 4+40	0.25	404.80	12.00	18.25	1.25	2.50	2.97	3.75	0.23
6+00 to 9+29	0.25	404.80	12.00	18.25	1.25	2.50	2.97	3.75	0.23
9+29 to 11+00	0.04	404.80	40.00	45.60	1.12	2.50	0.46	0.58	0.27

Fr = Froude Number = $V/(gA/T)^{0.5}$	$f(Fr)$ = $(0.28Fr/b)^{log(0.755/b)}$	$f(REG)$ = $13.434(T/D_{50})^{0.492}$ $b^{1.025(T/D50)^{0.118}}$	$f(CG)$ = $(T/d_n)^{-b}$	n = Manning's Roughness Coefficient	P = Wetted Perimeter	A = Area	R = Hydraulic Radius = A/P	P/R	V = Velocity = Q_d/A
					(ft)	(ft²)	(ft)	(ft/ft)	(ft/s)
5.08	3.81	3.03	0.61	0.04	17.55	15.01	0.86	20.51	26.96
3.71	2.20	4.70	0.54	0.05	18.73	18.91	1.01	18.56	21.41
3.71	2.20	4.70	0.54	0.05	18.73	18.91	1.01	18.56	21.41
1.45	1.20	12.15	0.37	0.05	46.03	47.94	1.04	44.20	8.41

R_e = Reynolds Number = $(gd_n S)^{0.5}D_{50}/v$	K = Ratio of Channel Side to Bottom Shear Stress = $0.066z+0.67$	β = $tan^{-1}(cos(atan(S)))/(2sin(atan(1/z))\eta tan(\Phi)+sin(atan(S)))$	Δ = $K(1+sin(atan(S)+\beta))tan(\Phi)/2(cos(atan(1/z))tan(\Phi)-SFsin(atan(1/z))cos(\beta))$	η = stability number = $\tau_s/F \cdot (\gamma_s-\gamma_w)D_{50}$	τ_s = Side Slope Shear Stress = $K\tau_d$	τ_d = Channel Bottom Shear Stress = $\gamma_w d_n S$	τ_p = Adopted Design (Permissible) Shear Stress = $SF\tau_d$	$D_{50\ Eqn\ 6-8}$ = $SFd_n S\Delta/F \cdot (SG-1)$	$D_{50\ Eqn\ 6-11}$ = $SFd_n S/F \cdot (SG-1)$
1836757.21	0.84	0.29	2.10	0.32	26.83	32.14	48.20	3.12	6.54
979625.67	0.84	0.30	1.88	0.28	16.37	19.61	29.41	1.90	3.58
979625.67	0.84	0.30	1.88	0.28	16.37	19.61	29.41	1.90	3.58
55042.20	0.84	0.28	1.64	0.24	2.16	2.59	3.88	0.25	0.41

1. USACE equations can be found in "Hydraulic Design of Flood Control Channels", p.3-6 to 3-8 (1994) [Assumes D_{85}/D_{15} = 2]
*All other equations can be found in the FHWA "Design of Roadside Channels with Flexible Pavement", Chapters 3 & 6 (2005)

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Spillway Outlet Channel Riprap Sizing Calculations, Spreadsheet 2 of 2 (USACE, 1994; FHA, 2005)

				R_c/T = Radius of Curvature/Top Channel Width	K_b = Ratio of Channel Bend to Bottom Shear Stress	τ_b = Shear Stress at Channel Bend = $K_b\tau_d$	Δd = Additional Freeboard Required because of Super-elevation = V^2T/gR_c
SF = Safety Factor = 1.5 (for Re over 2×10^5)							
Φ = Angle of Repose of Riprap							
v = Kinematic Viscosity							
R_c = 750 feet from ArcGIS				41.10	1.05	20.59	0.35
SG = 2.65							
Reaches to be Riprapped	S = Slope = ΔEl/ΔL	D_{50}	Spillway Channel Freeboard				
(ft)	(ft/ft)	(ft)	(ft)				
0+40 to 2+32	0.05	6.54	2.68				
2+32 to 4+40	0.25	3.75	2.58				
6+00 to 9+29	0.25	3.75	2.92				
9+29 to 11+00	0.04	0.58	2.22				

*All equations can be found in the FHWA "Design of Roadside Channels with Flexible Pavement", Chapters 3 & 6 (2005).

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Stilling Basin Design

The stilling basin was designed using the FHWA, "Hydraulic Design of Energy Dissipators for Culverts and Channels" (2006) and is detailed in Drawing TCP-17. The calculations were carried out in accordance with the general design procedure on page 8-3 of Chapter 8, "Stilling Basins". All figures and equations used in the calculation are from this chapter. The following design steps were taken to carry out the stilling basin calculations.

Step 1. The velocity (V_o), normal depth (y_o) and Froude number (Fr_o) at the spillway outlet channel into the stilling basin were determined in Flowmaster to be 21.42 feet per second, 1.25 feet and 3.38, respectively. The Froude number of 3.71 in the Flowmaster output is calculated using a hydraulic depth versus channel depth. Channel depth was used in this calculation to determine the more conservative Froude number for the design and to remain consistent with the equation in step 1 of the design procedure on p. 8-5 as shown below.

$$Fr_0 = \frac{V_0}{\sqrt{gy_0}} = \frac{21.42\,ft/s}{\sqrt{32.174\,ft/s^2 \times 1.25\,ft}} = 3.38$$

Where:

Fr_0 = Froude number at channel outlet
V_0 = Velocity at channel outlet (ft/s)
g = Acceleration due to gravity (ft/s^2)
y_0 = Depth at channel outlet (ft)

Step 2. Two methods were used in determining the allowable exit velocity, V_{allow}, and estimated depth in Red Dog Creek. The first method was based on use of the Manning's equation and the second was based on the analysis of the creek bed material. The following data were used to determine a range of creek velocities and depths using the Manning's equation:

- Geological boring data and photos relevant to Manning's "n" roughness coefficient determination (URS, 2006b)
- Flow data from Red Dog Mine Gage Station 140 (TCAK, 2008b)
- Red Dog Creek width and slope at the Spillway Outlet (Aero-Metric Anchorage, 2007)

The Red Dog Creek was determined to be an alluvial bed made up of gravel and silt. A boring from an upstream reach of the Red Dog Creek at the toe of the original starter dam for the tailings pond described the make up of the creek as gray, gravelly silt to silty gravel below a thin layer of tundra organic rootmat (URS, 2006b). This reach is prior to a confluence of two similar size reaches that flow past the spillway outlet area, but should be indicative of the makeup of that part of the creek. This description of the creek bed, minus tundra organic rootmat, is consistent with the visual inspection of the spillway outlet area from aerial photos (Aero-Metric Anchorage, 2007) and field accounts (Learned, 2008). This type of natural stream that's clean, winding, having some pools and shoals is estimated to have a Manning's "n" value of 0.033 (Chow, 1964; USACE, 2004).

The Red Dog Mine Gage Station 140 has recorded a maximum flow rate of 170 cubic feet per second within 24 hours after a 1.5 inch precipitation over a 4.37 acre catchment (URS, 2008a). During a PMP event of 13.26 inches, almost an order of magnitude larger in precipitation, it can be conservatively assumed that the creek flow rate will also increase almost an order of magnitude to as much as 1,500 cubic feet per second. Since the spillway outlet into Red Dog Creek is further down the creek from Red Dog Mine Gage Station 140, the flow will only increase from additional catchment runoff.

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In addition to the determination of an approximate "n" value and flow rate for the Red Dog Creek area at the spillway outlet, the bed slope and creek width were determined to be 1.5 percent and 27 feet, respectively, from survey data (Aero-Metric Anchorage, 2007). Based on contours in the floodplain of the creek, the overbank slope is estimated to be 10H:1V. Given these parameter values and assuming uniform flow, the depth at the spillway outlet area of Red Dog Creek can be calculated iteratively by the following Manning's equation (Chow, 1964) as shown below:

$$Q = \frac{1.486 A R^{2/3} S^{1/2}}{n} = \frac{1.486(Wd + zd^2)(\dfrac{Wd + zd^2}{W + 2\sqrt{(zd)^2 + d^2}})^{2/3} S^{1/2}}{n} =$$

$$\frac{1.486 \times (27\,ft \times d + 10 \times d^2)(\dfrac{27\,ft \times d + 10 \times d^2}{27\,ft + 2 \times \sqrt{(10 \times d)^2 + d^2}})^{2/3} 0.015^{1/2}}{0.033}$$

Where:

      Q = Flow rate (cfs)
      n = Manning roughness coefficient
      W = Creek width at spillway outlet area (ft)
      d = Creek depth at spillway outlet area (ft)
      S = Slope of Red Dog Creek at spillway outlet area (ft/ft)
      z = Side slope of channel z:1/H:V

The velocity of Red Dog Creek at the area of the spillway outlet can then be calculated by the following equation:

$$V = \frac{Q}{A} = \frac{Q}{Wd + zd^2}$$

Using these equations, the following tailwater velocities and depths were determined as shown in the table below:

Tailwater Velocities and Depths at Various Discharge Rates in Red Dog Creek

Discharge	Average water depth	Average velocity
(cfs)	(ft)	(ft/sec)
1500.00	3.02	8.67
800.00	2.22	7.31
648.41	2.00	6.90
604.80	1.93	6.77
200.00	1.08	4.90

The second method used to determine the allowable exit velocity from the spillway outlet was a design procedure from the NRCS Technical Release No. 25, "Design of Open Channels" (1977). Figure 6.1 on page 6-2 shows the channel evaluation procedure. The earth materials of the banks and bed of the creek were classified as gray gravelly silt (ML) to silty gravel (GM) (URS, 2006a). It is unknown whether the

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creek is sediment laden flow versus sediment free flow or whether it's discrete versus coherent material, so a range of allowable velocities were determined for each case, as shown in the table below:

Allowable Velocities for Red Dog Creek Based on Sediment Characteristics

Creek Bed Properties	Basic Velocity	Correction Factor A[1]	Correction Factor B[1]	Correction Factor C_e[1]	Correction Factor D[1]	Correction Factor F[1]	Allowable Velocity
(PI = Plasticity Index)	(ft/s)	(R_c/b=62.5)	(z=2.5)	(Void Ratio = 1.0)	(depth = 2 ft)	(Frequency < 0.01%)	(ft/s)
Sediment Laden, PI=20, GM[2]	5.50	1.00	N/A	1.00	0.93	1.64	8.39
Sediment Laden, PI=10, ML[2]	3.50	1.00	N/A	1.00	0.93	1.64	5.34
Sediment Laden, Discrete[2]	2.00	1.00	0.82	N/A	0.93	N/A	1.53
Sediment Free, PI=20, GM[2]	4.00	1.00	N/A	1.00	0.93	1.64	6.10
Sediment Free, PI=10, ML[2]	2.00	1.00	N/A	1.00	0.93	1.64	3.05
Sediment Free, Discrete[2]	2.00	1.00	0.82	N/A	0.93	N/A	1.53

Note 1: Basic velocities and correction factors were taken from Figure 6.2 on p. 6-4 of "Design of Open Channels" (NRCS, 1977)
Note 2: Allowable velocities for coherent (plastic) materials were calculated with the following equation: Basic velocity x D x A x F x C_e. Allowable velocities for discrete particles were calculated with the following equation: Basic velocity x D x A x B.

An allowable velocity range from 1.53 to 8.39 feet per second was determined using the two methods above. The low end range of the allowable velocities represents silts and fine sands (ML), whereas the upper end range represents silty gravels and gravel-silt-sand mixtures (GM). The representative boring from an upstream reach of Red Dog Creek describes the Red Dog Creek as being more silty gravels than pure silts, which favors using a design outlet velocity in the higher end of the allowable velocity range. Using this reasoning, an allowable exit velocity of less than 7 feet per second was selected for this design. Further field investigation of the Red Dog Creek will be necessary to determine a more accurate allowable exit velocity at final design.

Given the range of depths possible in the Red Dog Creek, a height of approximately 2.0 feet was used in stilling basin sizing calculations. A minimum of 648 cubic feet per second flowing through Red Dog Creek during a PMF event was chosen as a conservative assumption. It is more probable that a higher flow will occur in Red Dog Creek to control the incoming velocity from the spillway outlet.

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Step 3. The estimate of the conjugate depth for the channel outlet conditions was determined using the equation shown below:

$$y_2 = \frac{Cy_0}{2}(\sqrt{1+8Fr_0{}^2}-1) = \frac{1.0 \times 1.25}{2}(\sqrt{1+8 \times 3.38^2}-1) = 5.38$$

Where:
      y_2 = Conjugate depth (ft)
      y_0 = Depth approaching the jump (ft)
      C = Ratio of tailwater to conjugate depth, TW/y_2 (1.0 for free hydraulic jump)
      Fr_0 = Approach Froude number

The estimated tailwater at the exit of the stilling basin was determined using HEC-RAS modeling from the entrance of Red Dog Creek at station 0+28 back to the exit of the stilling basin at station 1+46. The depth of Red Dog Creek was assumed to be 2 feet as a downstream boundary condition and the head and expansion losses from the stilling basin exit to the creek entrance were calculated to estimate a tailwater height of 2.77 feet at the stilling basin exit at station 1+46.

The estimated conjugate depth is much higher (5.38 feet>2.77 feet) than the expected tailwater produced from Red Dog Creek at the stilling basin exit, so a jump will not form and a basin is needed.

Step 4. The bottom elevation, z_1, of the stilling basin is estimated to be 4 feet below the Red Dog Creek channel (El. 771) in order for the flow exiting the basin to equal the approximate depth of the Red Dog Creek tailwater (~2.77 feet).

The slope of the transition entering the basin, S_T, was set at 2H:1V to achieve a Froude number of 4.7 prior to the hydraulic jump in the stilling basin. This steeper transition slope was selected because a Froude number over 4.5 results in a more efficient jump (Chow, 1959).

The basin inlet transition length, L_T, was calculated using the equation below from p. 8.9:

$$L_T = \frac{z_0 - z_1}{S_T} = \frac{790\,ft - 771\,ft}{0.5} = 38\,ft$$

A basin entrance width of 12 feet was selected, since a steady hydraulic jump will occur without expanding the stilling basin transition width. The maximum basin width, W_B, was validated with the equation below from p.8.3:

$$W_B \le W_0 + \frac{2L_T\sqrt{S_T{}^2+1}}{3Fr_0} = \frac{2 \times 38\,ft\sqrt{0.5^2+1}}{3 \times 3.38} = 20.38\,ft$$

The basin entrance width is less than the maximum allowed width of 20.38 feet.

The normal velocity ($V_1 = 27.03$), depth ($d_1 = 1.03$) and respective Froude number ($Fr_1 = 4.70$) at the entrance to the stilling basin were calculated using the Manning's Equation in Flowmaster (Bentley Systems, 2005) and Equation 8.1 on p. 8-2 for the Froude number using channel depth instead of hydraulic depth as shown below:

$$Fr_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{27.03\,ft/s}{\sqrt{32.174\,ft/s^2 \times 1.03\,ft}} = 4.70$$

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Where:

Fr_1 = Froude number at basin entrance
V_1 = Velocity at basin entrance (ft/s)
g = Acceleration due to gravity (ft/s^2)
y_1 = Depth at basin entrance (ft)

Step 5. The conjugate depth for the hydraulic conditions entering the basin was calculated using Equation 8.4 on p. 8-3 as shown below:

$$y_2 = \frac{Cy_1}{2}(\sqrt{1+8Fr_1^2} - 1) = \frac{1.0 \times 1.03}{2}(\sqrt{1+8 \times 4.70^2} - 1) = 6.35\,ft$$

Where:

y_2 = Conjugate depth (ft)
y_1 = Depth approaching the jump (ft)
C = Ratio of tailwater to conjugate depth, TW/y_2 (1.0 for free hydraulic jump)
Fr_1 = Approach Froude number

The basin length was calculated using Figure 8.2 on p. 8-4 as shown below:

$$L_B = \frac{L_B}{y_2} \times y_2 = 6.1 \times 6.35\,ft = 38.74\,ft$$

Where:

L_B = Basin length (ft)
y_2 = Conjugate depth (ft)
L_B/y_2 = Value from Figure 8.2 on p. 8-4 for free jump with Fr_1 = 4.7

The basing length will be adjusted to 40 feet for construction purposes.

The slope leaving the basin exit (S_S) was chosen to be 2H:1V as recommended in step 4 of the general design procedure on p. 8-5. Therefore, to bring the basin sill to El. 775 to meet the Red Dog Creek channel bottom, the length of the basin floor to the sill (L_S) was calculated by dividing the rise in elevation by the slope as shown below:

$$L_S = \frac{z_3 - z_2}{S_S} = \frac{775\,ft - 771\,ft}{.50} = 8\,ft$$

Where:

L_S = Length from basin floor to sill (ft)
z_3 = Elevation of sill (ft)
z_2 = Elevation of basin (ft)
S_S = Slope leaving the basin (ft/ft)

The exit elevation of the stilling basin was designed to be El. 775 to meet the bed elevation of Red Dog Creek. The verification that sufficient tailwater exists to force the hydraulic jump was calculated using the equation on p. 8-8 as shown below:

$$y_2 + z_2 < z_3 + TW = 6.35\,ft + 771\,ft < 775\,ft + 2.77\,ft = 777.35\,ft \sim 777.77\,ft$$

Where:

y_2 = Conjugate depth (ft)
z_2 = Elevation of basin (ft)
z_3 = Elevation of sill (ft)

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TW = Tailwater elevation at stilling basin exit (ft)

The calculated values above indicate that sufficient tailwater is available to produce a hydraulic jump. Further field investigation of Red Dog Creek will be necessary to verify the depth of water expected in Red Dog Creek during a PMF.

Thus, the total length of the basin (including inclined portions) = 38+40+8 = 86 feet. Further calculations are not needed in this case, since the stilling basin will not continue at the same slope as the channel upstream of the drop. The design slope for the downstream channel is almost zero to minimize velocities in the downstream channel.

To reduce the exit velocity into Red Dog Creek, the stilling basin apron was widened from the basin exit at a flare of 15:1 (length:width) for a distance of 146 feet at a negligible bed slope. The widened bed width is 28 feet at the end of the basin apron. The basin apron will be riprapped until the velocity in the channel is less than 7 feet per second. A 5-foot deep rock toe (width = 3 feet) will be provided at the end of the riprapped apron (station 0+85) to prevent scour at the outlet of the apron. The apron riprap was sized using Riprap Design Systems Software (West Consultants Inc., 2005). This software uses multiple methods for riprap sizing. A riprap of 2 feet D_{50} was selected using the USGS method since it produced the most conservative estimate.

After station 0+85, the apron will continue without riprap to the entrance of Red Dog Creek. The exit velocity into Red Dog Creek is 6.57 feet per second as determined by the HEC-RAS analysis, which is less than the design allowable velocity. However, further field work will have to be conducted to better estimate the allowable exit velocities in Red Dog Creek for the final design. The outlet area at Red Dog Creek will require a site survey at final design to refine the channel bed depth at the entrance to the creek.

Soil borings in the vicinity of this stilling basin indicate that the bottom of the stilling basin may be in shale. Depending on the competence of this shale, further protection may or may not be required. If further protection is required, riprap sizing in the stilling basin was estimated for the maximum channel velocity of 27.0 ft/sec leading into the stilling basin using USACE and FHWA (1994; 2005) methods, as used in the outlet channel riprap sizing. The 50 percent slope of the stilling basin inlet channel is outside of the USACE slope maximum of 20 percent for riprap calculations, but still provides a rough estimate comparable to the FHWA method. The riprap sizes estimated for the stilling basin were D_{50}'s of 5.49 feet and 6.54 feet, using the USACE and FHWA methods respectively (Appendix C-6).

These required stone sizes may not be available in the vicinity. Therefore, use of gabions made of galvanized, PVC-coated or other weather-proof synthetic wires may be considered. Alternatively, concrete blocks cast in suitable shapes may be considered (e.g., rectangular, cubical, tetrapod, quadripod, etc.).

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