4. Mine Site Water Management and Water Storage Facility

4.1 SITE AND DEVELOPMENT DESCRIPTION

4.1.1 Site and Facilities Development

As mentioned above, the geology and topography lead to a project covering two distinct geographic areas. The mineralized deposits to be exploited by the three pits (Kerr, Sulphurets, and Mitchell) are located within the watershed of the tributary creeks of the Unuk River. The ore processing facilities and the TMF are situated in the neighbouring valley which flow to the Teigen and Treaty Creek tributaries of the Bell-Irving River. This section of the report addresses site conditions and water management in the former area, which is referred to as the “Mine Site”.

4.1.2 Geohazards

The principal geohazards at the Mine Site are associated with potentially marginal slope stability in three main areas:

- Along the flanks of the retreating glaciers, upstream of the Mitchell tunnel intake, where the removal of ice confinement may trigger localized slope instability resulting in temporary flow blockage;
- At the slopes in the altered rock mass upstream and downstream of the Mitchell pit, where evidence of marginal natural slope stability has been reported, i.e. open crevices, and localized deteriorating slope surfaces. Slope instability downstream of the pit could interfere with rock storage operations but will not affect drainage and water treatment aspects; and
- Along the flanks of McTagg Creek, where toppling of the steeply inclined sedimentary formations striking along the creek could be triggered by rainfall and/or snowmelt infiltration, interfering with rock storage operations.

The potential occurrence of these geohazards has been taken into account in the location of the tunnel portals and other structures, and preliminary mitigation measures have been proposed, i.e. surface water collection ditches, raise bored shafts and dewatering adits, and mining sequences that stabilize the snowfields slide. Recommendations for further investigation and assessment of the extent and potential for the amplification of the geohazards are provided in Section 4.8 below. *Pertinent data collected by other parties, such as BGC Engineering Consultants (BGC), needs to be more closely integrated for a more comprehensive assessment of the geohazards.*

4.1.3 Hydrology

The Mine Site is drained by Mitchell Creek which flows into Sulphurets Creek and then ultimately into the Unuk River. Surface elevations at the Mine Site range from 550 m to over 2,100 m. There are five glacier systems in the headwaters of Sulphurets Creek and its two major tributaries, Mitchell and McTagg Creeks, and numerous permanent snow fields at the higher elevations. The glaciers are retreating and contribute to the mean annual runoff of the creeks.

The estimated average annual precipitation at the Mine Site ranges from 1,650 mm at the lower elevations, to 3,000 mm at the higher elevations. Peak runoff occurs in June associated with snowmelt and rainfall and to a lesser extent in September due to rainfall events. This data is based on 20-year records at the Eskay climatic station to the north-west and a station at the site that has been in operation since 2008.

The methods used to determine the climatologic and hydrologic parameters of the site are appropriate. They should be considered approximate because of the limited amount of site-specific data that can reasonably be collected and the local variability in the parameters due to the large variation in altitude, slope aspects and distance from the ocean. *The Board suggests that the Design Team update the regional analysis of precipitation, snowpack and runoff utilizing long term meteorological and gauging stations operated by Environment Canada, other governmental or private entities with data up to 2015 and, secondly, perform sensitivity design calculations that assume the estimated parameters are either over or underestimated.* The result of the regional study will provide more information on precipitation and runoff during extreme dry and wet periods and the spatial variability of these parameters. These considerations also apply to the climate and hydrology parameters estimated for the TMF site (Section 5.1.3 below).

The cofferdam and diversion tunnels used in the construction of the WSD are sized for the 25-year, 24-hour storm event. Consideration should be given to increasing this design criterion to at least a 50-year event to be more in line with current practice. This reduces the probability of an event exceeding the diversion tunnel capacity from 8% to 4% in a two-year period.

The capacity of the WSD is sized, and will be operated, to manage the 200-year annual cumulative inflow, without discharge, in order to protect downstream water quality. The spillway is designed for the Probable Maximum Flood (PMF) for dam safety reasons for both during operations and after closure periods.

Non-contact water runoff diversion ditches are sized for the 24-hour, 200-year return period precipitation plus glacial melt flows. *Consideration should be given to increasing the hydraulic capacity of the ditches to handle the instantaneous peak flows during this flood event.* Because of worker safety considerations, adequate drainage and underground storage capacity for the 1,000-year, 24-hour event volume has been provided for the underground workings.

Contact water diversion ditches and tunnels are either designed for the 200-year average flow for conditions when overtopping results in contact water flows being directed towards the WSD, or the 200-year, 24-hour peak flow when overtopping flows would discharge to downstream natural drainages.

4.1.4 Hydrogeology

The WSF and RSF are located within the Mitchell Valley, a U-shaped drainage in terrain with high topographic relief. General site conditions fit well with the concept of mountain-block groundwater flow systems, with upward flow in the valley bottom and a component of down-valley flow in shallow sediments beneath Mitchell Creek. The high annual precipitation for the WSD area with substantial snow pack development (~ 1 m/yr. snow water equivalent) supports appreciable recharge to bedrock flow systems and the RSF in particular. The bedrock that underlies the RSF and WSF consists of steeply dipping sedimentary units, gently folded, with several major thrust faults and secondary faulting. There are relatively thick deposits of unconsolidated sediments on the valley floor, which thin on the mid-slopes and are generally absent at higher elevation.

The scope of the hydrogeologic field investigations completed in the Project area, and the groundwater models developed to aid data interpretation, engineering design, and environmental assessment are consistent with PFS-level studies. Recommendations for future site investigation are discussed in Section 4.8.

The topographic profile of the Mitchell Valley promotes secure lateral hydrodynamic containment of the mine contact water in the WSD impoundment and for the waters infiltrating through the RSF. The RSF is to be built up from the centre of the valley and placed against the south valley wall. The highest elevation of rock-fill placement does not exceed the elevation of the water table in Sulphurets ridge. The seepage pathway of concern for which defensive measures must be set in place is in the lower part of Mitchell

Valley, upstream of the confluence with Sulphurets Creek. Similar topographic conditions provide hydrodynamic containment in the McTagg Valley.

4.1.5 Water Chemistry

Waters in the Mitchell and Sulphurets Creeks are naturally impacted by the surface exposure of mineralized bedrock. The background levels of aluminum, cadmium, chromium, copper, zinc and selenium have exceeded British Columbia Water Quality Goals (BCWQGs) and relatively high suspended solids concentrations occur. This effect is still evident in the water quality in the Unuk River, two kilometres downstream from the Sulphurets confluence, where aluminium, cadmium, chromium, copper and zinc currently exceed BCWQGs. Further downstream, at the international border, water quality in the Unuk River under natural conditions still exceeds BCWQGs for aluminium, cadmium, chromium, copper, lead and zinc.

4.1.6 Strategy for Water Management

The Mine Site water management plan appropriately separates non-contact runoff water from potentially impacted contact waters derived from mine disturbed areas and naturally occurring mineralized rock. The non-contact water is discharged to the downstream reaches of Sulphurets Creek, while the contact water is collected in the WSD for treatment and then discharge. This plan incorporates the elements that are needed to cope with the difficult conditions at the site, i.e. it considers geologic hazards and includes avalanche controls and redundant tunnels, amongst others. It also incorporates an innovative method of tunnelling under the glacier to collect non-contact water using tunnels and vertical borings. This approach has been used successfully in Norway to supply water to a hydropower plant. *The Board recommends that a field trial be undertaken of this approach as the design details are being finalized.*

The Mine Site facilities, including the ore stacking area, the crusher system and the RSF are contained in the upper Mitchell Creek that naturally drains to the location of the proposed WSD. The water management plan contemplates clean water diversion of approximately 60% of this watershed area via:

- The four parallel Mitchell-Sulphurets tunnels, each about 6.5 km long;
- Two McTagg tunnels, each about 5 km long;
- Two diversion ditches located on the western and northern flanks of the RSF respectively; and
- Several smaller pipe and diversion ditch systems that divert sub-catchments of the WSF.

Flow in the Mitchell tunnels and, during the later mine life stages, within the McTagg tunnels will be used to generate hydroelectric power.

Contact water from the RSF, the initial Mitchell Pit and the lower portion of the snowfield slide area (east of Mitchell Pit) will be directed to the WSD for storage, treatment and discharge. Contact water from the Mitchell pit will be conveyed to the WSF by the Mitchell Valley Drainage Tunnel. After block caving commences under the Mitchell Pit, drainage of the workings will involve pumping from the Mitchell Underground Drainage Tunnels either to the WSF or directly to the water treatment plant.

The Board has reviewed the layouts developed and concurs that the location of the various drainage ditches and tunnels, and the WSD and its associated seepage collection pond, are appropriate. The Board recognizes that, as further design details are developed, there will likely be some changes in locations and sizes of the facilities to better suit actual field conditions.

The risks associated with the diversion systems are the geologic and avalanche hazards. The designers have recognized this and are proposing both proactive engineered mitigation measures, as well as contingency measures in the event that a failure occurs. Risks of underground tunnel failure have been

mitigated by providing redundant tunnels in all cases except for the Mitchell Valley tunnel. In the event of a failure in this tunnel, the contact water would flow along the RSF into the WSD, thus avoiding a release to Sulphurets Creek.

In addition, surface ditches are to be constructed along the flanks of the final rock storage surface to convey surface water after closure. These facilities, including the tunnels, are to be maintained for an extended period of time, after closure, for potential hydropower development, i.e. the McTagg tunnel, and to minimize water treatment flows. However, all tunnels can be eventually plugged, if required, with the ditches routing surface water around the RSF.

4.2 TUNNELS

4.2.1 Introduction

Water management within the Mine Site and the RSF area is to be accomplished through an extensive tunnel system of about 40 km total length at start-up, of which about 15 km are twin tunnels for redundancy. Over the life of the mine about 10 tunnels are constructed in this area for non-contact and contact water conveyance. These tunnels would be excavated at different stages of mine development. The non-contact water tunnels, WSD diversion, Mitchell-Sulphurets, and McTagg tunnels will be constructed at the same time as the WSD, followed by the contact water tunnels for Mitchell Pit (in the north wall of the pit), Mitchell Valley, and Mitchell underground drainage at later stages during mine life. Some of these tunnels are intended to be operated for a period of time after mine closure.

4.2.2 Tunnel Alignments

The proposed tunnels are to be excavated in different lithological formations of varied strengths with significant rock cover above the alignments.

In the Board's opinion, the tunnel alignments have been judiciously selected to avoid high ground cover in the less competent sedimentary formations, i.e. McTagg tunnels, or altered zones in the Mitchell-Sulphurets tunnels. Also, geological reconnaissance has been conducted to locate tunnel portals in areas of competent rock, where the risk of slides and avalanches is minimized. Based on the current information, the Board has no reservations regarding the current tunnel locations.

4.2.3 Construction Methods

Given the high ground cover and heterogeneous nature of the rock materials (i.e. altered rock, hard monzonite, and tilted, interbedded sedimentary sequences), the drill and blast excavation method is, in the Board's view, the most prudent option. The Board does not perceive exceptional construction risks for the proposed tunnel excavations. Localized high water inflow can be encountered in the more permeable formations, which could have an impact on the anticipated tunnel advance rates.

4.2.4 Operating Risks

The Board has identified several potential operational risks that deserve further evaluation as indicated below.

The permanent, clean water conveyance tunnels have been designed as twin, mainly unlined tunnel structures, each one capable of accommodating the entire anticipated flow, while maintenance, if required, is carried out in the adjacent tunnel. The twin tunnels are connected at regular intervals to facilitate water transfer, if required, and for worker safety during construction. Note that lining is envisaged in local areas of weak rock formations or where high ARD causes concern. *Although this strategy serves the purpose for the operating mine period, in our view it can be optimized by completely lining some of the*

tunnels, thus improving their long-term performance into closure as discussed in item 4.2.5 below. Redundancy for the pit drainage tunnel is provided by surface collection structures diverting water to the Mitchell-Sulphurets tunnel.

The water collection system at the toe of the retreating glacier at the entrance of the Mitchell-Sulphurets tunnels consists of a series of vertical, unlined shafts, ~ 20 m to 30 m deep, to be excavated in generally competent but heterogeneous rock underlying the glacier. *Although there is precedent for this type of water collection system, the proposed construction method needs to be validated.* In addition, the ongoing retreat of the glacier may require additional surface collection structures and/or lateral tunnels along the glacier flanks to enhance the collection system.

Additional, larger surface structures (i.e. concrete weirs) may also be required to provide adequate surface water collection upstream of the pit for diversion into the Mitchell tunnels.

4.2.5 Effectiveness of Tunnel Operations

The current design contemplates one pair of the non-contact water Mitchell-Sulphurets tunnels to be excavated first to accommodate peak flows with a 200-year return period. An additional pair of larger diameter tunnels are to be excavated later to increase the total capacity to accommodate the 1,000-year flow for the block caving. Redundancy is provided by designing each tunnel to accommodate the anticipated flow while the adjacent tunnel is undergoing maintenance. *In the view of the Board, this design can be optimized by eliminating one of the large diameter tunnels, installing only a single additional tunnel (with or without lining, as appropriate) to accommodate the 1,000-year flood while still providing adequate long-term reliability.* Afterwards, one of the smaller tunnels can be lined to extend its operational life for the reduced long-term water treatment flows.

A similar strategy can be used in the McTagg tunnels, where one of the conveyance tunnels can be lined during mine operations, providing long-term reliability for hydropower production and reducing maintenance.

4.2.6 Safety of the Structures

The design approach contemplates installation of adequate support during tunnel excavation, which can be reinforced in critical areas, for example, through graphite layers in the WSD diversion tunnel, or at tunnel sections under the footprint of the pit.

The portal areas have been located in competent rock and the designers contemplate robust reinforcement of the adjacent rock slopes. In addition, adequate tunnel approach structures are to be installed in the tunnel portals.

The twin tunnel approach provides for adequate resilience, which can be enhanced by installing liners during operations.

Satisfactory behavior of tunnels of a comparable diameter excavated in similar formations in the Granduc mine, and the long-term performance of tunnels at the nearby Brucejack site, demonstrates the reliability of the proposed tunnel design.

4.3 WATER STORAGE FACILITY

4.3.1 General

The purpose of the WSF is to store mine contact water, including flows during a storm or wet period event to permit a controlled release of water via the water treatment plant to Sulphurets Creek.

The WSD is a major component of the WSF. It is an asphalt core rock-fill dam with a height of 165 m, a crest width of 12 m and a length of about 670 m.

The maximum storage capacity is approximately 50 Mm^3 and the normal operating range between reservoir el. 630 m and el. 650 m provides storage of up to 10 Mm^3. The remaining reservoir volume (approximately 40 Mm^3) up to the spillway level (el. 706 m) is used for attenuation of flows during wet years or large short-term runoff events. A freeboard above the spillway invert of 10 m (crest elevation 716 m) allows for potential snow avalanche effects and normal wave run-up requirements. *For the long-term operation, the Board seeks a validation of the 300-year return period avalanche and clarification of the relation to the "Maximum Credible Avalanche", or the 1:10,000- year event to arrive at a comparable level of risk to the risk posed by other hazards such as the seismic and inflow design flood. What would be the consequence of such an event?*

4.3.2 Location and Design of the Water Storage Dam

Considering the topographic constraints and current knowledge of geological conditions, the location of the dam is appropriate although some adjustment may be made after more detailed investigations are completed. Moving the axis about 20 m downstream would limit the overburden excavation, though the effectiveness of the seepage collection dam needs also to be considered.

The asphalt core will need to be constructed on a reinforced concrete slab and this "plinth" must be on competent rock. Preparation of the rock foundation is required to achieve the usual design criteria. The concrete slab is also used as a grouting cap. In this case, considering environmental constraints due to water chemistry (see Section 4.1.6), a high-integrity and deep (150 m) grout curtain has to be constructed. Since this work is intensive and could take considerable time, *the Board recommends incorporating a grouting gallery along the dam axis, excavated into the rock, in order to remove grouting from the construction critical path. If there is no time constraint, then grouting could be done from the plinth surface without the gallery. However, there would still be the issue of how to deal with any remedial grouting that may be required at some time in the future to control the seepage rate. In this latter case, a gallery would be required. The Board recommends a concrete plug be installed in the gorge in the valley floor to form the base of required plinth. This plug could incorporate the gallery which would be extended on the valley sides either as a cut-and-cover gallery or rock adits, allowing the grouting works to proceed independently of core construction.*

The "plug" will minimise the excavation of the abutments necessary to meet the maximum allowable abutment inclination and permit the use of the paving machine more quickly. Note that a minimum 30 m length is required for such equipment. Bituminous protection may be applied to the upstream face of the plug concrete to enhance resistance to attack from low pH water. Thorough grouting of the plug-rock interface needs to be carried out from the gallery after dam construction, with grouting pressures compatible with anticipated water loads to minimize seepage flows.

The basic section of the WSD embankment, as shown on the drawings, is somewhat schematic but the essential elements are present. Regardless of rock source, the materials must be well graded and compacted to minimize deformation, particularly during impoundment. Monzonite from Sulphurets quarry should constitute a good source and is envisaged for the upstream shell. For the downstream shell, NAG, weaker rock such as the Stuhini formation, if well graded, can be acceptable.

If the investigations indicate that the till in the WSD foundation has sufficient strength, stiffness and imperviousness, *the Board recommends leaving it in place under the embankment shells.*

4.3.3 Diversion Gallery

For dimensioning the diversion gallery and cofferdam, *the Board recommends that the flood design return period should be increased from 25-years to 50-years.* Geohazards have been considered in portal locations, particularly the avalanche corridors and rock slides. *Lining of the diversion gallery should be considered for increased hydraulic efficiency and potential adaptation for use beyond the construction stage. Its use for an off-take system or as a low level outlet may be considered.*

4.3.4 Spillway

The spillway, situated on the left bank, is designed for the PMF. Currently the spillway invert is situated on rock and the discharge will flow on excavated rock. For a more reliable performance in the long term (particularly after closure) and even in the short term, *consideration should be given to installing a concrete sill and chute designed to take into account the quality of the rock and the water velocity to avoid damage by cavitation and erosion. Upgrading the spillway for closure must be considered and should provide for dissipating energy and reducing flow velocities.*

4.3.5 Seepage Collection Dam

The SCD is located on Mitchell Creek as far downstream as possible to maximise seepage capture while remaining upstream of a tributary creek. The dam is a small asphalt core rock-fill dam. The foundation conditions are adequate.

4.3.6 Seepage/Hydrodynamic Containment

The flow of groundwater, originating as seepage from the WSD pond, is required to be less than 1 l/s downstream of the SCD. Additional seepage of non-contact water is not part of the control. The allowable solute load passing the SCD is controlled by water quality concentration guidelines for selenium in the Unuk River, in order to ensure preservation of the receiving water quality within this river. Baseline water quality at the US/Canadian border downstream remains unchanged during mine operation. The allowable WSD seepage rate requires high collection efficiency, a factor that has been anticipated in the current design of the WSF and in the control measures that have been proposed.

The location of the SCD must accommodate a number of factors, including foundation conditions, geohazards, surface water drainage patterns, and the anticipated seepage pathways and groundwater discharge in the region downstream of the axis of the WSD. In the PFS, a broad suite of seepage mitigation measures was examined using three-dimensional 3-D groundwater models. *Optimization of the design of the seepage mitigation measures is to be completed as part of the FS level design.*

The Board notes:

- A clear distinction should be maintained between the seepage of pond water out of the WSD, interception of contact water in the drainage galleries below the WSD, emergence of seepage water in the valley floor between the axis of the WSD and the SCD, and seepage intercepted at the SCD. System performance will be judged by seepage that bypasses the SCD and that has a measurable impact on Sulphurets Creek;
- Site investigations indicate complex hydrogeologic conditions in the local vicinity of the WSD. There is evidence of higher permeability zones at both shallow and intermediate depths in the bedrock forming the left abutment. Hydraulic head values vary considerably in nearby piezometers in some locations. *Additional drilling and piezometer installation are recommended in this area, to refine the understanding of the factors that control fluid pressures and groundwater flow in the foundation of the WSD. Dual piezometer installations in single boreholes will allow*

better definition of vertical hydraulic gradients. One or more pumping tests, supplementing single borehole hydraulic testing, should be considered as part of the FS;

- A thick and laterally continuous layer of till is present on the floor and lower side slopes of the Mitchell Valley, upstream of the axis of the WSD. The till extends to el. 650 m, which coincides with the operating level of the pond. The till is expected to serve as a natural liner for the WSD pond, reducing the seepage of acidic contact water into the bedrock flow system. Disturbance of the till layer should be minimized to the extent practical. *It will be necessary to consider the potential impact on containment efficiency as a consequence of higher seepage fluxes from the pond during those periods in time when the pond water level is above el. 650 m;*
- The SCD is located in a narrow canyon, which creates focused seepage pathways in the bedrock and should act to promote high collection efficiencies; and
- At this time the Board sees no need for additional groundwater modeling studies until the data from the recommended field studies for FS are available.

One of the most significant risks for the WSF is the performance of the grout curtain, particularly given the proposed depth of the curtain. In addition, the apertures of the fractures and the rock temperatures influence the ability to grout the rock mass and the curing of the mix to ensure long-term efficiency. The temperature of the mass must, at least, be equal to, or above, 4°C for curing. The temperature of the rock mass at a depth of about 10 m is usually equal to the mean annual temperature that is approximately 0°C at this site. *The Board recommends the installation of thermistor strings to determine the rock temperatures at different depths for planning efficiency of the grout mixes. Consideration should also be given to carrying out in-situ grout testing.*

High rock mass permeability was reported in the left abutment at significant depths. The *Board recommends that additional investigations be carried out, including the installation of multi-level sealed piezometers for a better understanding of the flow regime in this area.*

As mentioned below in Section 4.4.3, alluvium and colluvium with a silt or clay fraction are encountered in the valley floor and on the lower parts of the flanks. These rather impervious materials can be left in place beneath the shells and consideration should be given to placing till immediately upstream of the concrete plinth of the dam to reconstitute the natural blanket.

Results of 3-D seepage analyses illustrate that flow passing beneath the WSD can be captured at the location of the SCD, but the perceived efficiency is a function of the numerical model and the assumed parameters. Reality is a structurally controlled network of discrete features formed by bedding planes and discontinuities, which may not match the flow regime of the porous media in the model. This is judged to present a risk. *To address this risk, consideration should be given to carrying out in-situ pumping tests, which will serve to gain an appreciation of the anisotropy of the system and provide information for refinement of designs for the grouting of the seepage control curtain and other seepage mitigation features such as the seepage interception tunnels*.

All results of the recommended investigations and the above considerations have to be taken into account in the model to obtain a better understanding of the future seepage flow. The depth of the grout curtain at the WSD and the SCDs as well as the location of this latter dam can then be optimized for maximum efficiency.

4.4 ROCK STORAGE FACILITY

4.4.1 Introduction

The Mitchell/McTagg RSF will store waste rock from the Mitchell and Sulphurets pits, while rock from the Kerr pit will be placed in the mined out Sulphurets pit. The rock types are primarily volcanic in origin with some sedimentary rock from the upper part of the pit excavations. However, the degree of weathering of the rock will dictate many of the characteristics to be taken into account wherever selective placement has to be considered.

It has been assumed that all the waste rock is potentially acid generating (PAG) and the facilities are designed to include capture of drainage. Various means to limit the volume of water coming into contact with the rock were mentioned in Section 4.1.6.

4.4.2 Areas Available

The large volume of rock limits the choice of sites and the Mitchell and McTagg valleys present the only feasible option in the proximity of the mine pits. Moreover, the location minimizes the total area affected by the mine operations and the area downstream of the confluence of the two valleys provides a convenient point for capture of drainage waters. As mentioned above, the Sulphurets pit will be used for the waste from the Kerr mine pit, which also limits the overall footprint of the operation and allows for simpler operating design and management of water treatment and control.

4.4.3 Foundation conditions

As far as the foundations of the facility are concerned, the dominant bedrock types are marine sedimentary rocks of the Stuhini Group with minor intrusive rock. Weathering can reach depths of 20 m. However, geotechnical behavior is governed by the significant depths of alluvium and colluvium encountered, respectively, in the valley floor and on the lower parts of the valley flanks. The alluvial materials are generally described as medium dense sandy gravels and cobbles, though a more detailed review of the drill logs shows that a silt or clay fraction is common. Glacio-lacustrine deposits (firm to stiff silty clays and interbedded sands) are encountered in a local area of the Upper Mitchell valley, upstream of the WSF.

While Standard Penetration Test (SPT) values are largely in the 20-40 range some of the tests conducted in the upper 5 to 6 m gave values less than 10*. The potential for contractive behaviour under shear, pore pressure generation and liquefaction potential for any glacio-lacustrine deposits under existing and applied loads, particularly with rapid rock placement, as well as seismic induced deformations, should be included in stability evaluations.*

4.4.4 Design Criteria

Ensuring stability of the dump slopes is paramount and minimum Factors of Safety for the analyses have been establishment commensurate with the risk posed in each location. A value of 1.4 is noted for the slope adjacent to the WSD. *It is recommended that this value be increased to 1.5 for any slide having potential impact on the WSD, either directly or by tsunami effect.*

Other criteria included in design considerations relate to:

- Worker safety;
- Confinement of weak materials;
- Avoidance of incorporation of snow and ice;
- Consideration for geohazards;

- Access;
- Diversion of fresh water around the facility; and
- Selective capture for treatment of selenium-contaminated water.

4.4.5 Stability Analyses

The information provided in the design report indicates that acceptably conservative parameters have been adopted and the results indicate that the minimum factors of safety are met. Note that the toe area of the RSF will be flooded by the impoundment behind the WSD.

4.4.6 Site Preparation

Construction of the water storage dam diversion tunnel, the seepage galleries associated with the WSD, the McTagg Inlet and diversion tunnels portal along with the installation of temporary water treatment plants with associated settling ponds and camp support infrastructure will precede the development of the RSF's.

Foundation preparation would include removal of organics and other deleterious materials where these could contribute to instability. Shallow deposits of loose sands may be removed *or consideration can be given to in-situ ground improvement techniques if the deposits are of significant depth.*

4.4.7 Early Placement

It is envisaged that a layer of coarse, sound, free draining rock of up to 30 m depth, will be placed in a base zone on the valley bottom to facilitate under-drainage and ensure rapid equalization of pressures caused by fluctuating levels in the reservoir of the WSD.

4.4.8 Features Built into the RSF

As mentioned above, the selective capture of water containing selenium is to be included to the extent practical, to improve efficiency in the control of this contaminant. A layer or layers of low permeability material is shown schematically on the cross-section of the downstream face to direct water to a collection point, however, the conceptual design still remains to be carried out and the Board anticipates being briefed on this feature at forthcoming meetings.

The outer slope of the Kerr waste backfilled into Sulphurets pit incorporates geomembrane elements that act as an "umbrella" or "shingles" feature designed to shed precipitation. Run-off from this system together with dump under-drainage would be routed directly by gravity to the Selenium Treatment Plant and subsequently released into the WSF for treatment at the WTP. There is at least one case of the successful use of such a concept on the downstream slope of a hydro-electric dam built with sulphide-rich rock. *However, the degree of compaction used for the latter may be greater than a rock dump and the eventual design needs to account for anticipated settlement.*

As mentioned in Section 4.1.6, non-contact water from the upstream part of the McTagg valley will be diverted through a tunnel system. Run-off from the catchment downstream of the intakes will be routed across the lower benches of the RSF in lined ditches. For the most part, these are located along the contours of the valley walls and, as such, will be constructed on shallow depths of fill. The overall schedule of construction of the RSF will allow for much of the settlement of the rock-fill to take place prior to shaping of the ditches and the placing of geomembrane. The drawings (not-for-construction) indicate the use of bituminous membrane, which has limited extensibility and relies for integrity on a well prepared base. Ensuring a uniform gradient and the avoidance of "soft spots" will need to be part of future designs. Furthermore, these ditches may be impacted by avalanches and landslides and therefore ease of maintenance should be an additional criterion. Members of the Board have experience with bituminous

membranes at a mine where removal of ice-bound rock above the membrane in winter was required. The ice appeared to adhere to the bituminous membrane and the liner had limited resistance to mechanical abrasion, plus there was considerable membrane damage during ice-rich rock removal. Experience with mechanical removal of ice from the surface of High Density Polyethylene (HDPE) liners appears to be better.

Locations for the placement of highly weathered materials and lined cells for sludge from water treatment will also be incorporated into the body of the dump. *For lined cells the potential for differential settlement and cracking should be recognized.*

4.4.9 Routine Placement

In view of the requirements for stability, and the incorporation of drainage and other features, waste rock placement will be by a bottom-up procedure rather than high lift end dumping. Each lift may still have an outer slope at the angle of repose, but the overall slope will be flatter and include berms to arrest falling rock which could otherwise compromise safety. Placing in lifts will also permit greater selective placing in critical areas such as adjacent to the WSD and the Ore Preparation Complex (OPC).

4.4.10 Monitoring

Piezometers in the foundation, extensometers and surface monuments will be incorporated to complement visual observation of the slope stability and overall behaviour. No details have been presented at this stage.

4.4.11 Closure Additions

The aforementioned surface drainage system will be completed at closure. *The Board also recommends that consideration be given to providing diversion ditches with erosion resistant overflow segments that would discharge flood flows larger than the design flood peak without significant damage to the ditches.*

4.5 WATER TREATMENT

Treatment and discharge of the impacted water collected from the various open pit mines and the RSF is proposed to remove excess water from the Mine Site. This is an acceptable approach at water surplus sites such as KSM. The proposed treatment technologies, High Density Sludge (HDS) for metal containing ARD, and ion exchange for selenium removal are commonly used and acceptable technologies.

Runoff and seepage that is impacted with mining related constituents (i.e. “contact water”) due to sulphide oxidation and leaching of constituents, will be generated in the RSF, the walls of the open pits and underground mines, and potentially other locations where bedrock is exposed or excavated for the construction of other mine facilities such as the ore preparation complex, water treatment plant and energy recovery power plant, access and haul roads and tunnel portals. Runoff and seepage from the RSF is collected behind the WSD where it is temporarily stored before treatment and discharge. Ditches and pipelines are proposed to convey water from the three open pits (Kerr, Sulphurets and Mitchell) and other sources of impacted water such as the ore and waste rock lay down areas, to the WSF. Impacted water from natural sources of mineralized rock located on the valley sides and in the Snowfields Slide area below the Mitchell Glacier will also be collected.

As discussed in Section 4.1.6, natural runoff, referred to as “non-contact water” is diverted away from the WSF into natural drainages using ditches and tunnels in order to minimize the volume of water requiring treatment.

The HDS Water Treatment Plant is located along the banks of Sulphurets Creek, which is topographically lower than the various contact water sources and downstream from the WSF. The selenium treatment

plant is located on the left abutment of the WSD, which is topographically below the RSF. Water treated in this plant will be discharged to the impoundment behind the WSD where it can be pumped to the HDS for treatment of other metals that may be present. Other package selenium treatment plants could potentially be strategically located near selenium sources such as the Snowfields Slide and the Kerr and Sulphurets pits. Consideration will be given to interconnecting some of the remote selenium treatment plants to the WDF, or directly to the HDS Water Treatment Plant in the event further metals removal is required for these water streams. The plants generate wastes, particularly the HDS plant which generates sludge. This sludge will be temporarily stored near the plant, filtered and then added to the ore that is conveyed to the process plant for disposal in the TMF.

The Board concurs with these locations and this general approach to water treatment.

The average amount of treatment required is determined by the excess contact water that is generated, which has been estimated at approximately 2.3 m^3/sec. The capacity of the HDS Water Treatment Plant is based on treating sufficient water that can be discharged to match the current downstream flow quantities, which is a requirement established as part of the EA process. As a result, the treatment plant capacity has provisionally been established at 7.5 m^3/sec, close to three times the annual average flow rate.

The Project Team described a focussed selenium collection and treatment system that is intended to selectively capture the large selenium sources and treat these in separate, specifically designed treatment plants. Since the selenium in surface waters of the area occurs predominantly in the form of oxidized selenate, which is not removed by the HDS Water Treatment Plant, ion exchange technology is needed to remove it. In order to limit the size of such a plant or plants, a focussed collection system has been evaluated and approved as part of the EA process. It provides for capturing runoff from the RSF and temporally storing it in lined sediment ponds constructed within the RSF and also creating a low permeability layer at year five on the surface of the RSF to facilitate collection of infiltration through the RSF before it migrates into the under drainage system and mixes with the water collected behind the WSD. Opportunities for selective capturing and treating selenium from other sources such as the Snowfield area are also being considered.

The Board recommends that the plans presented in the EA Documents be further developed for the updated PFS to include more details on the systems that will be used to collect the higher selenium water, how these will be transported to an appropriately located selenium treatment plant or plants and managed.

The Project Team have recognized risk associated with operating several treatment plants, and treatment plants in general. These include failure to meet discharge standards, mechanical breakdowns or shut-downs because of the lack of access for operating personnel, or reagents which need to be delivered to remote locations, and failure of the conveyance pipelines or ditches due to the presence of geologic hazards. Other risks include those associated with spillages of reagents, fuels and lubricants. Most of these risks are understood and can be managed by developing appropriate operations and contingency plans and by including adequate storage and the appropriate redundancy in the design of the plants, such as excess pumping capacity, additional treatment trains, amongst others, and containment features around reagent storage tanks and containers. A primary risk mitigation factor is the size and robustness of the WSF. There will be residual unknown risks associated with geologic and snow hazards that need to be addressed by contingency measures.

Treatment plants, while automated, require attention and maintenance and need to be accessed by operating staff and equipment used for repairs. Personnel access, particularly in the winter months, poses safety risks, which can be managed by providing safe access ways and roads and health and safety work procedures at the mine.

4.6 CLOSURE

4.6.1 Surface Water Diversions and Seepage Control

On closure, the bottom of Mitchell pit will fill as a lake. Drainage from this lake and the reclaimed RSF will be directed towards the WSF for storage and treatment. Non-contact water ditches will be extended around all four sides of the RSF to maximize diversion of natural runoff. The Mitchell Underground Drainage tunnels will be plugged, while the Mitchell Diversion and McTagg tunnels will continue to be used for hydropower generation. It is considered advantageous to maintain the Mitchell Diversion tunnel operation at closure to allow generation of hydropower and reduce water treatment requirements.

The Board has reviewed the layouts developed and concurs that the location of the various drainage ditches and tunnels and the WSD and its associated seepage collection pond are appropriate. The Board recognizes that as further design details are developed there will likely be some changes in locations and sizes of the facilities to better suit actual field conditions.

Both the HDS and the selenium water treatment plants will have to be operated in the long term after closure since ARD will continue to emanate from the RSF. Runoff and seepage from the RSF area will be collected and treated in the Selenium Treatment Plant prior to discharge to the WSF. Water from the WSD is then pumped to the HDS Water Treatment Plant prior to discharge. Water from the Kerr pit would be conveyed by a pipeline (which crosses Sulphurets Creek) to the treatment plants. These water conveyances will require continued maintenance, as well as routine replacement of the equipment as it wears out. The effective life of these types of mechanical systems is typically 20 to 30 years. There will be opportunities in the future for improved treatment technologies to be installed at the Site. It is intended that the treatment sludge generated by these facilities will be transported to the upper surface of the RSF for disposal.

The Board recommends that consideration be given to providing for gravity drainage of water collected behind the WSD and the SCD to the water treatment plant, to minimize or reduce the need for long-term operation of pumping systems. Consideration should also be given to filling the reservoir behind the SCD with coarse rock covered by low permeability material and converting it to a seepage collection sump, which sheds runoff water and reduces the amount of seepage requiring capture.

The Board furthermore recommends that prior to closure, disposal of the water treatment plant sludge in the Mitchell pit lake be evaluated as it potentially provides for a stable chemical environment that is closer than the TMF.

4.6.2 Long-term Stability

The seepage control systems in the Mitchell and Sulphurets valleys should be expected to be in operation for at least several hundred years and likely longer. Monitoring systems to ensure groundwater quality is within limits for the expected seepage rates must be maintained and replaced when necessary.

4.7 BOARD RESPONSE TO QUESTIONS

4.7.1 Are the dams and major structures appropriately located?

<u>Tunnels</u>

The Board considers that the tunnel alignments have been judiciously chosen and has no suggestions for adjustments or modifications.

Water Storage Facility

The Board considers that the WSD and SCD are positioned in an appropriate location downstream of the RSF, given the topography and the current understanding of seepage pathways derived from the 3-D groundwater models of the mine site area. The WSD is located as close to the RSF as feasible at a suitable location for dam construction on Mitchell Creek. The SCD is located a relatively short distance (~ 40 m) beyond the toe of the WSD. While local adjustments in location could be envisioned based on results of future site characterization activities and final selection of seepage mitigation measures, the Board does not anticipate there would be a requirement for a material change in location of the WSD or the SCD.

Rock Storage Facility

The facility is appropriately located in valleys near to the mine pits, which permit confinement and collection of run-off.

The surface drainage ditches basically follow the valley side-wall contours and thus ensure uniform settlement. The sections of the ditches that cross the RSF will be constructed on rock-fill placed several years previously and therefore settlement will be limited.

Taking into account the above considerations the Board considers the facilities can be developed for the intended use at the locations selected.

4.7.2 Are dam sections, materials and construction methods and sequencing appropriate for the site and purpose?

Tunnels

In the Board's view, the proposed drill and blast excavation method is a prudent and flexible approach to accommodate the heterogeneous ground conditions anticipated along the alignments.

Water Storage Facility

Asphalt concrete is a flexible and ductile material with viscoelastic-plastic properties, which provide a self-healing and self-sealing ability. Located in the centre of the dam, the asphalt core is embedded and protected from extreme weather conditions. It is resistant to ageing and is a so-called "forgiving" material. Asphalt core dams have an excellent demonstrated field performance with no recorded leakage through the core or the core-plinth interface at the base of the core. The materials comprising the core, especially the type of bitumen, are chosen according to anticipated placing and operating conditions including the water chemistry.

A reasonable set of seepage mitigation measures has been considered for both the WSD and the SCD (grout curtains and the use of seepage collection galleries to create a hydraulic sink). 3-D groundwater models have been used to quantify the performance of these containment measures to a level appropriate for a PFS. *The Board suggests consideration of using pumping wells to either augment seepage interception in the foundation of the WSD, or as an alternative to extensive use of seepage collection galleries*. *This could be designed as a contingency measure to be applied only if proposed methods prove to be insufficient.*

With limited opportunity to locate the SCD farther downstream, seepage interception beneath the downstream footprint of the WSD is a key design element, as is an effective grout curtain beneath the SCD. *There is a trade-off that needs to be evaluated in the FS between the depth of the grout curtain below the WSD, the extent and configuration of seepage interception galleries below the downstream embankment and in the abutments of the WSD, and the final design location of the SCD within the*

canyon. A deep curtain (say >100 m) below the WSD may not be the optimal configuration for seepage interception given the footprint space available within the canyon.

The downstream shell of the WSD should be constructed using non-acid generating (NAG) rock to minimize incremental mass loading of solute to groundwater passing beneath the foundation of the WSD. This may also be a requirement for the long-term physical integrity of this rock-fill. Leaching characteristics of the rock forming the downstream shell under neutral pH conditions also requires close attention.

Only one borehole is located at the proposed site of the SCD. *Additional site investigation at the proposed location of the SCD is required during the FS.*

Rock Storage Facility

The Board concurs with the design and construction methods. Construction materials are to be derived from the mine pits. Some selection is to be used to place weaker materials within the body of the dump. The actual sequence will be dictated by mining.

The Board concludes that the dam sections, materials and construction methods and sequencing are substantially appropriate for the referenced facilities.

4.7.3 What, in the opinion of the Board, are the greatest design, construction and operating risks?

Tunnels

The current design provides adequate redundancy to accommodate the design flows. However, in the Board's view this redundancy can be optimized and enhanced by lining installation as suggested in Section 4.7.4 below. The Board perceives some risk in the construction and performance of the proposed shaft water collection system at the toe of the Mitchell glacier, *and suggests conducting a more comprehensive evaluation, including site visits to existing prototypes and development of potential mitigation options.*

Water Storage Facility

With further site characterization during the FS, and the implementation of an optimized set of seepage mitigation measures, it is reasonable to anticipate that seepage rates less than 1 l/s containing a component of mine contact water can be achieved for the WSF. The Board notes, however, that because of the low value of allowable seepage and footprint limits, this is a significant risk to water quality. Seepage interception in fractured bedrock is challenging and requirements may differ substantially from those developed based on porous media approximations. While groundwater models provide an appropriate basis for examination of design concepts supporting a high level of seepage interception, confirmation of the interception efficiency is realistically only possible following start-up of operations. *The Board advises that implementation of the suite of seepage mitigation measures follow an observational approach (adaptive management). Furthermore, a set of groundwater wells in the canyon, both between the centreline of the WSD and the SCD, and downstream of the SCD, will be required to monitor water levels and water quality and permit timely evaluation of the system's performance. It will be appropriate to install these monitoring wells at the construction phase of the Project. In addition, contingency plans should be developed to provide for seepage collection in the event water quality impacts were detected.*

It is clear that SG/P has recognized the geochemical risk associated with backfilling the Sulphurets pit with waste rock from the Kerr open pit. The Board anticipates a high level of control will be required to reduce the rate of infiltration into the waste rock and measures must be in place to collect contact waters

reaching the base of the pit. *Additional subsurface investigation will be required at FS to support the assumption that, with the proposed protection measures in place, any residual seepage of contact water entering the bedrock flow system below the pit will not compromise water quality in Sulphurets Creek.*

Rock Storage Facility

Geohazards have been identified and must be fully taken into account during construction.

Maintenance of drainage ditches in the long term is seen as a challenge and all means necessary to facilitate maintenance and to recover operation after any avalanche, landslide, or flood event have to be considered at the outset.

4.7.4 Are the facilities designed to operate effectively (and efficiently)?

Tunnels

In the Board's view the tunnels are design to operate effectively. The efficiency of the current non-contact water conveyance tunnel design can be optimized by:

- Eliminating one of the large diameter Mitchell-Sulphurets tunnels, and installing only one lined tunnel to accommodate the 1,000-year return period flow. This option would also enhance the reliability of the tunnel for long-term performance;
- Installing a liner in one of the McTagg tunnels during mine operations, to enhance long-term performance for hydropower development; and
- Installing a liner in one of the smaller Mitchell-Sulphurets tunnels to minimize long-term maintenance.

Water Storage Facility

The WSF is designed to operate effectively. Careful monitoring and contingency seepage collection systems may be required to ensure this system of dams meets the design seepage objective of 1 l/s. Long-term operations may be made more efficient by converting the WSD's construction diversion tunnel to a gravity outlet to the Water Treatment Plant instead of pumping water over the dam to the Water Treatment Plant.

Rock Storage Facility

The RSF is designed to operate effectively, however, the effectiveness and efficiency of the selenium capture system remains to be demonstrated.

Overall, the facilities referenced are designed to operate effectively.

4.7.5 Are the facilities designed to be safe?

Tunnels

In the Board's view the current conceptual designs can be safely constructed and operated provided that additional, required information is collected during the following design stages.

Water Storage Facility

In the Board's view the current designs can be safely constructed and operated.

Rock Storage Facility

Essentially the answer is yes, if foundation conditions are assessed in detail prior to construction, and adequate Factors of Safety can be demonstrated.

The Board concludes that designs for the operational period of the mine are to standards generally accepted in the industry. Site terrain, weather, geohazards and requirement for long-term maintenance require further design and attention to operational and construction methods.

4.8 RECOMMENDATIONS FOR FOLLOW-UP DESIGNS

The anticipated next steps in developing the designs include updating the current PFS and then proceeding with a FS design, followed by a Detailed (for construction) Design. The Board's recommendations for the PFS and FS phases are summarized below.

4.8.1 Pre-Feasibility Study

The recommendations for the RSF/WSD system include describing concepts for capturing the higher selenium concentration seeps within the RSF during operations and closure.

4.8.2 Feasibility Study

Additional field investigation is recommended to refine the understanding of hydrogeologic conditions (structural elements, permeability distribution, and hydraulic anisotropy) in the foundations of the WSD and the SCD. These studies are required to reduce uncertainties in the definition of potential seepage pathways and to refine estimates of the potential magnitude of the fluid fluxes.

The extent of both grouting beneath the SCD and the scale of the drainage works will need to be confirmed during FS. At that time, grouting trials will be required. If it were to be the case that seepage fluxes with a component of contact water exceeded the 1 l/s criterion, it is likely that contingency interception wells would need to be operated at locations beyond the SCD to avoid the narrow canyon location. *It will be important to demonstrate the practicality of operating effective pump-back wells located on flat ground above the canyon walls beyond the SCD, in the event they are required to achieve the required degree of seepage interception.*

5. Tailings Storage Facility

5.1 SITE AND DEVELOPMENT DESCRIPTION

5.1.1 Site and Facility Development

The proposed TMF will be located southeast of the Treaty Process Plant, within the upper reaches of two creeks – South Teigen Creek, which is a north-flowing tributary of Teigen Creek, and a south flowing tributary of Treaty Creek as shown on the Figures 1.1-2 and 5.1-1. Both Teigen and Treaty creeks are tributaries of the Bell-Irving River.

The TMF site is favourably located at a saddle between South Teigen and North Treaty creeks and has a total catchment area of 55 square kilometers. This includes a valley on the east side of the TMF (East Valley) that has a catchment area of 15 square kilometers. Natural runoff water from this area will need to be managed with diversions during the site development, operations and closure/post closure stages of the Project. The natural topography in the vicinity of the TMF site ranges from about 800 m to over 1,800 m at the top of the abutment ridges. Steep valley side slopes, especially on the east side of the TMF, will make construction difficult and will present challenges for reliable operation of diversions.

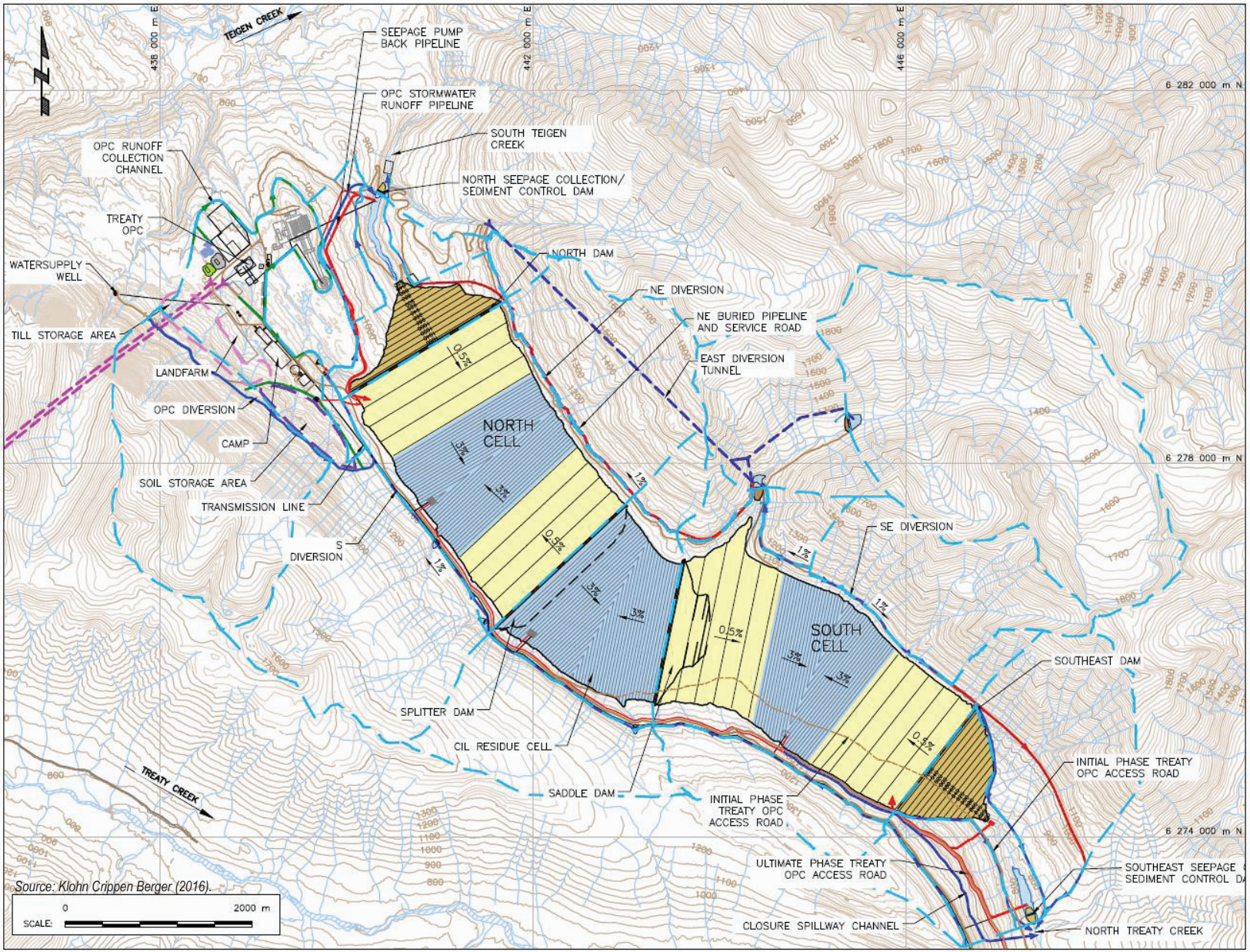
High rainfall, requirements for water discharge, high snowfall and the need for seasonal construction are characteristics that will present challenges to the development of TMF.

The TMF has been sized to store 2.2 Bt of tailings. Tailings will be delivered to the TMF by HDPE pipelines originating from the nearby process plant. Separate pipelines will be provided for transport of floatation (which are NAG) and CIL tailings (which are PAG). Barge mounted pumps will be utilized to remove water from reclaim water ponds on the active impoundments. The pipeline corridor has been appropriately located on the west side of the valley where side slopes are relatively favourable and avalanche risk is reduced compared to the east side of the valley.

The TMF is divided into three cells in order to manage flows from the East Valley Catchment, to separate the CIL residue from the flotation tailings, to minimize surplus water in the TMF, and to allow progressive reclamation of the TMF. The North Cell and South Cell will store rougher floatation tailings, and the CIL Residue Cell will store the treated (for cyanide removal) sulphide-rich cleaner tailings (CIL residue). The CIL residue will be in contact with cyanide during processing, and the CIL Residue Cell will be fully lined with geomembrane and located between the North and South cells at the time of mine closure. The rougher floatation tailings and the CIL tailings will flow separately by gravity or will be pumped to the TMF. The CIL tailings are estimated to represent 10% of the total tailings with the remaining rougher floatation tailings representing about 90% by weight.

During operations, surplus water from the CIL pond will be reclaimed with a barge mounted pump and routed through the Treaty Process Plant for treatment and eventual discharge to the North and South flotation cell ponds. Management of surplus water from the North and South cells during the operating phases will use a combination of storage (during low flow months from approximately November through March) and pumped discharge to a diffuser situated in Treaty Creek (during higher flow months from approximately May through October).

The design concept involves utilizing the TMF impoundments as the source of makeup water. Diversion systems are included to reduce run-on to the TMF, but on average, the water balance indicates the pond on the impoundments will have a maximum volume of approximately 9 million (M) m^3.



Source: Klohn Crippen Berger (2016).

Figure 5.1-1

TMF Layout

Figure 5.1-1

The TMF will be developed in stages. The North Cell with be constructed during Stage 1 and will store floatation tailing production for approximately 25 years and will be closed and reclaimed thereafter over a five-year period. The CIL Residue Cell will be constructed and operated in parallel with the North Cell and will be only partially filled when the South Cell is constructed and put into operation, providing floatation tailing storage for the remaining mine life. At mine closure, the Centre and South cells will be closed and reclaimed over a five-year period.

5.1.2 Geologic Conditions

The bedrock underlying the proposed TMF consists of folded, layered, sedimentary and metasedimentary rocks of the Bowser Lake Group. The bedrock comprises hard metamorphic sandstones, siltstones, mudstones and occasional conglomerates. Steeply dipping beds and occasional minor locally contorted folded sections are present. Discontinuities consist of regional jointing and strike slip faults cutting across and offsetting bedding planes with predominant strike direction of southwest-northeast. Bedrock crops out along valley slopes at higher elevations.

Surficial soils cover bedrock over most of the TMF site. Five major surficial overburden materials have been identified including glacial basal till, glacial moraine till, colluvium, fluvial and debris fan deposits, and alluvial deposits. The glacial basal till forms a blanket of dense consolidated material up to 20 to 60 m thick in the bottom of the bedrock valley below elevation 1,000 m. The glacial moraine till forms lateral moraines of silty sand with some gravel, cobbles, and occasional of boulders and is up to 10 m thick covering the lower valley walls. The colluvium consists of boulders and gravel with some silt and sand derived from erosion of moraine soils and weathered bedrock; it occurs on the upper slopes of the east valley wall above elevation 1,050 m and at the base of steeper slopes as fan deposits. Fluvial and debris fan deposits are found in pockets at the foot of the valley slopes and comprise colluvium and talus debris eroded from the upper valley walls in small streams. The recent alluvial deposits are located on the valley floor within the centre of the TMF and have been found to be up to 22 m thick. Poorly drained areas are covered by wet organic material in various stages of decomposition.

The TMF is in an area of groundwater discharge with groundwater flow controlled by the topographic relief and bedrock discontinuities (faults and fractures). Groundwater levels have been recorded in boreholes and groundwater wells drilled at the site and indicate an upward hydraulic gradient.

Overall, geologic conditions at the site are favourable for development of the TMF. However, shallow alluvium in some areas of the central TMF area has been characterized as liquefiable under earthquake loading and the permeability of the surficial soils is not sufficiently low to serve as a barrier to seepage flows beneath the TMF structures without engineered seepage cut-offs.

5.1.3 Geohazards

The steep high side slopes of the TMF valley are prone to a number of geohazards. These include snow avalanches, particularly in zones identified as avalanche tracks, and slope instability, particularly where water diversion ditches are installed to divert surface runoff with associated access roads and pipeline benches. High precipitation events may be expected to generate debris flows under high precipitation or run-off conditions. Each of these hazards could cause blockage of diversion ditches and endanger personnel on access roads or performing maintenance, including emergency maintenance. Overtopping of diversion ditches can result in washouts of access roads and pipeline benches and severe downslope erosion, including cutting off access, rupturing pipelines and causing damage to downslope liner systems. The Project Team have identified these hazards and recognized the need to take these into consideration in selecting slopes on which to install access road and pipeline benches.

Cold winter temperatures cause freezing of slow flowing water in ditches. Warm groundwater inflows and snowmelt during marginal snowmelt periods are expected to result in ice accumulation in diversion ditches. Ice occupies flow volume, which must be maintained for potential high runoff periods or ditch overtopping could potentially occur. Diversion ditch access has been designed for and ditch maintenance will be included in the operating procedures.

Diversion ditches are required to minimize surface water inflow to the TMF and hence the requirements for water treatment. There is limited flexibility as to where these diversions can be located. The nature of the cuts to develop the slope section that contains the required diversion ditch, as well as access and utility road widths is highly dependent on the detailed slope geometry – both the slope angle, as well as the slope irregularity formed by erosion or drainage gullies and rock outcrops through which the diversion channels have to be installed. Generalized slope sections seldom provide a true reflection of the variability of section that will be required. The realizable slope geometry is further complicated by the nature of the slope materials in which the cuts have to be made. The colluvium, talus and weathered rock conditions in which cuts have to be made have a large influence on upslope cut and downslope fill heights. *During the FS design it will be necessary to develop more detailed cross sections at relatively close spacing using accurate topography to develop accurate quantities of earthworks and assess local construction access and construction methods.*

The east slope of the North and Central cells is particularly steep and the cost of construction of diversion will be high as will the risk of failure. The diversion letdown channel on the right abutment (Northeast side) of the North Dam utilizes a natural channel which has been evaluated for capacity adequacy. A pipeline is initially installed to maintain riparian low flow sourced from the East Catchment to Teigen Creek. Provision is made for a short tunnel to bypass flow around this failure zone, should it mobilize. The diversion of drainage from this catchment, via a tunnel to the north, has been provided.

5.1.4 Hydrology

The northern part of the TMF site is drained to the north by a tributary to Teigen Creek, which in turn drains into the Bell-Irving River (Figure 5.1-1). The southern part of the TMF site is drained to the south into Treaty Creek and from there also into the Bell-Irving River. Surface elevation at the Site ranges from 800 m to approximately 1,800 m. The East Creek tributary drains from the north-east into the central portion of the TMF site. Glaciers occur in the upper headwaters of East Creek.

The estimated average annual precipitation in the TMF catchment is approximately 1,540 mm and generally increases with elevation. The mean annual runoff is estimated to amount to approximately 90% of the precipitation; i.e. approximately 1,400 mm. Peak flows occur in June associated with snowmelt and rainfall (28% of the annual amount) and to a lesser extent in September due to rainfall events (12 %). The temperature ranges from -28°C to 30°C, and freezing conditions typically occur from mid-October to mid-April. Annual evaporation is estimated at 350 mm, which is substantially lower than the precipitation.

Clean water diversion ditches and tunnels are designed for the peak 200-year, 24-hour storm event to maximize the volume of water diverted. It is furthermore assumed that the diversion efficiencies are less than 100%, which is appropriate. *The Design Team uses diversion efficiencies of 60 to 80% and the Board recommends that these estimates of efficiency be refined by conducting ditch seepage analyses for the FS design.*

During operations, a dry storage capacity for the 30-day PMF rainfall on top of a 100-year snowpack run-off is to be maintained to provide for tailings dam safety (and downstream water quality protection) assuming the diversion ditches are inoperative. *The Board understands that PMF protection is required by Canadian Dam Association (CDA) standards since the dam's hazard rating is "Extreme", and suggests that consideration be given to further refining this "mixed" criterion reflecting PMF conditions and that a*

critical storage duration (which may be longer or shorter that 30 days) be established. On closure each of the two TMF spillway exits will be sized for the PMF entering the TMF, so there will be a 100% redundancy, which is appropriate since there is a risk of the sides of either spillway failing.

A more detailed assessment is recommended by the Board for the PFS considering precipitation data for the maximum period of record of the meteorological stations as it could show large variations in pond volumes due to large storms such as the PMF and extended wet and dry periods. The potential benefits of an external water storage reservoir to aid in managing water quantity and water quality should also be considered.

During operations, the seepage collection dams will be sized and operated to store the 200-year, 24-hour storm event without discharge. The spillways will be sized for the 500-year, 24-hour storm events.

5.1.5 Hydrogeology

The TMF is to be located in a broad, U-shaped valley with ridges to the west and east that are at substantially higher elevation than the ultimate elevation of the tailings facility. The valley floor and lower valley flanks are covered by glacial tills, moraines, colluvium and fluvial sediments, which have been mapped to a level commensurate with PFS investigations. A sufficient data set for hydraulic conductivity of the unconsolidated sediments was obtained for the PFS. Results indicate the colluvium deposits have a geometric mean hydraulic conductivity about an order of magnitude higher (i.e. 4×10^{-6} m/s) than the tills and alluvial sediments. The calibrated groundwater model yields generally similar values to the field measurements, but with the highest hydraulic conductivity assigned to the lateral moraines (1×10^{-5} m/s). Springs emerge from the colluvium as it transitions to the till and alluvial valley fills. This upwelling has to be addressed in designs of drainage blankets and liners.

A natural surface water and groundwater divide occurs on the valley floor at the location of the proposed Saddle Dam; with flow to the north contributing to the Teigen Creek drainage and flow to the south contributing to the Treaty Creek drainage.

Artesian groundwater pressures have been measured in the footprint area of the TMF beneath the main valley floor; a common condition in this type of topographic setting. Current groundwater flow rates out of the valley to the north and south are interpreted to be relatively low based on the calibrated 3-D hydrogeologic model (i.e. 5 l/s and 4 l/s, respectively).

There is also a component of subsurface flow in East Valley that enters the main valley southeast of the drainage divide. If significant in quantity, there may be an advantage in intercepting this clean water flow.

The valley geometry creates favourable conditions for lateral hydrodynamic containment of water in the TMF. The water table in the valley flanks will move upward as the TMF is raised. Groundwater discharge on the lower flanks of the valley and the valley bottom will require that measures be taken to control uplift pressures for liner installation at the CIL Residue Cell. During Stage 1 of TMF operation, the highest hydraulic heads in the foundation of the TMF will occur beneath the North Cell. In Stage 2, with construction of the South Cell, the highest hydraulic heads in the foundation are expected to occur beneath the CIL Residue Cell, as it is located at the mid-section of the TMF. The current plan envisions pressure control using temporary dewatering wells. *In the opinion of the Board, this strategy needs additional development and it recommends consideration of an underdrain system beneath the liner to control uplift pressures on the proposed geosynthetic liner. It will be necessary to estimate the volumetric flow and water quality in the discharge from an underdrain system, to determine how best to manage this water.*

Interception of process water seepage is required at the North, Saddle and South Seepage Collection Dams (NSCD, SSD and SSCD). During Stage 1, the seepage management system at the north end of the valley will intercept water originating from the North Cell, construction water, and groundwater

originating as recharge on the flanks of the valley. The SSCD at the south end of the valley will collect water infiltrating from the North Cell, any leakage of CIL process water escaping through defects in the geomembrane liner, construction water from the Splitter Dam and the Saddle Dam, and groundwater originating as recharge on the flanks of the valley, including a contribution from East Valley. *Water quality and the required efficiency in seepage collection at the SSCD may require additional evaluation during the FS.* During Stage 2, when the South Cell is constructed, several large talus slopes on the east side of the valley could create preferential zones of higher permeability, forming a "permeable surround" for contact water in the South Cell pool to migrate toward the South-east Dam and the SSCD. *The potential impact on seepage rates of a permeable surround zone requires further evaluation at the FS stage.*

The groundwater model indicates that while the principal seepage pathway from the CIL Residue Cell will be to the south, there is a smaller component of CIL seepage beneath the North Cell. *These potential pathways require careful examination in the FS.*

5.1.6 Tailings Characterization

5.1.6.1 Tailings Production Schedule

Primary ore crushing will take place near the ore bodies. The ground ore will be transported via conveyor belts in the Ore Haulage Tunnel to the Process Plant located near the TMF for processing. Exploiting several different ore bodies using open pit and block cave mining methods will cause variations in yearly average plant throughputs over the mine life with an annual average rate of 130,000 tpd.

5.1.6.2 Tailing Properties

Laboratory testing was performed on samples of floatation and CIL tailings in 2009, 2010, 2011, 2013 and 2015. Based on this testing and experience on other mining projects, KCB has characterized the tailing properties for use in their engineering analyses and PFS design.

Specific Gravity

The average specific gravity, Gs, of the Flotation and CIL tailings was measured to be 2.75 and 3.45 respectively. The higher Gs of the CIL tailings reflect the concentration of sulphides, primarily pyrite, in this tailing stream.

Grain-size Distribution

Due to differences in the ore properties, it has been assumed that the floatation tailings will have two different target gradations with P_{80} values of 150 microns for the first 30 years of mining and 120 microns for the remaining mine life. Whole tailing gradations presented to the Board had 100% passing the No. 60 sieve and 55% passing the No. 200 sieve. The underflow gradation for the cyclone sand has been assumed by KCB to have 17% passing the No. 200 sieve.

The target gradation for the CIL tailings is a P_{80} value of 15 microns. The whole tailing CIL gradation presented to the Board had 100% passing the No. 100 sieve and 98% passing the No. 200 sieve.

Atterberg Limits

Atterberg limits testing was performed on one sample of the floatation tailings and on two samples of the CIL residue in 2010. The test results, presented in the table below, indicate that the floatation tailings sample classified as low plasticity clay (CL-ML). One of the CIL residue samples classified as silt (ML) and the second as clay (CL).

Table 5.1-1. Atterberg Limits

Test Campaign	Tailing Type	Sample Name	Data Source	W_L (%)	W_P (%)	PI (%)	% Fines	Soil Type
2010	Flotation	Low Pyrite Rougher Tailings	KCB 2010	21	17	4	57.9	CL-ML
2010	CIL	P4 Cleaner Tailings	KCB 2010	31	16	15	99.7	CI
2010	CIL	P4 Pyrite Tailings	KCB 2010	30	23	7	98.8	ML

Dry Density

The dry densities of three tailing materials are of primary importance to the TMF design: 1) the compacted cyclone underflow sand material; 2) the impounded floatation tailing material; and 3) the CIL residue deposit.

The PFS proposes that the underflow sand used for dam construction be compacted to 95% of Standard Proctor maximum dry density, which was measured in the laboratory to be 1.58 tonnes per cubic meter (t/m^3). A compacted minimum dry density of 1.51 t/m^3 representing 95% of Standard Proctor maximum dry density has been adopted for design. Relative density testing on the cyclone sand was also performed and indicated a minimum dry density of 1.28 t/m^3 and a maximum dry density of 1.55 t/m^3. *In the experience of the Board, in-place, dry densities in compacted cyclone sand embankments are typically in the range of 1.60 to 1.65 t/m^3 and it therefore recommends that a value in this range be considered and further investigated in a parametric study for the PFS.*

Laboratory tests were conducted on the whole tailing materials and the CIL residue to evaluate their densities in the tailing impoundments.

Jar settling tests performed on samples of whole floatation tailings at a range of initial solids densities showed settled densities of 0.93 to 1.13 t/m^3, while tests on samples representing cyclone overflow fines showed settled densities of 0.78 to 1.04 t/m^3. Since cycloning is only planned for the warmer months of the year, the impounded tailing materials will be a mixture of whole tailings and cyclone overflow fines and the settled density will be somewhere between these ranges. Large strain slurry consolidation testing was also performed on samples of whole tailing materials with loads up to 2,000 kilo pascals (kPa) to aid in consolidation modeling of the impounded tailings. Based on results of the laboratory testing and the experience of KCB from other projects, the average dry density adopted for the tailings is 1.53 t/m^3 for sizing of the TMF. The Board's experience with other projects is that the settled density may not achieve this average density value. *In the Board's experience, the settled density of the mixture of impounded whole tailing materials and cyclone overflow will be on the order of 1.0 t/m^3 early in the life of the TMF and that it will gradually increase over time to a maximum average settled density of approximately 1.4 t/m^3. For planning purposes at this PFS stage of the Project, the Board recommends that this increase over time be evaluated and taken into account, as appropriate. It is suggested that an initial density of 1.0 t/m^3 be assumed, increasing gradually to about 1.2 t/m^3 in two years and to a maximum density of about 1.4 t/m^3 in about five years of operation, should be considered to provide increased confidence that the starter facilities will be adequately sized and that sufficient material for raising of the dams is available.*

Large strain consolidation testing was also performed on the CIL tailing materials. An average density of 1.57 *t/m^3 was* adopted for these materials for sizing of this section of the TMF. *The Board recognizes that the CIL residue has a high specific gravity and will achieve high density relative to the floatation tailings, but it suggests using data from the large strain consolidation testing to develop a curve with conservative initial densities and increasing over time. The density curve should recognize that the CIL residue will be deposited into water, which will result in lower density values than above water deposition. The PFS material balance calculations should be based on this density curve.*

Permeability and Strength

Permeability and strength parameters for the various tailing materials were estimated based on laboratory test results and the following values were adopted for design (KCB, December "2012 Engineering Design Update of Tailings Management Facility":

Table 5.1-2. Tailing Permeability and Strength Parameters for Design

Tailing	Fines Content (% <74 µm)	Vertical Hydraulic Conductivity K_V (m/s)	Vertical Hydraulic Conductivity K_h (m/s)	Vertical Conductivity Ratio K_h/K_v	Peak Friction Angle φ'
Flotation Tailing	50 – 65	5×10^{-8}	5×10^{-7}	10	32°
CIL Tailing	> 90	2×10^{-8}	2×10^{-7}	10	32°
Compacted Cyclone Sands	17	3×10^{-6} (σ_v'<1 MPa) 2×10^{-6} (1 MPa<σ_v'<2 MPa) 1×10^{-6} (2 MPa<σ_v'<3 MPa)	1×10^{-5} <1 MPa) 8×10^{-6} (1 MPa<σ_v'<2 MPa) 4×10^{-6} (2 MPa<σ_v'<3 MPa)	4	34°
Flotation Cyclone Overview Fines	> 80	1×10^{-9} – 5×10^{-8}	1×10^{-8} – 5×10^{-7}	10	33° - 37

σ_v' = vertical effective confining stress

Based on its understanding of these materials, the Board finds these permeability and strength values to be reasonable.

Geochemistry

It was indicated to the Board that the floatation tailings are non-PAG as it is expected that the sulfides will be floated out. The CIL residue is characterized as being PAG with an estimated 40% to 45% sulfides content. It is planned that the CIL residue will be kept submerged in the CIL Residue Cell to prevent oxidation.

Deposition Slope Angles

Average slope angles that have been assumed for the floatation tailings above and below water are 0.5% and 3%, respectively. The average slope angle assumed for the CIL, which will be below water, is 3%. The Board is in agreement with the slope angles below water. The Boards experience with above water deposition of cyclone overflow on other projects with tailings ponds that have high rates of rise, is that the slopes can be as flat at 0.2% to 0.3%. *Therefore, the Board recommends that an average slope flatter than 0.5% is considered in a parametric evaluation for the floatation tailings above water, particularly during the summer season when the tailings will be cycloned.*

Cyclone Sand Production

The prefeasibility design envisions two-stage cycloning with the first stage at a central cyclone station and the second stage on the dam with mobile cyclone stations, similar to the Highland Valley Copper operation in BC. Two-stage cyclone simulation modeling was done by Krebs Engineers to evaluate cyclone performance. The simulation modeling assumed two different tailing feeds to the primary cyclone, one with a P_{80} of 150 microns and the other with a P_{80} of 120 microns. For the P_{80} of 150 microns, the model predicted 40% sand recovery with a fines content of 7% and an underflow slurry density of 68% by weight. For the P_{80} of 120 microns, the model predicted 33% sand recovery with a fines content of 11% and an underflow slurry density of 68% by weight. *The Board recommends two-stage cycloning continue to be considered at this stage to produce a sand product with less fines, which will be more permeable*. This will improve drainage during compaction, but more importantly, it will enhance drainage into the face of the dam structures during precipitation events and thereby minimize erosion. Erosion of tailings sand

from the downstream face of the tailings dams is a material concern in the relatively high precipitation environment of the site.

Testing done for the PFS design suggests that single stage cycloning may be possible. *Therefore the Board also suggests that consideration should be given to a trade-off study between the current two stage proposal and having both the single and two-stage cycloning at a central station*. The Board believes having a central cycloning station would have the following advantages over having the second stage cyclones on the dam:

- Better control of operating parameters resulting in better controls and a more uniform underflow product (both gradation and slurry density);
- Easier to maintain the secondary cyclones if they are at a central station where they are easily accessed and a permanent crane can be used for maintenance; and
- Simpler piping arrangements on the dams, which can be relocated more easily and which would result in less interference with construction equipment.

5.1.7 Water Chemistry and Quality

The water chemistry and quality assessments, modeling and prediction are not within the mandate of the Board. However, the effects of water quality on the ability to discharge water from storage reservoirs; surface or seepage discharges from diversion ditches; or discharges and bypass seepage at Seepage Collection Ponds, determine many of the design criteria related to these structures. *The Board seeks to be advised of the pertinent contaminant sources that affect water chemistry and quality assessment and prediction results, as well as the water quality standards that have to be met and the locations of monitoring points. It also needs to be advised regarding seepage and discharge rates that will be acceptable so as to establish the efficiency that must be met in containment control provided by designs.*

With regard to the tailings and the potential for ARD, the Board was advised the floatation circuit in the plant is efficient in removing sulphides from the tailings and that these would not be acid generating. There may be other constituents that may be leached at neutral pH. In addition, there will be constituents in the tailings water resulting from contact with ore (some of which may be partially oxidized) and due to the introduction of water in the plant from other mine sources which may contain dissolved constituents.

The CIL tailings will contain high concentrations of sulphides (~50%) and would be highly acid generating. Placement under water in the CIL Residue Cell is therefore appropriate to control potential oxidation. With oxidation controlled, it is anticipated the contaminant loads resulting from ARD activity within the pond would be extremely low. The water cover over the CIL tailings would be required to be sufficiently deep to prevent suspension and distribution of CIL tailings on beaches. The Project Team has proposed that prior to closure the surface of the CIL tailings will be flooded with a depth of floatation tailings to prevent such suspension in the long term. This is viewed by the Board as a robust solution. The residual cyanide content of the CIL tailings is a contaminant of particular concern, which is the rational used for lining of the CIL TMF Pond.

The Board seeks to be advised of the water quality that is anticipated for each of the tailings ponds, as well as that yielded to groundwater where seepage occurs.

Based on water chemistry data provided, the strategy for controlling water quality and seepage from the TMF is considered by the Board to be well conceived.

5.1.8 Strategy for Water Management

The water balance for the TMF is positive for the entire operating life of the Project and requires that water be discharged from the TMF each year to avoid accumulating water on a year over year basis. Planning for and implementing this annual water release will be the most important water management activity for this facility.

Spillways are not provided for the tailings dams during the time that the dams are being raised using cyclone sand; rather the TMF is designed to have the capacity to store the total runoff from a 30-day probable maximum flood event, considering that all the water diversion systems have failed.

Water management for the TMF involves diverting the natural runoff (non-contact water) around the TMF to Teigen Creek in the north and Treaty Creek located south of the TMF using ditches, pipelines and a tunnel. Water that collects in the TMF is stored and then released to Treaty Creek at flow rates that prevent degradation of water quality in that creek. Storage is provided in the TMF for the 30-day Probable Maximum Flood (PMF) event. *Since relative short duration flood events, such as 30 days, are seldom the cause of overtopping, the Board recommends the designers consider a critical duration PMF event, i.e., an event that results in the maximum elevation rise in the TMF and which takes into account the water gains and losses during that period. The recommend forecasting method discussed below would accommodate and define such a critical duration event.*

A staged plan has been developed that matches the development sequence of the TMF. The stages are:

Construction:

- Establish the Northeast Diversion and East Catchment stream maintenance flow pipeline to provide 2 m^3/s of flow to Teigen Creek during winter low flow months, which is required for maintenance of fish habitat;
- Tunnel constructed in East Valley as a mitigation measure should landslides block the pipeline inlet;
- Divert TMF North Cell perimeter slope runoff to Teigen Creek to facilitate construction dewatering and sediment control;
- Build NSCD and pond to function as construction period sediment collection pond;
- Cofferdams, local diversion pipelines and diversion channels used around dam footprints and borrow areas to control erosion and dewater construction sites; and
- At plant start-up, the South Diversion is extended along the entire south site of the TMF valley to capture local flows from the south valley slope and divert these to Teigen Creek.

Stage 1 – North Cell Operation:

- Northeast Diversion and East Catchment buried pipeline continue to divert non-contact runoff to Teigen Creek;
- East Valley flood flows continue to be diverted to Treaty Creek;
- Water from CIL Residue Cell pond is reclaimed for use in Process Plant or treated and discharged to North Cell;
- Total suspended solids (TSS) are settled out in North Cell pond and the water is discharged (pumped) to the Treaty Creek Diffuser at a rate to match stream flows during the period matching the flows during spring freshet to the end of the annual high flow period (approximately May to October) each year;

- Runoff and solids from hydraulic placement of cyclone sand are collected at the dam toes and pumped back to the Cell;
- Water collected in the SCD ponds is pumped to the Cells;
- Inspections, monitoring and maintenance of diversion channels and diversion pipelines is carried out; and
- The diversion tunnel from East Catchment to Teigen Creek is constructed in years 20 to 25.

Stage 2 – Transition between North and South Cell Operation

- Northeast Diversion routes non-contact slope runoff to Teigen Creek;
- East Catchment floods are routed to Teigen Creek via a tunnel;
- TMF South Diversion routes south side perimeter runoff to South Teigen Creek;
- Southeast Diversion routes north side perimeter runoff to North Treaty Creek;
- Water for discharge from the TMF is pumped to Treaty Creek Diffuser for discharge;
- North Dam is covered and reclaimed using excavated rock;
- Runoff and solids from hydraulic placement of cyclone sand are collected at the dam toes and pumped back to the Cell;
- Water collected in the SCD ponds is pumped to the Cells; and
- Inspections, monitoring and maintenance of diversion channels and diversion pipelines are carried out.

Stage 3 – South Cell Operation

- Northeast Diversion routes non-contact slope runoff to Teigen Creek;
- East Catchment floods are routed to Teigen Creek via a tunnel;
- TMF South Diversion routes south side perimeter runoff to Teigen Creek;
- Southeast Diversion routes north side perimeter runoff to Treaty Creek;
- TSS are settled out in South Cell and water from the North and South Cells routed to the Treaty Creek Diffuser at a rate to match stream flows during the period from about May to about October each year;
- Construct spillway for the North Cell after water quality improves to a level suitable for discharge to Teigen Creek;
- Water collected in the SCD ponds is pumped to the Cells; and
- Inspections, monitoring and maintenance of diversion channels and diversion pipelines are carried out.

Stage 4 – Initial Closure Period

- Northeast Diversion routes non-contact slope runoff to Teigen Creek;
- East Catchment floods routed to Teigen Creek via tunnel;
- Southeast Diversion routes north side perimeter runoff to Treaty Creek;

- South Diversion routes south side perimeter runoff to South Teigen Creek via North Dam Closure Spillway;
- South Cell discharge water pumped to the Treaty Creek Diffuser during the period from about May to about October each year at a rate to match stream flows;
- Water from the closed North Cell is discharged through the North Dam Closure Spillway to Teigen Creek;
- CIL and South Cell level is adjusted to match the future spillway invert;
- Southeast Dam is covered and reclaimed using excavated rock and Southeast Dam Closure Spillway is excavated; and
- Closure and reclamation of South Cell and CIL Residue Cell commences.

Stage 5 – Final Closure

- Final closure begins when water quality meets discharge requirements;
- All diversion channels, tunnels and pipelines are decommissioned and slope runoff is directed to ponds located in each cell;
- Two spillways are provided, each capable of conveying the PMF flow; and
- Flows regulated in spillways to replicate pre-mining flow patterns in Treaty Creek and Teigen Creek.

During North Cell operation the average annual discharge requirement is 300 l/s, with the total average discharge over the six-month discharge period being 600 l/s. During this period the maximum daily discharge rate will be approximately 2,100 l/s to match spring freshet flows. During operation of the South Cell, the average annual discharge requirement will increase to about 600 l/s and the discharge required during the six-month period will increase accordingly.

Updates to the Water Management Plan should consider contingency measures for:

- Variation of water quality as ore types change raising the potential for water quality not being suitable for discharge during some portion of the May to October period;
- Low stream flows during the discharge period not allowing planned quantity of water to be discharged;
- Accumulation of water during large flood events, and need to discharge water to recover capacity to store capacity for the design flood event; and
- Damage to the water diversion system and accumulation of water while repairs are made.

Mitigation measures could include raising the dams faster to provide additional water store capacity and construction of a water storage facility outside the area of the TMF. The Southeast starter dam could also be constructed earlier, to provide additional water storage capacity during the North Cell operating period.

The East Valley Diversion Tunnel could be located so that it can be used as a temporary spillway for the North Cell during the 25 years of operation of the South Cell. An inlet structure and short connector tunnel would be needed between the North Cell pond and the diversion tunnel. This could delay the time for the North Spillway and channel to be constructed and could simplify water management in the left abutment area of the North Dam.

On closure, the three tailings cells are partially covered with soil and permanent water ponds would form in the west central areas of each cell. These ponds join along the western perimeter of the TMF. In addition, the CIL cell will be covered with flotation tails and the water pond level will be high enough to keep the underlying CIL tailings permanently flooded. Excess water that accumulates on the TMF surface and in the ponds would be discharged to both Teigen and Treaty Creeks. Flow control gates and weirs would be used to control the amount diverted to each of the creeks. Diversion ditches would be maintained along the north- and south-western perimeters of the TMF to divert natural runoff around the TMF.

The Board agrees that this general approach is appropriate. Risks include failures of the diversion system due to geologic hazards, water quality that is poorer than projected, and encroachment on the operation freeboard during operations which could lead to overtopping. These have been minimized by the design that includes geologic hazard mitigation and contingency measures, contingency water treatment if necessary, and the establishment of tailings operation and water management plans and procedures.

Since the avoidance of any overtopping risk during operations is a critical element of the TMF, the Board considers it extremely important that, during operations and water management, plans be expanded to incorporate water balance forecasting for the future two to three years on a regular basis (e.g. monthly) to establish at each time whether the design flood event can be stored without overtopping. This will allow contingency measures to be implemented before the water levels reach a critical stage. This method also allows for more practical management of seasonal fluctuations in the TMF water levels since the projected effect of the design flood event immediately after the freshet will be less than its projected effect just before the freshet.

Projecting two to three years into the future from an actual measured water level in the TMF also avoids the persistent problem of determining when there is risk that the water levels can encroach into the required freeboard during a wet period. By using the results of the forecast decision making becomes simpler. In the event the forecast predicts potential for overtopping for the design flood event, appropriate contingency measures are immediately undertaken.

5.1.9 Screening and Selection Studies

The tailings storage alternatives considered conventional on-land storage, ocean disposal, and alternate tailings technologies. Initial screening carried out between 2004 and 2011 considered alternative technologies but identified technical challenges that resulted in selection of conventional slurry tailings as the Best Available Technology. The conclusion, as presented in EA Documents, was that the potential environmental benefits of using an alternative tailings technology are not significant and are outweighed by the technical challenges and risks and increased cost associated with the alternatives.

The screening for conventional sites covered potential tailings storage sites within an area of approximately 50 km by 50 km. 14 potential tailings storage sites were identified for assessment using Environment Canada's guidelines for the assessment of tailings impoundment areas. The sites included areas adjacent to the mine and sites remote from the mine associated with major streams or drainages that potentially had the required tailings storage capacity.

Alternatives were assessed based on conceptual designs developed for each site. Dam heights of up to 250 m were used as a maximum design limitation, based on precedents for existing similar dams. Dams were designed to the 2007 CDA Dam Safety Guidelines. The assessments incorporated the results of site-specific investigations that were carried out for most of the sites. Hydrological aspects and the requirements for spillways and diversions were examined for each site. The feasibility of measures to manage the estimated floods at each site was assessed. In addition, the feasibility of operational water management was assessed using a water balance developed on the basis of the un-diverted catchment

area, precipitation data and the tailings deposition plan to determine surplus water quantities, which would potentially require treatment prior to discharge.

A comparative ranking between sites was developed using parameters related to capacity of the facility, water management during operations and post closure, ease of operations, geologic hazards, constructability, and capital and operating cost. Sites were ranked as having the required storage capacity and being feasible for development, as having significant storage capacity and being potentially feasible for development; as having significant storage capacity but not feasible for development; or as having insufficient storage capacity and other issues not feasible for development.

Of the 14 sites identified, only the South Teigen, North Treaty Creek site had the capacity to store 2.2 Bt of tailings. The site with the next largest capacity (Upper Treaty Alternative) could store 70% of the projected tailings but had significant water management issues due to the 114 km^2 catchment area; significant geohazards related to avalanches and highly glaciated upstream areas.

The screening and site selection study has been carried out in sufficient detail to define the attributes of the potential sites and the ranking system includes environmental (water management) and societal issues, as well as cost and operating related issues.

As discussed in Section 1.5, SG/P is conducting a more detailed evaluation of BAT for tailings management applicable to KSM. This assessment is evaluating filtered tailings options, as well as slurry, thickened tailings and paste alternatives that may potentially achieve the goal of reducing the consequence of failure by reducing the mobility of the tailings, or reducing the quantity of water stored in the TMF.

5.2 DAMS

5.2.1 Storage Dams

5.2.1.1 Dam Design

The North, Saddle and Southeast Dams are proposed to be compacted cyclone sand dams constructed by the centreline method with a crest width of 20 m, and downstream slope of 3H:1V. The details of the dam designs are summarized in Table 5.2-1 below.

Table 5.2-1. Dam Designs

Dam	Crest Elevation (m)	Maximum Height (m)	Ultimate Crest Length (m)	Cyclone Sand Volume (Mm^3)	Core Volume Above Starter (Mm^3)
North Dam	1,068	218	1,900	47.16	3.42
Splitter Dam	1,068	194	1,930	31.31	3.75
Saddle Dam	1,068	168	1,600	22.99	3.39
Southeast Dam	1,068	239	1,400	60.45	3.20
Total			**6,830**	**161.91**	**13.76**

The Splitter Dam separates the North and CIL Residue Cell. Its crest elevation will rise with the North Cell tailing level. The dam will be built with the centreline method using compacted cyclone sand; however it is planned that the downstream part of the embankment will be supported by CIL tailings as the CIL Residue Cell is filled. Thus, the raising schedules for the North Cell and the CIL Residue Cell would be coupled. *The Board is of the opinion that the coupling of the raise schedules for the North, Splitter and Saddle Dams could be removed by constructing the Splitter Dam section with compacted underflow for its*

full section to eliminate the dependence of the construction schedules for these two cells on one another. This would require more cyclone sand, which would need to be considered in the material balance. A trade-off study to evaluate the benefit of partial or total decoupling is recommended.

A vertical till core is provided in each dam to restrict seepage. The width of the core is 20% of the dam height, with a minimum width of 20 m to allow for possible future dam raises. The core will be constructed of locally borrowed till with a minimum fines content of 25% and a maximum hydraulic conductivity of $1x10^{-7}$ m/s. A geomembrane liner will be installed in the cores of the Splitter Dam and the Saddle Dam and will be connected to the liner that surrounds the remainder of the CIL Residue Cell. A system of finger drains will be installed at the base of the cyclone sand shell to keep the water level depressed within the dam sections. Main drains in the centre of the valley floor will collect and convey seepage to the toe of the dams. Smaller secondary drains will convey water laterally into the main drains. All drains will comprise an inner zone of highly pervious, processed 25 mm to 100 mm drain rock. This inner zone will be surrounded by 0.5 m of well graded 38 mm minus sand and gravel filter (<2% fines) to prevent piping of the cyclone sand or native foundation soils into the drain core. All drains will be installed in trenches excavated into the native ground to prevent erosion of the drains during initial placement of the cyclone sand and to provide pressure relief in the foundation soils. The total flow capacity of the drainage system will be at least 10 times the expected normal seepage from the dam and underlying foundation soils.

The Board notes that a zone of compacted cyclone sands will need to be placed upstream of the till cores to provide support for the core and to prevent cracking, and that the material placed upstream will need to be founded on firm foundations. The width of this zone should be established in future engineering studies, but for planning purposes in the PFS, a rational basis for the minimum compacted width upstream of the core is required. Material balance calculations will need to account for this additional cyclone sand requirement.

The Board is of the opinion that cyclone sand is an appropriate construction material for the dams. However, because of the high precipitation environment, the Board recommends that the design includes measures to control erosion of the sand embankment. In particular, other projects have experienced significant erosion at the interface of the sand embankment with natural ground at the downstream groins of the dam. Paddy or cell construction has the advantage of capturing runoff from precipitation and presents an opportunity to leave benches on the downstream slope to help capture and manage runoff.

The starter dams will be earth-fill embankments with shells of compacted random fill supporting the central till core. The core will be keyed into the underlying foundation to cut off seepage through weathered near-surface soils and any pervious strata. A geomembrane liner will be installed in the cores of the Splitter Dam and the Saddle Dam. A 1.5 m thick drain blanket will be provided to depress phreatic levels in the downstream half of the starter dam sections. The details of the starter dam designs are summarized in Table 5.2-2 below.

Table 5.2-2. Starter Dam Designs

Starter Dam	Crest Elevation (m)	Maximum Height (m)	Ultimate Crest Length (m)	Random Fill Volume (Mm^3)	Core Volume (Mm^3)
North Dam	930	80	680	3.59	0.95
Splitter Dam	935	61	890	3.74	1.08
Saddle Dam	935	35	780	2.09	0.75
Southeast Dam	930	101	890	12.32	1.72
Total			**3,240**	**21.74**	**4.50**

The till for the core will have the design as described above. Compacted random fill of the outer dam shells will be placed and compacted similar to the core, but the material properties will be less restrictive. The drainage blanket will comprise a well-graded sand and gravel or crushed rock with a maximum particle size of 38 mm and less than 2% fines passing the No. 200 sieve.

As previously mentioned, the Centre Cell will be geomembrane lined. A drain beneath the pond bottom is included in the current design but extending the drain up the sides slopes will need to be included to prevent floating of the liner. Criteria should be developed for water control beneath the liner based on water level and water quality. Levels outside of the cell should be maintained slightly lower than the water level in the Centre Cell. Examine requirements for extending the existing pumping system to areas where high inflows may be encountered in order to maintain local water levels slightly below the water level in the centre cell. If water quality requires, provide capability to discharge pumped water to centre cell. The Board notes that surface water controls and sediment management will be critical during initial capital construction to avoid environmental impacts on water quality. The Board understands such plans are being developed and looks forward to reviewing these at the FS design stage.

5.2.1.2 Tailings Deposition

Flotation tailings will be routed by gravity from the Process Plant to the TMF during early years and will be pumped during later years. Flotation tailings will be distributed to the TMF as follows:

- During the “winter” months of November to March, Flotation tailings will be discharged into the impoundment discharge points located every 200 m along the crest of the dams. This will develop the beach and push the tailings pond towards the centre of the cells. The deposition will also build up the tailings beach against the dam to support the construction of the central till core during the following summer.
- During the “summer” months of April to October, underflow from the primary cyclone station will be pumped to secondary on-dam cyclone stations. The cyclone underflow sands will be conveyed in a pipeline at a solids density of 60% to 68% on the downstream dam slope to the construction areas where the sand will be sluiced into cells and compacted. The cyclone overflow will discharge onto the upstream beach. Target parameters for cyclone underflow sands are fines content less than 17% and a vertical percolation rate in the initially compacted sands greater than $3x10^{-6}$ m/s.

Since water is only planned to be discharged to Treaty Creek during the months of May to October, it appears that initially 20 to 25 meters of freeboard, reducing to 10 m at maturity, will need to be provided going into the winter in order to provide enough storage for the deposited tailing materials and accumulated water and to provide storage for the design flood. In the experience of the Board, provision for this amount of freeboard will require a substantial upstream compacted zone. The Board looks forward to being provided with the design basis in the FS.

Considering the earthwork that will be needed to build the freeboard and core zones during the summer season, the Board recommends that the crest width above the impounded tailings be increased to provide sufficient space to place materials in three different areas independent of one another; i.e. (1) on the crest to raise the core, (2) a compacted cyclone sand placement area used to raise the area upstream of the core, and, (3) a compacted cyclone sand placement area to raise a wider zone downstream of the crest. The construction sequences for each step of raising the dams will be critical and must be well planned and executed. Underdrain capacity should be sufficient to handle construction water from maximum sand production placement rates at any one sand placement location.

CIL tailings will be deposited subaqueously, year round in the CIL Residue Cell. Two different deposition schemes are planned to facilitate subaqueous tailing deposition throughout the year:

- For “winter deposition”, four floating tailing lines will be placed at equal intervals in the pond and moored in place before the pond freezes. Their location will be decided based on bathometric survey of the tailing pond carried out in the fall. Enough spigot outlets will be placed to provide capacity for tailing deposition to continue over the entire winter period, until the ice melts in the spring. Additional lines will be provided for redundancy. Winter tailing lines are to be floating lines to avoid having them buried as tailings are deposited.
- “Summer deposition” is planned to occur along the perimeters of the Splitter and Saddle Dams. Sacrificial cyclone sand berms will be pushed into the CIL residue tailing pond to form small benches above the water level. Tailing lines will be laid out on the berms and from these points tailings will be spigotted subaqueously into the pond.

5.2.1.3 Cyclone Sand Placement

The sand shells of the dams will be constructed by hydraulic sluicing of the sand into cells oriented parallel to the dam crest. “Dykes” of sand will be pushed up by dozers to confine the perimeter of the cells. The cells will be nominally 10 m to 50 m wide and 150 m to 300 m long. For each year of construction, sand placement will start at the downstream toe of the dam and create a berm raised up the dam slope to required crest elevation. Because the final crest elevation will not be achieved until October at the end of each construction season, each year’s dam raise will provide the required storage needed through October of the following year.

A spill box will be installed at the downstream end of the cells to decant the release water and segregated sediment from finer tailings out of the cells. The outflow will be carried in a pipeline to the dam toe where the slurry of fines and construction water will be pumped back into the TMF impoundment. When sand builds up at the discharge end of the cells to between 3 m to 6 m, the cell deposition area will be advanced along the dam slope. The cycle will be repeated when the full length of the dam has been raised 3 m to 6 m. This technique of “cell construction” has been used extensively at Highland Valley Copper, Kennecott Utah Copper, and Southern Peru Copper and in the oil sand industry.

During operation, the dozers will be used to maintain the side dykes and to grade sand within the cells to provide positive drainage of water from the cell. The sand placed in the cells will receive compaction by the trafficking of dozers and the effects of downward drainage into the base of the cell.

The available volume of cyclone sand for dam construction during a seven month construction period (April to October) has been estimated by KCB to average 4.5 Mm^3 per year, yielding a total of 232 Mm^3 over the 51.5 year operating life. This is based on the following assumptions:

- Design ore throughput of 2.2 Billion tonnes over 51.5 years;
- 90% of tailings are flotation tailings;
- 7 of 12 months cyclone season;
- 95% efficiency of cyclone utilization;
- 24% solids recovery from cyclones;
- 95% sand capture in the cells; and
- Deposited dry density of 1.55 tonnes/m^3.

The total volume of the cyclone sand dams for the ultimate dam crest elevations of 1,068 m has been estimated to be 162 Mm^3. While the PFS concludes that, there would be sufficient sand available for dam construction, the Board has noted that additional sand could be required to account for the following factors:

- Additional sand for the Splitter Dam to decouple the construction sequencing of the North Cell and the CIL Residue Cell;
- Additional placement of compacted cyclone sand in a wider zone upstream of the core of the dams;
- Increasing the width of the crest to accommodate embankment construction in three separate parts of each dam independently; and
- Placement of sufficient embankment material to provide freeboard going into the winter for storage of winter tailings deposition, accumulated water in the TMF impoundment and design flood storage
- Allowing for storage of a PMF flood of critical duration

The Board recommends that, for the PFS, an initial assessment be made to develop a better understanding of the freeboard requirements and that initial estimates of the material balance and construction sequencing also be made for the cyclone sand requirements for the above described factors. A detailed evaluation will be required for FS. In addition, the Board recommends that the updated material balance assessments include sensitivity analyses to consider possible variability of the following:

- Winter start-up;
- Finer initial grind;
- Initial Process Plant or cyclone/delivery system upset;
- Rapid Process Plant ramp-up;
- Low achieved density of settled tailings;
- Higher compacted density of sand;
- Increased water due to extreme events (less than PMF) or quality constraints on discharge;
- The possibility of six months of cycloning per year rather than 7 months; and
- The possibility of non-dischargeable water for 6 to 8 months.

The Board is of the opinion that starter dam heights may need to be increased, or geometry modified, to accommodate possible sand shortfalls resulting from the above considerations and recommends that the PFS is updated as a matter of priority, due to the possible implications on starter dam height and subsequent raising of the dams. The geometry of the embankments developed in PFS will need to consider sand availability in the first year depending on the time of startup, commissioning of the cyclones and potential cyclone commissioning delays relative to production ramp up, and water control structures internal to the TMF impoundments to facilitate construction in the dry. Clearly, the climate at the site will require cyclone operation and embankment construction to be maximized in the summer to be able to generate sufficient operating freeboard for the winter period.

5.2.1.4 Dam Site Preparation

Vegetation on the ground surface in the starter dam and cyclone sand dam footprints will be stripped prior to construction. The stripped surface will then be scarified and compacted. The stripping is planned to be carried out in stages in advance of each year's dam construction.

The ground surface extending 500 m upstream of the tailing dams will also be stripped of soil which will be stockpiled for reclamation at closure of the TMF. The exposed ground surface will be inspected for evidence of highly pervious zones (talus or colluvial materials) that could cause increased seepage or piping of tailing into the foundation.

5.2.1.5 Seepage Cut-offs

A key trench will be excavated below the North Dam starter dam to tie the till core into lower permeability till or to bedrock. Slurry trench cutoff walls will be built where weathered near surface soils and pervious strata are too thick for key trenches. The slurry trench cutoffs will comprise a 1 m wide excavated trench, backfilled with native soils amended with 2% to 4% bentonite, to achieve a hydraulic conductivity less than $1x10^{-8}$ m/s.

At the Splitter and Southeast Dams, a 1 m wide soil-bentonite slurry trench cutoff wall will also be excavated through the pervious alluvial soils below the centreline of the starter dam. These walls will extend 3 m into low permeability basal till or to bedrock. The estimated maximum wall depth is 25 m. The slurry wall backfill will be comprised of the excavated alluvial soils amended with 2% to 4% bentonite.

5.2.1.6 Seepage Analyses

Two dimensional seepage analyses were carried out to estimate seepage through the cyclone sand dams, to size the underdrains that will be installed in the dams, to predict piezometric levels for analysis of dam stability and to estimate seepage flows that will report to the drain systems. A three dimensional groundwater flow modeling assessment was also undertaken by KCB using FEFLOW, primarily to assess the performance of the seepage mitigation system. The FEFLOW model incorporates additional details of the foundation bedrock which are not included in the two dimensional analyses. *The Board notes that the two dimensional seepage analyses may not account for all of the seepage in the foundation materials beneath the dams. In particular, seepage that flows in the permeable surface soils of the valley side slopes and reports to the TMF needs to be included in the seepage estimates. The Board recommends that more detailed three-dimensional hydrogeologic modeling is done in future FS phases of the design so that this foundation seepage in the valley slopes is included in estimates of seepage flows in the TMF foundation. This could potentially increase the design capacity of the drains.*

5.2.1.7 Stability Analyses

Table 5.2-3 below lists the feasibility level design criteria for the tailing dam stability.

Table 5.2-3. Feasibility Level Design Criteria for Tailing Dam Stability

Loading Condition	Design Standard	Minimum Safety Factor
Static Loading Conditions	Two-Dimensional Limit-Equilibrium factor of safety (FOS) with operating pore pressures	1.5 for operating conditions and closure
Pseudo-Static Loading Conditions	Two-Dimensional Limit-Equilibrium FOS with operating pore pressures and horizontal pseudo-static coefficient of 0.07 g	1.0 assuming 50% strength reduction of un-compacted tailing deposits
Post-Earthquake Loading Conditions	Two-Dimensional Limit-Equilibrium FOS with operating pore pressures	1.2 assuming full liquefaction of un-compacted tailing deposits

The design earthquake selected for the project Maximum Credible Earthquake (MCE) with a peak ground acceleration of 0.14 g.

Two-dimensional limit equilibrium stability analyses of the tailing dam were carried out for the North Splitter, Saddle and Southeast Dams using the computer program SLOPE/W. Planar and circular slip surfaces were analyzed using the Morgenstern-Price method of slices. Pore pressures in the tailing and foundation soils were input as piezometric elevations based on the seepage analyses discussed above.

The calculated minimum static FOS for the four dams range from 1.7 to 2.2, which are well above the minimum design criteria listed above. The pseudo-static minimum FOSs range from 1.2 to 1.3 for all dams. The minimum modeled post-earthquake factors of safety, ranging from 1.2 to 1.6, are above the design criteria for all dams. The only exception is the post-earthquake scenario for the Splitter dam at the end of Stage 1 with a factor of safety 1.14. This was considered by KCB to be adequate given that this dam is an internal structure and the FOS will be greater than 1.2 during later stages and closure.

Based on a foundation liquefaction potential assessment, the North Dam will be founded on dense till, and any loose or soft overburden surficial soils underlying the dam will be stripped or removed prior to construction of the dam; hence liquefaction is not expected to occur within the North Dam foundation.

Alluvial soils below the Saddle Dam toe are potentially liquefiable during an earthquake. A 35 m high and 100 m wide toe-berm is proposed at the Saddle Dam to prevent liquefaction and meet the design criteria. *Since instability due to liquefaction can be mitigated with shear keys as an alternative to the proposed toe berm, the Board recommends evaluation of both the toe berm and shear key alternatives.*

Assessment of the Southeast Dam foundation soils found only pockets of soil with potential for liquefaction in relatively thin layers, less than 1 m thick, which are not expected to be spatially continuous or pervasive. KCB does not expect the seismic stability to be affected by such localized liquefaction.

5.2.1.8 Borrow Sources

Borrow material for construction will be obtained locally. Three types of geologic materials are available for building the TMF starter dams and seepage recovery dams, and for other construction purposes:

- Glacial till;
- Alluvium; and
- Waste from rock excavations (including tunnel mining) and quarries as required.

To assess borrow availability, glacial till with a fines content of at least 25% is deemed by KCB to be suitable for use as dam cores. Glacial till with less than 25% fines content and alluvium are planned for use as dam shells and for other random-fill needs.

Waste from rock excavations is suitable for use as concrete aggregate, filters, and drains, provided:

- The rock is not potentially acid generating; and
- The rock is not subject to alkali-silica reactivity.

The availability of glacial till and alluvium as borrow material is related to the distribution of surficial geologic deposits.

Near the North Dam, glacial till is the only surficial deposit available in large volumes, although a small amount of alluvium is present along the South Tributary of Teigen Creek and a thin veneer of colluvium and organic soil overlies both bedrock and till deposits.

In the vicinity of the Splitter and Saddle dams, till deposits are present along the west side of the valley; the till is locally overlain by small alluvial fans. Till deposits in this area above creek level are located above the water table and are thus suitable for use as dam core materials. Bog deposits along the east side of the valley are unsuitable as building material. Both till and alluvium underlie the bog deposits at depth, but are not considered as potential sources of borrow material due to depth of burial and saturated soil conditions.

Near the Southeast Dam, glacial till is present on both the east and west sides of the valley, but till deposits on the east are overlain by large alluvial fans. A small amount of alluvium is present along the North Tributary of Treaty Creek, and a thin veneer of colluvium and organic soil overlies the bedrock and till deposits.

Additional borrow may be available in the East Catchment Valley. Glacial till, alluvium, and colluvium (in the form of talus or scree cones) are present, but the volumes of available materials have not yet been fully defined. *The Board recommends that rock excavated for site grading at the Process Plant site, also be considered as a possible alternative or additional source of borrow material for use in constructing the shells of the starter dams and seepage recovery dams.*

5.2.1.9 Borrow Requirements vs. Borrow Availability

Estimates of available borrow materials from each borrow pit were derived from interpretations of seismic velocity profiles and borehole data. ‘Top of Bedrock’ and ‘Top of Till’ surfaces were estimated based on these data. Within the plan view extent of a borrow pit, till volumes between these surfaces were calculated as a function of elevation. Only till above average creek elevation was interpreted as available total till volume. Till volume with >25% fines content, suitable for dam core construction, was taken as 80% of total till available above creek level.

Alluvium overlying the borrow pits has been assumed to be suitable for dam shells. Alluvial material overlying till was calculated using stripping ratios derived from seismic velocity profiles. The stripping ratio was taken as vertical area of material overlying till, divided by vertical area of till above creek level. Alluvial material was then calculated by multiplying the stripping ratio with the till volume present above creek level.

Random fill available for dams shell construction was calculated as sum of alluvial volume overlying till, and till volume above creek level with <25% fines content.

Borrow requirements were computed from starter and seepage dam volumes based on design drawings. Table 5.2-4 below lists the volume of borrow material required for construction of starter and seepage dam cores and shells (“2012 Engineering Design Update of Tailings Management Facility”, KCB, December).

Relevant conclusions are:

- Comparing required core volumes to available till volumes with >25% fines content, sufficient material is available to construct starter and seepage dam cores in each area.
- Comparing required shell volumes with available random fill volumes stripped from till borrow pits, shows that available random fill alone is not sufficient to construct starter and seepage dam shells. However, excess till core material (i.e. >25% fines) can be used to construct upstream starter dam shells. Material available for shell construction in the table above is the sum of random fill plus excess till core material. These available volumes exceed required shell material in each area.

- Sufficient borrow sources for shell and core construction of Seepage and Starter dams have been identified. Additional borrow sources are present in the TMF area, but require further definition. Excess till with >25% fines content, not used to construct seepage and starter dam shells, may be stock piled or sourced during operations for use as core material in annual dam raises. If quantities and/or quality of till are not available for annual dam raises, then the till core may be augmented by a geomembrane liner or by a 6 m-wide zone of bentonite-amended till or cyclone sand.

Table 5.2-4. Starter and Seepage Dam Construction Borrow Requirements vs. Borrow Availability

	Required Borrow [1]			Available Borrow			
Area	**Dam Element**	**Core (Mm^3)**	**Shell (Mm^3)**	**Borrow Pit**	**Till >25% Fines [2] (Mm^3)**	**Random Fill [3] (Mm^3)**	**Available for Shell [4] (Mm^3)**
North Cell	Seepage Recovery Dam	0.02	0.08	X01	1.17	0.60	
	Starter Dam	0.95	3.59	N01	1.83	1.37	
	Total	**0.97**	**3.67**	**Total**	**3.00**	**1.97**	**4.00**
CIL Cell	Splitter Starter Dam	1.08	3.74				
	Saddle Starter Dam	0.75	2.09	N02	1.87	2.12	
	Splitter Seepage Recovery Dam	0.16	0.24	C01	3.20	1.92	
	Total	**1.99**	**6.07**	**Total**	**5.07**	**4.04**	**7.12**
South Cell				S01	4.07	4.32	
	Seepage Recovery Dam	0.03	0.07	Y01	1.48	0.89	
	Starter Dam	1.72	12.32	Y01	3.98	1.84	
	Total	**1.75**	**12.39**	**Total**	**9.52**	**7.05**	**14.82**

Notes:
1. Borrow requirement based on starter and seepage dam design drawings.
2. Available till borrow with >25% Fines is assumed as 80% of in-situ till volume above creek level.
3. Available random fill is assumed as stripping volume above till, plus 20% of in-situ till volume above creek level (i.e. till <25% Fines)
4. Available material for Shell = Random Fill + ("Available Till >25% Fines" – "Required Core".)

The Board notes that it is prudent to identify at least 1.5 times the required borrow material at FS level to account for unknowns and to provide confidence that shortages will not be encountered during construction. The Board recommends that at FS stage additional borrow investigations are conducted with the objective of identifying no less than 1.5 the required volumes of each construction material. The Board agrees with KCB's plan to utilize test pits in future borrow investigations when access is available for excavators to excavate large test pits.

Other observations by the Board are that till with fines contents less than 25% will likely satisfy the design criteria for core material and it is recommended that the adequacy of a lower fines content is evaluated. The Board noted that the laboratory testing on the till appeared to be on samples where the particles larger than 3 inches had been scalped off. Thus, the actual fines content of the full gradation would have been less if larger particles had been included in the samples and tested. It will be important to consider this factor when preparing the specifications in order to avoid confusion and to accurately define the available materials. Since the permeability of soils with oversize materials is usually controlled by the "matrix soils", it may be desirable to specify the percentage of fines in the matrix soils below some particle size rather than in the full gradation of the pit run materials.

5.2.2 Seepage and Sediment Control Dams

The seepage recovery dams use conventional water retaining dam technology. The dams are located sufficiently far downstream from the tailings dams that seepage beneath the tailings dams will emerge to surface and be collected by the seepage recovery dams. Three dams will be required, the North Seepage Collection Dam and the Saddle Seepage Collection Dam will be constructed as part of the initial construction and the Southeast Seepage Collection Dam will be constructed prior to operation of the South Cell.

The dams are assessed to have a "Significant" consequence classification under CDA guidelines. The design criteria include:

- Seismic: 2,475-year return period earthquake ground acceleration;
- Operating capacity: 14 days of tailings dam seepage assuming failure of pumping system;
- Flood: 200-year 24-hour flood event to be stored without discharge;
- Spillway: pass 500-year 24-hr flood event;
- Construction water storage: equivalent to 14 days of cyclone sand placement with tailings dam seepage assuming reclaim pump failure;
- Minimum pool depth: 6 m to float reclaim barge; and
- Freeboard: 3.0 m above maximum flood level.

The North Seepage Collection Dam will have a height of 30 m, measured from the downstream toe to the crest, and will be constructed with 3H:1V upstream and downstream slopes. The depth of soil in the foundation is up to about 15 m. The dam will be constructed of random fill with an inclined core on the upstream face. A drain will be constructed in the valley bottom to control the location of the phreatic surface within the dam. Seepage beneath the dam will be controlled using a cut-off trench beneath the core and a slurry trench cut-off wall through the deeper alluvium. Allowance is also provided for a three row grout curtain.

The Saddle Seepage Collection Dam will have a height of 15 m, measured from the downstream toe to the crest, and will be constructed with 3H:1V upstream and downstream slopes. Foundation conditions comprise alluvium in the valley bottom. The depth of soil in the valley bottom is up to about 15 m. The dam will be constructed of random fill with an inclined core on the upstream face. A drain will be constructed in the valley bottom to control the location of the phreatic surface within the dam. Seepage beneath the dam will be controlled using a slurry trench cut-off wall.

The Southeast Seepage Collection Dam will have a height of 35 m, measured from the downstream toe to the crest, and will be constructed with 3H:1V upstream and downstream slopes. Foundation conditions comprise alluvium and till. The depth of soil in the valley bottom is up to about 25 m. The dam will be constructed of random fill with an inclined core on the upstream face. A drain will be constructed in the valley bottom to control the location of the phreatic surface within the dam. Seepage beneath the dam will be controlled using a slurry trench cut-off wall and grouting of the bedrock foundation.

The 3 m freeboard above maximum water level is greater than expected for these dams given the heights, small water surface areas to generate waves, relatively low seismicity of the project area and low normal operating water level.

In each of the seepage collection dams, a geotextile is included between the upstream core material and the random dam fill. If the random fill will be filter compatible with the core material, the geotextile would be considered to be in a non-critical application for the use of geotextiles in dams. *However, if the random*

fill is not filter compatible with the core material, the geotextile may be viewed as being in a critical filter application and an additional zone of transitionally graded granular filter material may be required.

5.3 LINERS

A geosynthetic liner is proposed for the CIL Residue Cell. The Board makes a number of observations:

- The area is one of artesian pressures with upward flow in the base of the valley. The source and flow paths of this groundwater are not known in detail. While it is reasonable to observe that the valley has high bounding ridges in which there is recharge, the valley is a gaining stream which locally depresses the groundwater table, the actual local groundwater situation may be much more complex. There are potentially a number of shallow pathways for groundwater flow. These include talus on the valley sides, shallow weathered bedrock, faults and shear zones in the bedrock, and various more permeable channels in the valley alluvium. Recharge to any of these flow zones may occur rapidly, particularly in the period of freshet in the spring. The high fluctuations in water elevations in some piezometers on the site may be reflective of such mechanisms. Many of these flow paths may yield significant flows from springs that are active only in the freshet period. To relieve uplift pressures on the liner a substantial underdrainage system with appropriate high capacity at potential spring locations may be required. If there is uncertainty as to where such springs may occur, then an underdrainage blanket may have to be installed over a wide area, including the valley side slopes. *The Board recommends that the potential for high flows and pressures be assessed in an appropriate hydrogeological investigation and evaluation. Conservative assumptions can be made for PFS designs but for the FS the design of the underdrainage system should be developed from the results of such an investigation;*
- The effect of seepage from the North Cell and potential for uplift on the Central Residue Cell liner must be considered as previously discussed;
- The valley sides are steep and installation of liners may require considerable valley side shaping, removal of stumps and rock outcrop, and the construction of berms that enable reliable construction of an underdrainage system, liner bedding and liner that extends up valley sides. *Berms with benches will likely be required for installation, and also to provide access for liner inspection and repair;*
- The Board has not been provided with any water quality information for the CIL tailings and it is possible that an unlined CIL Residue Facility may provide for sufficient containment provided adequate seepage cut-offs are incorporated in the Splitter and Saddle Dams.
- The geohazards associated with avalanches and debris torrents during high precipitation events, or rocks dislodged during successive side slope earthworks for incrementally installing liner have a high propensity to bring down rocks and boulders, and spring discharges may cause liner uplift stresses that may materially damage the liner. Damage above the pond level could be repaired but damage below the waterline may be difficult or impossible to detect and affect repairs. *The integrity of the liner must be considered, monitoring and maintenance plans developed and risk assessment performed of the potential for excessive seepage;*
- *Thick ice formation on the CIL pond and ice rafting stresses on the liner must be considered*; and
- *The basis for drainage of water captured in the underdrainage system has to be established.* Consideration may be given to gravity drainage to the south until such time as the South Cell is constructed, after which consideration may be given to gravity drainage to the South Cell at higher elevations or pumping from a sump using a riser.

5.4 DIVERSIONS & TUNNEL

Diversion ditches and tunnels are provided to reduce the inflow of natural runoff water from the adjacent catchments into the TMF. These are built in stages over time to accommodate initially the North and CIL Residue Cells, and by year 25, the South Cell as well. The diversion systems are design for the 200-year 24-hour peak flows.

Diversion of runoff from the East Creek, which is the major contributor of runoff to the TMF area, is accomplished by installing a collection dam (the Lower East Diversion Dam) in the base of the East Creek channel, and using it to divert runoff via a pipeline (for low flows) and a ditch (for higher flows) to Teigen Creek downstream from the TMF. Since ground conditions immediately upstream of the Lower East Diversion Dam are poor and slope instability is a concern, a second collection dam, the Upper East Diversion Dam, is provided just over one km further upstream. In the event a landslide blocks the channel between the two dams, flows from the upstream collection dam are conveyed via tunnel to the lower collection dam.

The other potential risk associated with this diversion system is at the northern corner of the TMF where a “sagging” area has been geologically mapped. A slope failure in this area could direct flows from a breach in the diversion ditch or a pipeline onto the north abutment or downstream slope of the North Dam’s cyclone sand embankment resulting in erosion damage. This risk has been assessed by the KCB and determined to be low.

During Stage 2 of the TMF, the approximately 4 km long East Diversion Tunnel is constructed to convey water from both collection dams to Teigen Creek.

Diversion ditches flowing to Teigen Creek are provided along the southern perimeter of the TMF to divert natural runoff from the western catchment, which serve to compensate for water losses in Teigen Creek caused by the construction of the TMF. This approach is also planned for the southeastern perimeter, where ditches are provided to divert runoff to the Lower East Diversion Dam to the north and Treaty Creek to the south.

Geohazards are greatest on east side of the valley where slopes are steepest and snow avalanches pose the highest risks. Other hazards include debris flows and un-stable slopes in East Valley Catchment.

5.5 CLOSURE

A conceptual closure plan has been developed for the TMF that adequately covers all aspects of the works required to decommission this facility. Closure measures will be tested on the North Cell during the 25 year period of operation of the South Cell, and the final closure plan will be developed using that information.

A key aspect of the closure plan is the merging of the ponds in the three cells by breaching of the internal dams and provision of a spillway capable of discharging the required design flow at each end of the TMF as protection against blockage of one of the spillways. The spillway invert elevation has been selected so that the high sulphide content CIL tailings will be permanently below the water table to prevent acid generation. Water diversion channels, pipelines and tunnels will be decommissioned and water flow patterns that mimic the pre-development patterns will be re-established. Spillways and the sections of the discharge channels that cross along the hillsides above the tailings dams will be constructed in rock excavations. The sand dams will be protected against surface erosion by placing a layer or rock-fill on the sand and then covering this with till and soil. Flotation tailings will be dredged and discharged over the CIL tailings. The exposed flotation tailings will be covered with till and vegetated to control surface erosion. Riprap will be used in areas that will be inundated by the ponds.

SG/P may consider extending the erosion protection rock-fill layer that will be placed on the faces of the sand dams over the crest of the dams, and will need to develop a method to place and compact till on the exposed tailings beaches at closure.

The TMF North and South Cells will be closed primarily as dry covered surfaces, each with a small internal pond area and gravel-lined beaches and closure channels. A closure cover of stockpiled soil will be placed on the surfaces of the flotation tailings cells to promote re-vegetation. At closure, the CIL tailings will remain saturated to mitigate oxidation. The CIL tailing beaches will be dredged into the pond centre, and a layer of flotation tailings will be placed to form beaches and cover the CIL tailings and prevent re-suspension. The final CIL cell surface area will consist of reclaimed and revegetated beaches and a pond area to allow routing of surface runoff.

Surface water flows and flood flows from the reclaimed TMF will be routed from the closed cells via closure channels between cells into a spillway channel excavated in bedrock around the west abutment of the Southeast Dam. An auxiliary closure spillway around the North Dam is being considered as well in order to return post closure drainage patterns to as close as possible to pre-mining conditions. The cyclone dams and beach surfaces are net non-acid-generating due to the removal of sulphides. The dam faces will be first covered with an erosion protection layer of rock-fill quarried from the closure channel cuts. The dam surfaces will then be covered with till and re-vegetated using soil stockpiled during construction of the dams.

5.6 BOARD RESPONSE TO QUESTIONS

In considering the questions posed by SB/P regarding the TMF the Board members developed the following summary list of observations. While this list does not always provide a definitive response to the SG/P questions they reflect issues identified and provide a concluding statement. The issues identified are not considered to be insurmountable and require more detailed evaluation or design during future PFS and FS stages of the TMF design development.

5.6.1 Are the dams and major structures appropriately located?

In the opinion of the Board, the site that has been selected for the TMF has a combination of attributes that make the site particularly attractive compared to other sites that were considered. These attributes may be summarized as follows:

- The absence of salmon within footprint of development and Upper Treaty Creek makes it less sensitive to environment impacts from process affected water discharge;
- The site has favourable foundation conditions;
- The site has hydrodynamic containment on the flanks of the valley with groundwater discharge occurring at the downstream toes of the proposed facilities. Seepage collection dams are located sufficiently far downstream for secure hydrodynamic capture of any process affected seepage;
- The site has a relatively small headwater catchment area (55 km^2);
- The area experiences Low to Moderate seismicity; and
- The site has capacity of slightly more than 2.3 Bt which is sufficient for the identified ore reserve.

There are a number of characteristics that require additional attention during the detailed design of the TMF. These are:

- Possible paucity of low permeability construction materials (relative to requirements of dam sections);
- Full consideration of all the extreme climatic conditions: high rainfall, requirement for water discharge, high snowfall, seasonal construction;
- Accommodation of geohazards: avalanches and debris flows; and
- Steep side slopes that impact construction and reliable operation of diversion ditches.

Based on these attributes and characteristics the Board considers the site can be developed for the intended use.

5.6.2 Are dam sections, materials and construction methods and sequencing appropriate for the site and purpose?

The Board has the following observations and comments on the construction of the TMF dams and cells:

- Starter Dam sections
  - Homogeneous sections with till cores and internal drainage are suited to local earth-fill material types identified in the investigations. Work is needed to confirm adequate quantities of borrow for starter dam construction;
  - Rock-fill dam section may be an alternative for some or all of the starter dams. Evaluation of alternative dam sections is an opportunity that can be explored in the FS; for example, rock-fill north dams and earth-fill south dams;.
  - Cut-off and grout curtains require detailed evaluation and it is anticipated that the saddle dam will require a cut-off;
  - Starter dam heights may need to be increased, or geometry modified, to accommodate a possible sand shortfall; and
  - Additional runoff and sediment management controls may be needed for initial construction (initial 2 to 3 year period).
- Main Dam sections
  - Sand is an appropriate material and the technology is well understood at the proposed production rate;
  - There are several cyclone sand dams of this size and height that are successfully operating in other countries. As distinct from many of these, the KSM dams also incorporate a low permeability cores that further reduce the potential for seepage loss. There is also a similar dam operating in BC;
  - Measures to control erosion of the sand embankment will be required;
  - Board experience favours two stage cycloning at central cyclone stations;
  - Seepage evaluations should involve 3-D models considering surrounding permeable conditions which may modify under drain conditions;
  - Sand dam construction water controls should address maximum sand production rates at any one sand placement location;
  - Sand zones upstream of the core must be founded on firm ground conditions;
  - Initial sand requirement should be re-evaluated considering 6 month cycloning annually, a winter start up, providing water storage for 6 to 8 months without recycle and assuming a Process Plant ramp-up schedule;

- Design of the saddle dam over loose alluvium should consider a shear key as an alternative to soil excavation or buttress;
- Appropriate operating freeboard (typically of the order of 10 to 20 m) must be provided to accommodate design flood of critical duration and appropriate construction methods must be established and the stability verified;
- Tailings settled density should be re-evaluated. A density of 1.2 to 1.3 t/m^3 may be applicable for the first years of operation; and
- Beach slopes of 0.2 to 0.3% are typical for overflow deposition.

The Board concludes that the dam sections, materials and construction methods and sequencing are substantially appropriate for the referenced facilities.

5.6.3 What, in the opinion of the Board, are the greatest project design, construction and operating risks?

The Board has identified the following aspects for this stage of design. As the designs evolve these should be addressed to provide further assurance that risks are appropriately low for the nature and hazard potential of the TMF structures:

- Water balance analyses are needed to further understand system performance under likely ranges of climatic, operating and upset conditions.
  - Decoupling of the CIL cell from dam construction and other cells is desirable to ensure its development and use will meet project needs independently of the progress on the other tailings cells;
  - Sizing of starter facility should be further evaluated to establish:
    - Required storage capacity;
    - Geometry of the 3 dams to accommodate start-up during summer or winter;
  - On average water balance indicates water in impoundments will have a volume of 8.9 Mm^3. More detailed water balance analysis will likely show large variation in pond volume to large storms and extended wet periods;
  - Below the diversion channels all water management requires active pumping;
  - Planned discharge of water to Treaty Creek from approximately May to approximately October is an effective approach to limiting the TMF pond volumes, but requires effective and disciplined year on year water management in order not to accumulate excess quantities of water;
  - Water quality predictions are required to evaluate the potential requirement to treat for metals;
  - Sizing of sediment control structures is needed for capital construction;
  - Considerations for lining the CIL Residue Cell should include:
    - Underdrain/s to prevent liner blow-in;
    - An understanding that lining valley sides will be difficult;
  - Plant site operations need to be integrated into water management system;
  - Flood management between North Cell and CIL during 25 years of North Cell operation needs further development; and
  - Risk mitigation measures for the East valley landslide need further development.

The Board considers that the technical issues and cost of construction of a lined CIL Residue Cell, together with implementing underdrainage, and the preservation of the effectiveness of this underdrainage as the CIL Cell is raised during operations, offers the greatest potential for further reduction in the current design and construction risks related to the TMF. The control of geohazards, and

associated blockage of drainage, including drainage from the East valley, also present design, construction and operational opportunities to further reduce risks. Finally, the sources and adequacy of construction material need to be established. All three of these risks can be mitigated with additional investigation and design in the FS.

5.6.4 Are the facilities designed to operate effectively?

The Board's responses are as follows:

- The facility layout has been effectively developed to accommodate site conditions;
- The technology of sand dam construction is well understood;
- Slurry tailings transport and discharge can be operated with little operator intervention for extended periods of inclement weather. This allows operations to continue through winter storms and white-out conditions and requires adequate remote monitoring systems;
- Climate requires that cyclone operation and embankment construction is maximized during summer;
- Diversion channel, pipelines and tunnel will require ongoing maintenance;
- Winter storm conditions resulting in delayed repairs and interruption of operations need to be accommodated; and
- Detailed planning of dam construction and tailings deposition sequencing will be required.

Overall, the Board is of the opinion that the facilities are designed to operate effectively. There are a number of operational procedures that have yet to be developed. The operational requirements of the facilities are relatively complex. This complexity is required to appropriately address climatic conditions, topography, natural hazards, available and economic construction materials, geochemical characteristics of the tailings and tailings water quality. Further consideration needs to be given to the design and construction of the CIL Residue Cell and the recommended optimizations in this report.

5.6.5 Are the facilities designed to be safe?

- The design criteria include the PMF and 10,000 year earthquake;
- Foundation conditions for dam construction are reasonable;
- Operations must focus on worker safety considering the geohazards and climatic conditions;
- Maintenance of diversion channels will be important;
- Dependence on actively managed water control systems includes pumps and pipelines and back-up power systems are necessary; and
- Discharge of water is permissible and necessary to avoid ongoing water accumulation in TMF

The Board concludes that designs for the operational period of the mine are to standards generally accepted in the industry. Site terrain, weather, geohazards and requirement for long-term maintenance of some of the structures require further design and attention to operational methods and consequence assessments that should be addressed in the FS.

5.7 RECOMMENDATIONS FOR FOLLOW-UP DESIGN

The anticipated next steps in future developing of the designs include updating the current PFS and then proceeding with a FS design, followed by a Detailed (for construction) Design. The Board's recommendations for the PFS and FS phases are summarized below.

5.7.1 Pre-Feasibility Study

- Update the initial sand dam construction sequence and staged dam heights considering recommended dam design changes and deposited tailings densities, season of start-up and likely range of impounded water volumes, and considering the PMF of critical duration for flood storage requirements. Determine the need for any additional burrow source materials and for a water storage pond outside the footprint of the TMF; and
- Update of CIL Residue Cell liner system design considering the need for the underdrain to be effective up the sides of the cell for the operating life of this cell and the potential need to increase the underdrain dewatering system capacity.

5.7.2 Feasibility Study

For the feasibility study, additional site investigation will be required to more accurately define the characteristics, quantity and location of construction materials. Additional investigations will be required at the North Seepage Collection Dam and Saddle Seepage Collection Dam locations to allow the design of the seepage control measures (slurry cut-off trench and grouting) to be advanced, and to select the location and develop designs for any additional TMF water storage facility that may be required. A trade-off study may be considered for the section to be used for the starter dams for the North Dam, Splitter Dam and Saddle Dam. Rock-fill, if available near the dam sites, may allow a longer construction window than a section incorporating large quantities of till. Rock-fill can be placed during wet weather and in freezing conditions. More detailed construction material and water balance analyses will be required to refine the analyses completed for the PFS described in Section 5.7.1 above.