For the perimeter other than the toe of the dikes, it is possible that some tailings are present in some locations above or near the NWL. In these areas, tailings will be pushed, by bulldozer or backhoe, to below 369.2 m asl to provide at least 1 m water cover.

For the perimeter areas below the West Twin and Test Cell Dikes, tailings are exposed substantially above the NWL. In the case of the Test Cell Dike area, tailings rise to an elevation of approximately 371 m asl, the nominal elevation of the tailings beach upon which the Test Cell Dike is seated. In the case of the West Twin Dike, tailings are present in a relatively flat-sloped beach that extends from the pond edge to the toe of the dike, a distance of approximately 150 m. Within these two areas, it is estimated that there are more than 100,000 $m^3$ of tailings above the NWL in the Reservoir. It is considered impractical to remove all of these materials due to the potential long term effects on dike stability if ponded water (and resultant heat source) were allowed at the toe of the dikes.

In the long term, permafrost aggradation is anticipated to progress towards the perimeter of the Reservoir pond to the point where permafrost forms around the pond shoreline proximal or below the pond water level. This phenomenon is commonly observed in natural conditions in the north where a lake has a shallow sloping shoreline (MacKay 1962). Permafrost can form to a shoreline depth under water equivalent to approximately 2/3 of the typical thickness of ice. In the summer thaw season, the shoreline at the waters edge does not thaw and the lake bottom sediments may not thaw until a depth greater than 2/3 of the typical ice thickness is reached. Should this occur around the perimeter of the Reservoir Pond, as is expected, littoral tailings will be subject to permafrost aggradation. However, given that the timeframe for this phenomena to occur is unknown, reclamation efforts are required to protect against oxidation of tailings and transport of contaminants during the interim period, as described below.

### 6.2.2 Design Analyses

#### 6.2.2.1 Reservoir Water Levels in Closure

The estimates of the maximum water level (MWL) and low water level (LWL) made in the previous studies (Golder, 2002) were updated to reflect the proposed grading plans and water levels in the Surface Cell and the Reservoir. The updated water levels as derived from Golder (2004 b,c,d) are summarized in Table 8. The following observations could be made:

- The seasonal water level fluctuations in the Reservoir (West Twin Lake) will be limited to a few centimetres because of the small contributing drainage areas, low precipitation, and large spillways
- The peak water level corresponding to the PMP storm event was estimated to 370.8 m (i.e. a flow depth of 0.6 m on the outflow weir structure invert).
- The water level decline under the drought conditions had been evaluated in the previous study. The maximum water decline under extremely dry conditions was estimated to be 200 mm.

In summary, the extreme range of water level fluctuation in the Reservoir is estimated at 0.8 m, from 370.0 m (LWL) under extreme drought conditions, to 370.8 m (MWL) under PMP storm event conditions.

#### 6.2.2.2 Wave Effects

During the open water season, which lasts approximately four months per year at Nanisivik, wind action may cause waves on the water surface. Closure planning measures involved an estimation of the significant wave heights possible, and an assessment of the possible effects on the tailings and shoreline erosion protection design.

Because of the limited size of the Reservoir, the estimated significant wave height is only 0.4 m. Since the 1.0 m water cover provided is 2.5 times deeper than this wave height, tailings re-suspension is not expected to be problematic. In fact, a review of the water quality data submitted as part of the SNP portion of the License during operations, indicates TSS values and metal concentrations were well within the discharge criteria (despite the fact that 1.0 m of water cover was not maintained constantly during that period).

The available wind data were summarized, the design wind conditions selected, and the wave height calculated are detailed in the following paragraphs:

Design Wind Speed

Two sources of wind data were used: the site data, collected at the Nanisivik Mine meteorological station; and the regional data, collected at the Pond Inlet Airport meteorological station.

The on-site wind record is available for four years: 1993, 1995, 1996 and 1997. The data shows the highest observed wind velocity was 91 km/h. Because of the short period of record, the frequency of occurrence (or return period) cannot be assigned to this observed value.

To gain better insight into the frequency distribution of the wind speed data, the regional, long-term records at the Pond Inlet Airport were used. Pond Inlet is located approximately 220 km east from the Nanisivik Mine. The wind speed record at the Pond Inlet Airport includes 28 years of observations, from 1976 through 2003. The extreme wind speed statistics at the Pond Inlet Airport were provided by Environment Canada. The 100-yr return wind speed at the Pond Inlet Airport is estimated to be 95.6 km/h (with a 95% confidence interval from 86.1 km/hr to 105.2 km/h).

The maximum wind speed observed at the Nanisivik Mine (91 km/hr) is consistent with the estimated maximum wind velocity at the Pond Inlet Airport (95.6 km/hr).

The definition of the design wind speed for erosion protection normally involves two steps:

- Selection of the wind speed value based on the in-land meteorological records.
- Application of a correction factor, which accounts for the fact that the wind speed above the water surface is higher than that over the land surface.

For the purpose of conservative design, the upper limit of the 95% confidence interval of a 100-yr return wind speed (105.2 km/h) was used in the design. Because of the limited fetch on the Reservoir, the correction factor is small (i.e. 1.07). The correction results in a design wind speed on 112.6 km/h.

Design Wave Height

The wave height can be calculated using the following equation (Smith, 1995):

$$h_w = 0.00513\, V^{1.06} F_e^{0.47},$$

where:
$h_w$ is the wave height in meters,
V is the design speed in km/h (112.6 km/hr in this case),
$F_e$ is the effective fetch in km (0.3 km in this case).

Using this equation, the design wave height in the Reservoir (West Twin Lake) was calculated to be 0.4 m (Table 9).

### 6.2.3 Design Considerations

As shown on Figure 21, erosion protection will be provided around the perimeter of the Reservoir, wherever there is a potential for erosion of existing tailings. Erosion protection will not be provided on natural ground located along the new shoreline.

Erosion protection on remnant tailings will extend between a top elevation of 370.8 m and a bottom elevation of 369.2 m. The rationale for these elevations is as follows:

- The top elevation is equal to the MWL under PMP storm conditions (i.e. Elev. 370.8 m).
- The top elevation is equal to the NWL (370.2 m) plus 1.5 times the design wave height.
- The bottom elevation is equal to the LWL (i.e. 370.0 m predicted under extreme drought conditions) minus 1.0 m (to maintain 1 m of water cover over exposed tailings).

As a result, rip rap protection will be provided between elevation 369.4 and 370.8 m, a slope distance of 1.4 m.

### 6.2.3.1 Rip Rap Sizing

Rip rap is the most common means of protecting slopes against wave action. The required median rip rap size was calculated based on the wave height ($h_w$) and the bank slope. The median stone size is given by the following equation:

$$D_{50} = 377\ h_w / \sin(70-\alpha)$$

where:

$D_{50}$ = median stone size (mm)
$h_w$ is the wave height (m), and
$\alpha$ is the angle of the slope to be protected (degrees).

The required median stone size was calculated using parameters included in Table 9 for a slope of 4H:1V. Based on the analysis, it is recommended that rip rap with median size of 200 mm be used at the toe of the shoreline erosion protection. The recommended gradation specifications for the rip rap are included in Table 10.

Rip rap will not be provided on natural shoreline. Potential erosion of these areas would not result in the transport of tailings out of the Reservoir. If excessive erosion of the natural shoreline area is observed during routine surveillance, appropriate maintenance will be undertaken to repair the locally affected area.

### 6.2.3.2 Protection of Shoreline Against Moving Ice Masses

It is understood that the ice cover on the water bodies at Nanisivik can reach a thickness of approximately 2 m. During the ice break period, the ice cover can move and potentially exert forces on the shoreline causing erosion.

The following considerations apply to the ice erosion:

- In theory, the dynamic forces that could potentially be exerted by the free masses of ice are very high. Erosion protection against such theoretical forces is not practical, given the sedimentary bedrock units located at Nanisivik Mine.
- The movement of ice masses in the Reservoir will be constrained by the following structures that will remain after closure:
  - the remnant Test Cell Dike;
  - the baffle dike (lowered);
  - the access road causeway (lowered); and
  - the West Twin outlet channel and overflow weir.
- Due to the limited size of the Reservoir, the extent of ice cover movement and the impact on the shoreline within the Reservoir will be restricted. Accordingly, it is anticipated that the erosion by ice action will be limited.

In summary, the erosion and associated environmental impact caused by the ice impact are expected to be limited, but the cost of protecting the shoreline against it would be prohibitive. It is recommended, therefore, that the Reservoir shoreline erosion protection not be designed specifically to withstand the ice impact. Some later maintenance may be required to repair impacted rip rap. Additionally, water quality data collected during mine operations as part of the Surveillance Network Program (SNP) portion of the License, indicates TSS values and metal concentrations were well within the discharge criteria. Since the tailings were not protected from ice effects during this time, it is not anticipated that any negative impacts on water quality will result from tailings re-suspension related to ice effects.

#### 6.2.3.3 Prevention of Ice-entrained Tailings Transport

Because of the winter ice cover on the Reservoir may be approximately 2 m thick, it is possible the tailings may become frozen onto the base of the ice pan around the perimeter, and in shallow area of the Reservoir. Upon ice break-up in spring, these tailings could potentially be transported by moving ice. Ice movement will be restricted however because the baffle dike and causeway will be left in place with a lowered crest elevation 369.7 m (0.5 m below the NWL). The Outlet Control Embankment and spillway will also inhibit the movement of ice out of the Reservoir. It is therefore expected that the ice will melt out within the Reservoir, dropping any adfrozen tailings back into the pond.

Records of water quality at the outlet structure (Station 159-4) indicate that the total suspended solids values were low during the period of active mine operation, in spite of active tailings deposition. This suggests that the internal structures (i.e. the baffle dike, access road causeway and Outlet Control Structure) were effective at preventing off-site transport of tailings with ice.

### 6.2.4 Design of Shoreline Erosion Protection

#### 6.2.4.1 Design Elements

The erosion protection details are shown on Figure 21.

Three typical sections have been developed corresponding to the various zones:

- Section A illustrate the tailings relocation along the original West Twin Lake shoreline where a thin layer of tailings is present on the relatively steep bank. These tailings will be excavated and re-located to below Elev. 369.2 m to meet the 1.0 m minimum water cover requirement. The remaining bank will consist of native soils.
- Section B is for use in the area extending away from the toe of West Twin Dike which consists of a relatively shallow sloping tailings beach. The design involves covering the tailings with the thermal cover to Elev. 369.2 m (provides a miimum water cover of 1 m at that point) and providing erosion protection below the toe of the thermal cover to match the subgrade.

- Section C is for use on the tailings beyond the toe of the Test Cell Dike. The existing tailings in this area are relatively steeply sloping at approximately 12-15%. Again the thermal cover will be placed down to elevation 369.2 m and erosion protection will be placed below the toe.

The sections were developed to minimize tailings relocation and allow for construction on potentially saturated tailings beaches.

A bedding layer will be provided to prevent waves from "plucking" fine materials from the thermal cover beneath the rip rap. Table 10 provides gradation specifications for the bedding layer (Type 2) material.

#### 6.2.5 Construction Considerations

To reduce difficulties associated with the requirement for traffic of construction equipment on wet tailings beaches, the water level will be lowered to 368 m prior to construction. This will allow placement of the shale cover to beach elevation 369.2 m (and slightly lower at the toe of the cover). Where the tailings are saturated and soft, the 1 m thick shale layer may have to be pushed in a single lift. By over-building and then trimming the shale, traffic directly on tailings can be avoided. The erosion protection can then be placed with the equipment sitting on the shale layer.

### 6.3 Dike Stability Considerations

#### 6.3.1 Design Criteria

The West Twin Dike will remain in place during closure for permanent retention of Surface Cell tailings. The dike currently exists at a slope (3.7H:1V) shallower than its previously approved closure design and will be covered with an armouring layer of Twin Lakes sand and gravel to prevent erosion. The Test Cell Dike will be partially removed and regraded during closure. The remaining portion of the dike will merge into the shoreline reclamation cover. While this remnant dike will not retain water during closure, it is still considered important to assess the long term stability of the remnant dike and shoreline reclamation cover.

The closure design of the dikes must exhibit satisfactory stability in the short and the long term. To address this, several analyses were completed to assess the current and long term stability of the dike.

#### 6.3.2 West Twin Dike Stability Analyses

The stability analyses completed for the West Twin Dike are reviewed in detail in Appendix V. The stability of the West Twin Dike was reviewed considering static and pseudo-static (seismic loading) conditions. The stability was assessed using two methods; one considering the dike to act as a rigid block due to its frozen state with thawed tailings and water applying pressure on its upstream face; and the second considering the dike to contain a thawed zone at depth that exhibits artesian pore pressures. The results of the analyses indicate that the dike exhibits acceptable stability considering assumed strength parameters and applying static and pseudo-static loading conditions.

### 6.3.3 Test Cell Dike Stability Analyses

The stability analyses completed for the Test Cell Dike is reviewed in detail in Appendix V. The stability of the Test Cell Dike and associated sloping reclamation cover were assessed considering static and pseudo-static conditions. The stability was assessed assuming piezometric pressures in the slope equivalent to the water level in the adjacent Reservoir. In addition, the downstream slope of the cover was assumed to be 3H:1V. The results of the analyses indicate that the dike and associated sloping cover exhibit acceptable stability considering assumed strength parameters and applying static and pseudo-static loading conditions. Slope angles flatter than 3:1, such as 4:1, would be considered adequate as well.

### 6.3.4 Liquefaction

Although the stability analyses consider pseudo-static forces that may be applied to the dike during a seismic event, the analyses do not consider the possible liquefaction of the tailings. For liquefaction to occur, the tailings material needs to be both loose and saturated. Hence, for all areas where the tailings are frozen, liquefaction is not a concern. In the Surface Cell, if the tailings in the talik were to liquefy, the pore pressures could be transferred onto the West Twin Dike structure. As shown in the detailed analysis, very high pore pressures are required to reduce the Factor of Safety to unity for this dike.

The potential consequences of a liquefaction event also needs to be considered. To review, no water will be retained by the West Twin Dike during closure. Liquefaction of the tailings may result in disturbance of the surface cover, which would have to be repaired. Pore pressures generated during liquefaction may also result in deformation of the West Twin Dike, and worst case scenario, the release of some tailings and talik pore water into the Reservoir. Remedial work would then be required to handle and treat the pond water and to clean up any released tailings.

Considering the case of the Test Cell Dike and associated sloping reclamation cover, it is important to note that the dike is currently underlain by 14 to 18 m of frozen tailings. Hence, liquefaction would only be a concern for thawed tailings located beneath this frozen zone beneath the dike. Liquefaction would likely result in some settlement of the dike, with resulting disturbance to surface features such as design grade and rip rap placement. But again, no water will be retained by this structure during closure, so catastrophic release of surface water is not a consideration.

Additionally, liquefaction will only be a concern when the tailings are thawed. Hence, as permafrost aggradation occurs into the tailings over the next 30 years, the liquefaction concern will be reduced and then eliminated once the tailings are completely frozen.

#### 6.3.5 Closure Contouring of West Twin Dike

The West Twin Dike is currently graded to an overall slope of 3.7H:1V. The shale will be covered with an armouring layer of 0.25 m of Twin Lakes sand and gravel. One portion of the downstream slope of the dike still need to be covered with shale. The required volumes of shale and armouring sand and gravel is summarized in Table 7.

### 6.4 West Twin Dike Spillway Design

The main purpose of the spillway is to provide passive drainage of runoff out of the Surface Cell and around the West Twin Dike to prevent excessive ponding and a potential heat source to the underlying tailings. The spillway will be an open channel, generally excavated into bedrock. The details of this design are provided in the West Twin Dike Spillway Design report (Water License requirement Part G, Item 7) provided under a separate cover.

#### 6.4.1 Design Criteria

The spillway has been designed to safely convey flows from the routed 24-hour probable maximum precipitation (PMP) storm event. The inlet invert elevation will be 384.0 m. Flows will report to the Reservoir, which will have normal water level 370.2 m.

To prevent scour of tailings in the Reservoir, energy dissipation will be provided within the spillway outlet structure.

The spillway has been designed to require minimal maintenance for mine closure. Routine surveillance and maintenance requirements are set out in Section 6.4.4.

### 6.4.2 Design Considerations

The estimated PMP event is 140 mm in 24 hours (Section 2.6). Under this event, the estimated (routed) peak inflow to the spillway would be approximately 5.2 $m^3/s$, corresponding to a water level of 384.6 m in the Surface Cell. For sloping sections of the spillway, the flow depth would be less than 0.6 m (i.e. 0.52 m for segments sloping at 1% or 0.31 m depth for segments sloping at 5%). A protected flow depth of 0.6 m minimum is provided throughout the spillway.

The spillway alignment was selected to situate the base within intact bedrock while considering the required excavation volumes. The geologic conditions were inferred from the borehole and test pits completed in the spillway area by BGC and Nanisivik Mine (Golder 2004a). In general, the stratigraphy in the area of the spillway is as follows:

- Till, overlying;
- Frost shattered bedrock, overlying;
- Competent bedrock.

Ground ice was observed in boreholes near the Reservoir shoreline (i.e. BH1 and BGC03-06).

Where the design flow depth is not entirely within intact bedrock, erosion protection will be provided. Based on flow velocity calculations, it is recommended that segments of the spillway sloping at more than 1% gradient will require rip rap with a median stone size ($D_{50}$) of 300 mm. For flatter segments (sloping at up to 1%), the required median stone size was estimated to be 100 mm. A bedding layer will be provided below the rip rap and a granular filter (Type 3) will be placed on overburden as required.

A service road will be provided along the full length of the spillway to allow for periodic inspection and maintenance.

### 6.4.3 Design Details

The spillway design is presented on Figure 22.

The spillway will consist of a 6 m wide open channel approximately 565 m in length. The vertical alignment was selected to minimize excavation, while still situating the invert of the channel within intact bedrock to the extent possible. The base width selected is a reasonable minimum to allow for excavation considering that blasting will likely be required.

The spillway is comprised of three segments (Section A-A', Figure 22):

- The **inlet portion** of the spillway (about 120 m long) will be flat (slope of 0%) with a nominal invert of Elev. 384.0 m. The spillway inlet will be constructed to match the Surface Cell cover contours and rip rap protection will be provided.

- The **chute portion** will comprise a segment about 130 m long with 1% slope followed by a length of approximately 270 m at 5% slope.
- The **outlet portion** will consist of a plunge pool (to dissipate flow energy) and an associated outflow channel. To facilitate drainage, the plunge pool will have a floor level of Elev. 370.6 m, or 0.4 m above the normal water level in the Reservoir. An outlet channel sloping at about 1% will discharge flows to the Reservoir at Elev. 370.2 m.

A typical section of the spillway is shown on Figure 22 (Section B-B'). Excavated slopes in intact bedrock will be approximately 0.1H:1.0V. In frost shattered bedrock, flatter slopes of 1H:1V will be built. Overburden slopes will be cut at 4H:1V vertical for long term stability considerations. No erosion protection is required on these cuts slopes, except within the 0.6 m flow depth.

Side berms will be constructed to crest elevation 370.8 m for the outlet channel near the Reservoir. This is equivalent to 1.5 times the design wave height above the normal water level in the Reservoir. It is also above the maximum water level in the Reservoir predicted under extreme flood conditions. The approximate extent of rip rap protection is shown on Figure 22.

### 6.4.4 Construction Considerations

#### 6.4.4.1 Diversion of Upstream Watershed

The spillway is located in a valley where ephemeral flows are produced from the upstream watershed. To prevent erosion of the spillway side slopes, the flows will be directed to an inlet channel where erosion protection will be provided (Figure 22). A series of small culverts will be provided under the service road to prevent icing and subsequent loss of trafficability.

A small deflection berm will be provided at the crest of the cut slope on the uphill (south and west) side of the spillway to prevent run-off from eroding the slope. No ditching or berming is required on the opposite side of the spillway because the collecting watershed is not large.

#### 6.4.4.2 Excavation, Erosion Protection and Alignment

Where the design flow depth of 0.6 m is not entirely within intact bedrock, erosion protection (consisting of rip rap placed on a layer of bedding material) will be provided. Rip rap slopes within the flow depth should be 4H:1V. If overburden (soil) is exposed within the flow depth, a sand and gravel filter should be placed beneath the rip rap bedding to prevent the migration of fine materials through the bedding and rip rap.

There is some chance that ground ice may be encountered during the construction of the outflow channel. If so, the ground ice will be sub-excavated and the void will be backfilled with granular fill covered with bedding and rip rap. Alternatively, the alignment could be adjusted.

If poor rock conditions are identified in the field, the slopes will be flattened during excavation to ensure long term stability.

## 6.5 West Twin Lake Outlet Channel

For closure, the existing control structure at the outlet of the Polishing Pond will be removed. It will be replaced with an open channel and overflow weir structure which will passively control the Reservoir water level and minimize future maintenance requirements.

The channel will have a base width of 7 m to match the nominal width of Twin Lakes Creek receiving flows downstream. To create the channel, the existing outlet control structure will be removed and the opening in the existing embankment will be lowered and widened. A 0.3 m wide reinforced concrete cut-off wall with invert Elev. 370.2 m will be provided in the channel to control the normal water level in the Reservoir (West Twin Lake). Erosion protection will be provided in the channel upstream and downstream of the wall to prevent scour of the natural or fill materials adjacent to the channel.

Flood routing analysis indicated that the flow depth in the design storm would be 0.6 m. Consequently, the design maximum water level (MWL) in the Reservoir will be Elev. 370.8 m.

### 6.5.1 Design Criteria

The outlet control structure has been deigned to safely convey flows from the routed 24-hour probable maximum precipitation (PMP) storm event.

The normal water level (NWL) in the Reservoir will be controlled at 370.2 m. The MWL will be 370.8 m.

Erosion protection will be provided around the Reservoir where required to prevent the erosion of tailings. The top elevation of the erosion protection will be the NWL plus 1.5 times the significant wave height generated by a 100-year return wind velocity. Table 9 notes that the design wave height is 0.4 m so the design criteria would require erosion protection to an elevation of 370.8 m.

The outlet has been designed to require minimal maintenance. Routine surveillance and maintenance are required as set out in Section 6.5.6.

### 6.5.2 Design Considerations

#### 6.5.2.1 Hydraulic Considerations

The GAWSER model (Guelph All-Weather Sequential Event Runoff model) was used for the hydrological simulation of the Nanisivik Mine tailings system. This model is widely used in Canada for various types of hydrological analyses. The following design conditions were used for the hydrological modelling:

- The Surface Cell watershed area is 127 ha. The Reservoir watershed area is 173 ha (excluding the Surface Cell watershed). The total watershed area of the outflow control structure is 300 ha (127 ha plus 173 ha).
- The elevation-discharge relationship for the Surface Cell was developed from the hydraulic calculations of flow in the Surface Cell spillway inlet. Hydraulic roughness (Manning's) coefficient of 0.035 for the spillway channel was assumed.
- The elevation-discharge relationship for the Reservoir was developed from the hydraulic calculations of flow for the West Twin Lake (Reservoir) outlet control spillway, also using a Manning's roughness of 0.035.
- The elevation-storage relationships were developed based on the Surface Cell grading plan for closure and the Reservoir grading plan for closure.
- The watershed properties were set to reflect the frozen ground conditions and simulate high surface runoff. The resulting runoff coefficient for the PMP event was 0.94.

From a hydraulic perspective, both spillway inlets will act as a broad crested weirs.

#### 6.5.2.2 Design Precipitation Event

The extreme precipitation conditions at Nanisivik as derived from Golder (2004 b,c,d) are summarized as follows:

- The extreme daily rainfall and snowmelt amounts at Nanisivik are comparable. The daily rainfall PMP event is estimated to be approximately 140 mm. The daily snowmelt PMP event is estimated to be 155 mm. For comparison, a daily PMP event in Northern Ontario is approximately 500 mm to 700 mm.
- The amount of rainfall and snowmelt at the Nanisivik Mine is small. Therefore, a comparatively small spillway is required at the Nanisivik Mine, even to convey extreme floods.

For the mine closure, the Reservoir outflow channel was designed to convey a flood resulting from the PMP storm event (140 mm in 24 hours). The snowmelt distribution is more uniform than the rainfall distribution. Consequently, the PMP rain storm may produce greater peak flows than the snowmelt event. Hydraulic characteristics corresponding to a 100-year return period are also provided in the report for the purpose of comparison.

#### 6.5.2.3 Design Flow Conditions

The following hydraulic conditions will be observed at the West Twin Outlet Channel under the PMP conditions (Table 11):

- The calculated peak discharge from the Reservoir is approximately 6.5 $m^3/s$;
- The calculated peak water depth at the Reservoir Outlet is approximately 0.6 m;. and
- The calculated peak velocity at the Reservoir Outflow Structure is 1.5 m/s.

For comparison, under the 100-yr storm conditions (Table 11):

- The calculated peak discharge over the outflow structure is estimated to be 1.7 $m^3/s$;
- The calculated peak water depth is estimated to be approximately 0.2 m; and
- The calculated peak flow velocity is estimated to be approximately 1.5 m/s.

#### 6.5.2.4 Erosion Protection

The erosion protection against the flowing water was sized based on the calculated peak flow velocity and channel side slopes (Smith, 1995). As mentioned previously, the calculated peak velocity at the West Twin Outlet Channel is 1.5 m/s. To protect against these flows, the median rip rap ($D_{50}$) required would be less than 130 mm. For continuity with proximal toe protection, it is recommended that the channel be protected from erosion with rip rap having an average diameter $D_{50}$ of 200 mm. The material specifications for the rip rap are included in Table 10.

### 6.5.3 Geotechnical Considerations

#### 6.5.3.1 Subsurface Conditions

Information regarding the subsurface conditions at the outlet area was obtained from:

- The original design drawings for the existing Reservoir outlet structure (Kilborn, 1977) indicate bedrock at about Elev. 369.4 m;
- Borehole BGC03-23, located about 25 m from the spillway alignment, in which bedrock was encountered at Elev. 368.8 m, underlying sand and gravel sized till (inferred to be embankment fill based on the Kilborn drawing); and
- Borehole BGC03-24, located about 100 m southeast of BGC03-23. In this borehole, bedrock was encountered at Elev. 367.7 m. The upper 0.6 m of the bedrock is noted as being highly fractured.

Borehole logs for BGC03-23 and BGC03-24 are included in Appendix VI.

#### 6.5.3.2 Configuration of Existing Outlet Structure

The existing outlet structure (Figure 23) consists of a weir and an embankment which together close the former creek section. Based on the available subsurface data, it is inferred that the existing concrete control structure is founded on a bedrock foundation on the original shoreline (which forms the left abutment of the West Twin Outlet Control Embankment).

The new outlet structure will be formed by excavating into natural ground at the north side of the existing outlet location to take advantage of the natural ground and presumably higher bedrock elevation. For closure, it is generally preferable to excavate hydraulic channels into natural ground rather than embankment fill.

Construction details of the existing West Twin Outlet Control Embankment are not certain. As-built drawings show an upstream membrane in the Diversion Dike (Kilborn, 1977) but it is not clear if this was extended into the Outlet Control Embankment (Figure 23). After construction, a cut-off trench was excavated on the upstream face and backfilled with marine clay, presumably to reduce seepage. Since that time, any potential seepage amount has been low enough to maintain a water level of approximately 371.2 to 371.5 m in the polishing pond (or about 2 m of head across the structure). In addition, visual inspections since 1998 have not identified and seepage on the downstream side of the embankment. Lowering the water level to 370.2 m at closure will further reduce the potential for seepage. Based on anecdotal information from mine site staff, no water would be impinged on the embankment at a water level of 370.2 m. Therefore, no additional seepage mitigation measures are proposed for the existing embankment. Bedrock below the overflow weir will be sealed and the concrete wall will tie into the existing embankment material.

### 6.5.4 Design Details

The design of the West Twin Outlet Channel is shown on Figure 24. The design incorporates the following elements:

- A 0.3 m wide reinforced concrete cutoff wall will provide a fixed non-erodible invert and prevent seepage through the rip rap and bedding zones at the sides of the channel. Below Elev. 369.0 m. To limit seepage under the wall, the bedrock foundation for the control structure will be grouted. Except under extreme low precipitation conditions, this will prevent the Reservoir water level from dropping below the NWL. As a preliminary basis, the grouting would extend to a depth of 1.5 m below the wall founding level. As a contingency, the depth of grouting could be increased in localized areas, if required.
- The wall will extend 5 m laterally into the adjacent embankment fill and into the natural ground abutment to cut-off the rip rap and bedding layers and increase the seepage flow path around the ends of the wall.
- Upstream of the concrete cut-off wall, rip rap erosion protection will be provided on the side slopes up to Elev. 370.8 m. This is 0.6 m above the invert, which corresponds to the

NWL in the Reservoir plus 1.5 times the significant wave height. The inlet channel flares to meet the Reservoir discharge flow.

- Erosion protection requirements are set out on Table 12. Downstream of the wall, rip rap erosion protection will be provided to 0.3 m above the channel floor to protect up to the flow depth calculated under PMP storm conditions. To prevent possible scour of the natural (unprotected) creek bed, the sloping outlet channel will terminate in a 3 m long stilling basin with elevation 369.0 m (approx.) to force a hydraulic jump on the constructed erosion protection. These elevations may be adjusted following the required survey of the creek.
- Rip rap will be placed against the wall to smooth the flow lines and to protect the concrete surfaces that would otherwise be exposed to the elements.

#### 6.5.5 Construction Considerations

Construction quantities are summarized in Table 13.

The West Twin Outlet Channel has been located based on limited available information. The detailed location and construction will be verified through a test pitting investigation prior to construction. Field adjustments could be made to the position based on the bedrock foundation conditions encountered (including the depth to and extent of frost shattering of the bedrock).

The concrete cut-off wall will be founded on competent bedrock. Excavation to the bedrock surface is expected to be required to provide a suitable foundation. Based on the available borehole information (Appendix VI), it is anticipated that this excavation will extend down to approximate elevation 368 to 369 m. Deeper local excavation may be required. The wall foundation level should be approved in the field by the Field Representative.

It has been assumed that the water level in the Reservoir/Polishing Pond will be drawn down to Elev. 368 m at the time of construction. Normal local dewatering measures may be required in the excavation for the concrete wall.

Bedrock foundation grouting will extend to a depth of about 1.5 m below the wall prior to pouring the concrete. This will locally tighten the bedrock and reduce seepage through the near surface zone. The actual depth of the grouting will be adjusted based on observations and response tests done in the grout holes during drilling.

The cut-off wall will be doweled into bedrock. The actual depth of the dowels will be based on the conditions encountered in the field and approved by the Field Representative. In general, dowels will extend a minimum of 0.5 m into intact bedrock (i.e. non frost shattered).

For the efficient hydraulic operation of the West Twin Outlet Channel, it is important that the creek downstream of the structure flows relatively freely under flood conditions. A pre-construction survey of Twin Lakes Creek is required to verify this assumption.

#### 6.5.6 Maintenance

Visual observation of the Outlet Control Structure should be included as part of the routine surveillance program. Surveillance items should include inspection for the presence of upstream or downstream blockages and the condition of the concrete wall and rip rap.

### 6.6 East Twin Diversion Channel Upgrade

When the mine was constructed, the flow from East Twin Lake was diverted from its natural course, which became part of the Polishing Pond. A Diversion Dike was constructed to redirect the flow through a constructed Diversion Channel (Figure 25). Based on hydrologic considerations (Golder, 2004 b,c,d), it was determined that the diversion should be maintained after closure.

There is evidence of erosion on the upstream face of the East Twin Lake Diversion Dike and channel in the area indicated on Figure 25. Should the dam be eroded away, runoff from the large East Twin Lake watershed would report to the Reservoir, potentially eroding tailings or compromising the integrity of West Twin Dike and/or the West Twin Outlet Channel.

There is also evidence of erosion within the channel itself. Such erosion is of little potential consequence, except for sediment discharge to the channel.

#### 6.6.1 Design Criteria

Measures are required to protect the East Twin Diversion Dike against erosion under flows from the PMP storm event.

It is not practical or necessary to line the entire diversion channel with erosion protection because the consequences of erosion are not severe. Any channel erosion would not be classified as a catastrophic consequence.

#### 6.6.2 Design Considerations

The East Twin Lake drainage area (34.6 $km^2$) is much greater than that of the Reservoir (West Twin Lake) drainage area (3 $km^2$). The diversion will remain in place for closure. Erosion noted in the channel is not of consequence, but the dam itself will be upgraded.

The peak flows from the East Twin Lake watershed were evaluated in the hydrological study (Golder 2004 b,c,d). The peak flows from the East Twin Lake watershed were estimated to be