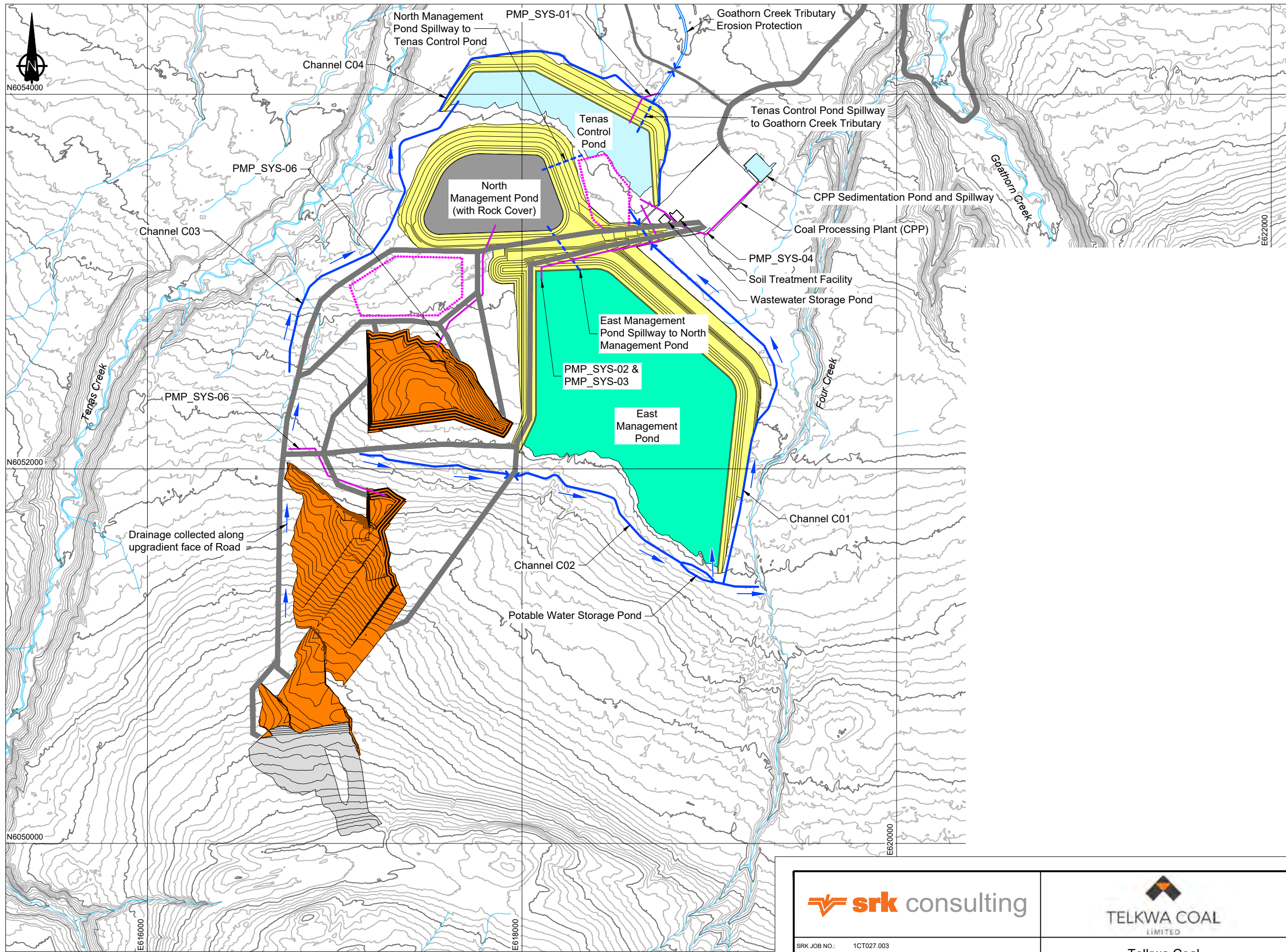


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LEGEND

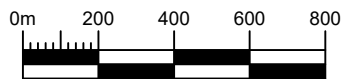
- Water Management Feature
- Water Flow Direction
- Spillway
- Culvert
- Existing Watercourse
- Pipeline
- Proposed Road
- Dam and Buttress
- Pit Excavation
- Sedimentation and Control Pond
- Active Management Pond
- Backfill / NAG Pile
- Rock Cover
- Topsoil Stockpile

NOTES

- Contours shown at 5.0m intervals.
- All dimensions shown in meters unless otherwise stated.
- East Management Pond is active. All PAG rock and processed rock to be placed in East Management Pond.
- Non-PAG rock and overburden cover placed across final PAG rock surface in North Management Pond. Final surface is vegetated and spillway upgraded to closure design.
- Non-PAG rock placed as backfill in open pit and as buttresses along Management Pond dams.
- All runoff collected in open pit areas pumped to East Management Pond.
- Long-term overflow weir from Channel C02 to East Management Pond is constructed, replacing the overflow weir to Channel C01.
- PMP_SYS-05 to be installed at Explosive Magazine, Explosive Facility and Rail Infrastructure Sedimentation Pond to discharge nominal flows up to 1 in 10-year event to downstream environment.

REFERENCE

NAD83 UTM Zone 9.



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Telkwa Coal

Environmental Assessment

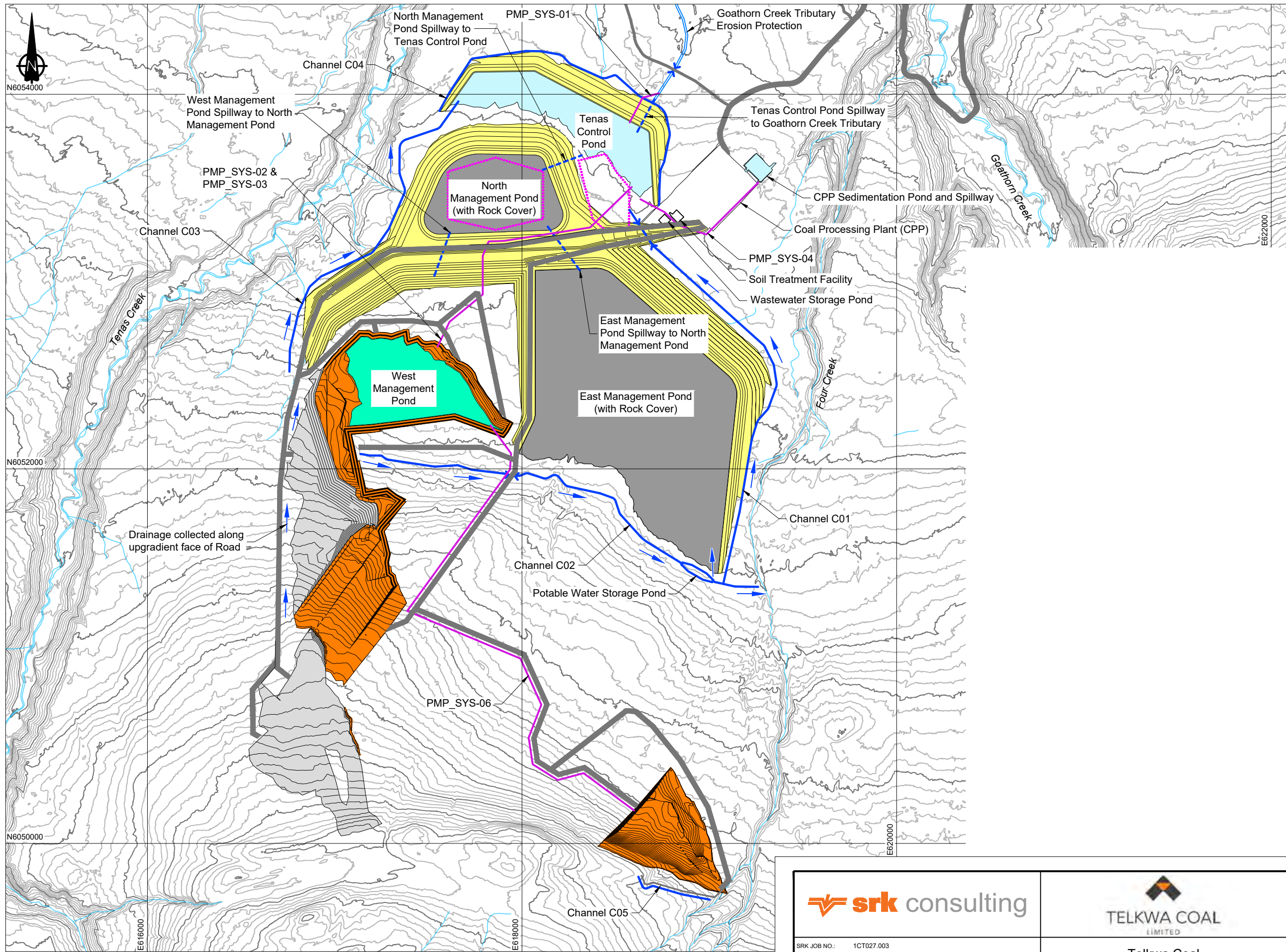
Water Management
End of Year 5

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FIGURE:
3-5

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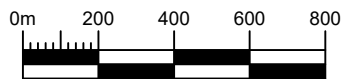
- Water Management Feature
- Water Flow Direction
- Spillway
- Culvert
- Existing Watercourse
- Pipeline
- Proposed Road
- Dam and Butress
- Pit Excavation
- Sedimentation and Control Pond
- Active Management Pond
- Backfill / NAG Pile
- Rock Cover
- Topsoil Stockpile

NOTES

- Contours shown at 5.0m intervals.
- All dimensions shown in meters unless otherwise stated.
- West Management Pond is active and encompasses the northern extents of the open pit. All PAG rock and processed rock to be placed in West Management Pond.
- Non-PAG rock and overburden cover placed across final PAG rock surface in East Management Pond. Final surface is vegetated and spillway upgraded to closure design.
- Non-PAG rock placed as backfill in open pit.
- All runoff collected in open pit areas pumped to West Management Pond.
- Channel C05 is constructed prior to open pit mining in southeastern region of Project footprint.
- PMP_SYS-05 to be installed at Explosive Magazine, Explosive Facility and Rail Infrastructure Sedimentation Pond to discharge nominal flows up to 1 in 10-year event to downstream environment.

REFERENCE

NAD83 UTM Zone 9.



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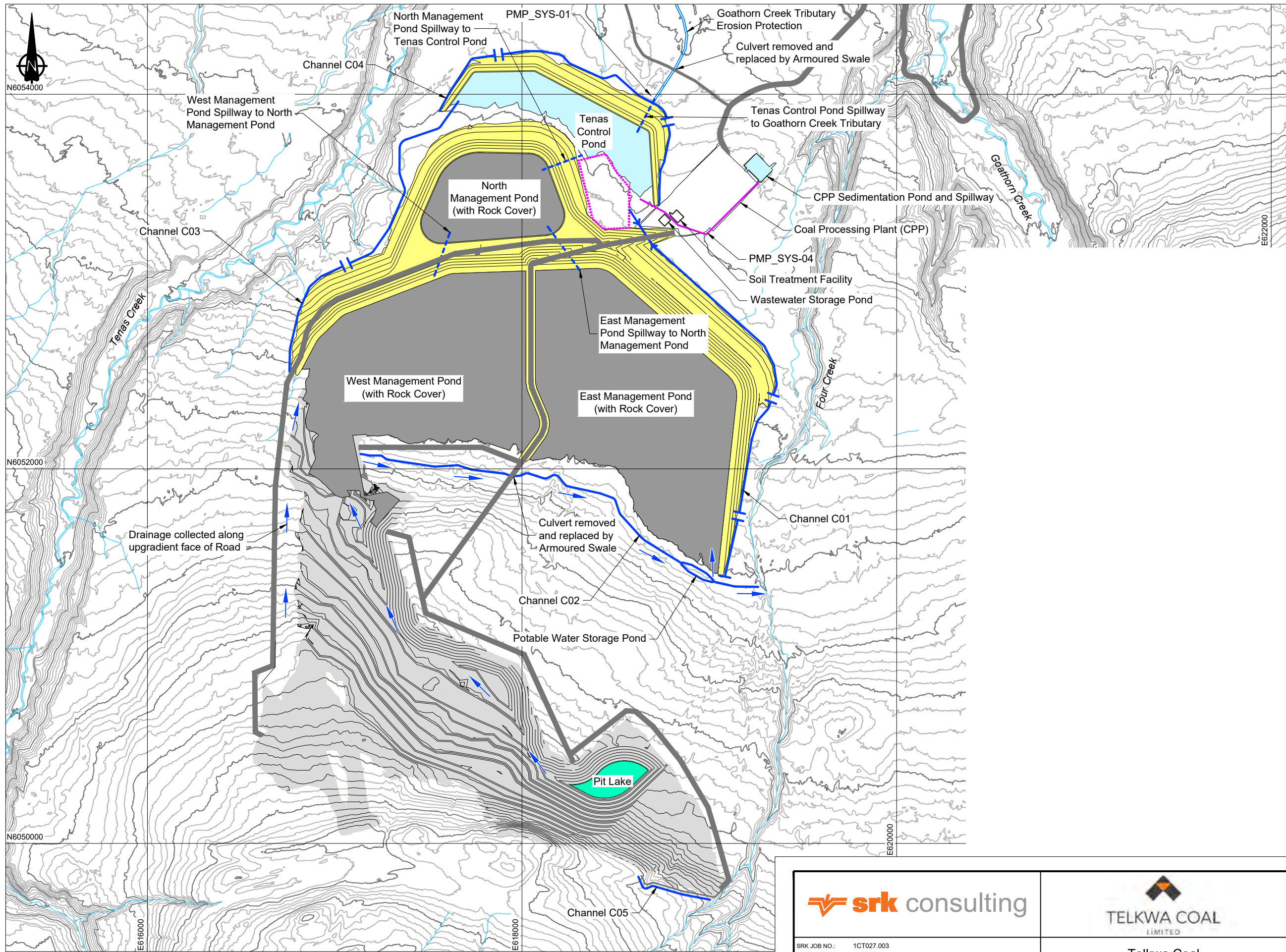
Water Management
End of Year 10

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3-6

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LEGEND

- Water Management Feature
- Water Flow Direction
- Spillway
- Culvert
- Channel Breach
- Existing Watercourse
- Pipeline
- Proposed Road
- Dam and Buttress Sedimentation and Control Pond
- Pit Lake
- Backfill / NAG Pile
- Rock Cover
- Topsoil Stockpile

NOTES

- Contours shown at 5.0m intervals.
- All dimensions shown in meters unless otherwise stated.
- Active mining and processing is complete.
- Non-PAG rock and overburden cover placed across final PAG rock surface in West Management Pond. Final surface is vegetated and spillway upgraded to closure design.
- Non-PAG rock placed as backfill in open pit or as dam buttresses is covered with overburden.
- Channels and sedimentation ponds continue to operate during reclamation and decommissioning of upstream infrastructure until water quality is acceptable for direct discharge.
- After revegetation and reclamation is complete, Tenas Control Pond is dewatered to Goathorn Creek and Plant Sedimentation Pond, Explosives Magazine and Facility Sedimentation ponds are decommissioned.
- Channels C01, C03 and C04 are breached.
- Channel C02 and C05 continue to divert water to Four Creek. Culvert on C02 is removed and replaced with swale channel.
- Pit lake forms in southern extents of open pit. Discharge flows through non-PAG rock backfill towards West Management Pond.

REFERENCE

NAD83 UTM Zone 9.



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Environmental Assessment

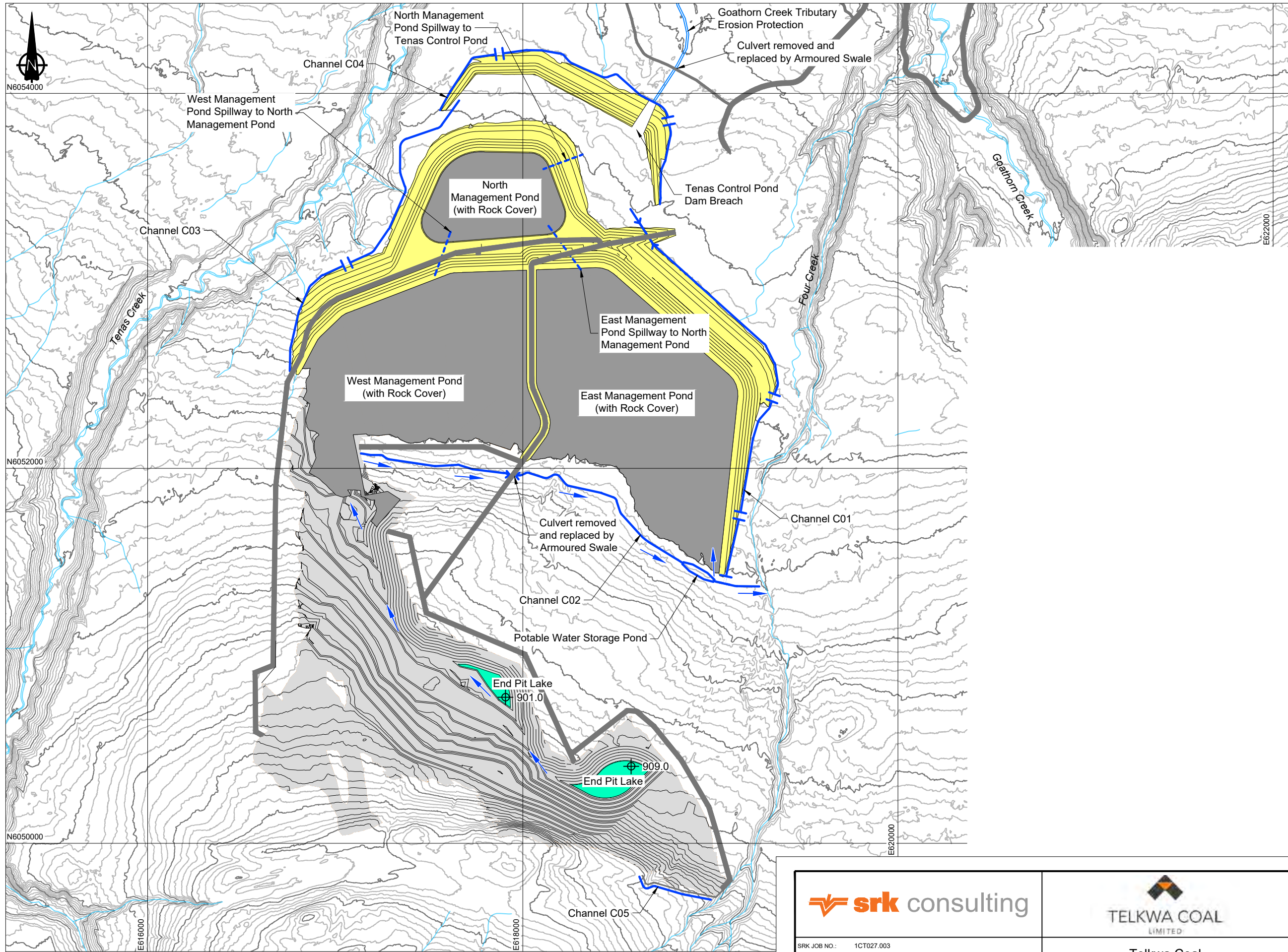
Water Management
Closure and Reclamation

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2021/07/16

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FIGURE:
3-7

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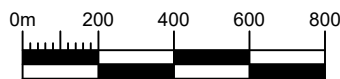
- Water Management Feature
- Water Flow Direction
- Spillway
- Culvert
- Channel Breach
- Existing Watercourse
- Proposed Road
- Dam and Butress
- End Pit Lake
- Backfill / NAG Pile
- Rock Cover

NOTES

- Contours shown at 5.0m intervals.
- All dimensions shown in meters unless otherwise stated.
- Tenas Control Pond is breached and Goathorn Creek tributary erosion protection is extended to the outlet of the North Management Pond spillway.
- Closure spillways route runoff from covered Management Ponds to Goathorn Creek tributary.
- Channels C01, C03 and C04 are breached.
- Channel C02 and C05 continue to divert water to Four Creek.
- Pit lake forms in southern extents of open pit. Discharge flows through non-PAG rock backfill towards West Management Pond.
- Coal Processing Plant (CPP), CPP sedimentation pond, Wastewater Storage Pond and Soil Treatment Facility are decommissioned.

REFERENCE

NAD83 UTM Zone 9.



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TELKWA COAL
LIMITED

Telkwa Coal

Environmental Assessment

Water Management
Post-Closure

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FIGURE: 8

4 Water Management Design

This section summarizes the design basis and the design of the water management structures listed in Table 3-1.

4.1 Design Peak Flow Estimates

A HEC-HMS model (USACE 2015) was developed to evaluate design peak flows and flood volumes. Model inputs were calibrated to match unit peak flows developed for the Goathorn Creek near Telkwa Water Survey of Canada station (08EE008), which consists of 60 years of daily flow records, using a frequency pairing approach between rainfall-depth return periods and flood event return periods.

The Soil Conservation Service (SCS) Curve Number (CN) method was used to estimate precipitation excess (*i.e.*, runoff). The Clark Unit Hydrograph method was used to transform precipitation excess into an outflow hydrograph (USACE 2015). The Clark Unit Hydrograph method uses a storage coefficient and time of concentration (T_c). T_c was estimated using the empirical equation developed by Watt and Chow (1985) and the storage coefficient was determined as a fraction of T_c (Sabol 1988).

The alternating block method and climate change adjusted DDF datasets were used to define the storm hyetographs for a range of return periods. The alternating block method is a straightforward way of developing a design hyetograph using precipitation depths from a corresponding storm duration (Chow, 1994).

Table 4-1 summarizes unit peak flows developed for Goathorn Creek (08EE008, area = 125 km²) and calibrated CN number for a range of return periods. Unit peak flows for Goathorn Creek were developed based on the Water Survey of Canada streamflow record. In addition, calibrated CN's were adjusted to simulate wet antecedent moisture conditions (AMC III), assuming that the calibrated results were representative of typical antecedent moisture conditions (AMC II). The adjusted CN's are also presented in Table 4-1.

Table 4-1: Goathorn Creek near Telkwa (WSC 08EE008) Curve Number Calibration Results

Return Period	Unit Peak Flow (L/s/km ²)	Peak Flow (m ³ /s)	Calibrated CN	Adjusted CN for AMC III
2	100	12.5	74.2	87.3
5	145	18.1	67.9	83.1
10	180	22.5	64.7	80.8
25	227	28.4	61.5	78.4
50	263	32.9	58.4	76.1
100	298	37.3	54.2	72.8
200	334	41.8	51.1	70.1
500	382	47.8	47.1	66.6
1000	418	52.3	44.2	64.0

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\300_Hydrology_Update\Telkwa_CNFlowCalibration_gmj_rev06_sab.xlsm

Figure 4-1 illustrates a decreasing trend in CN numbers for larger return periods. Decreasing CN with increasing storm size is an observed hydrological behavior, where results below represent a standard behavior output with a near-constant CN for higher return periods (Hawkins, 1993).

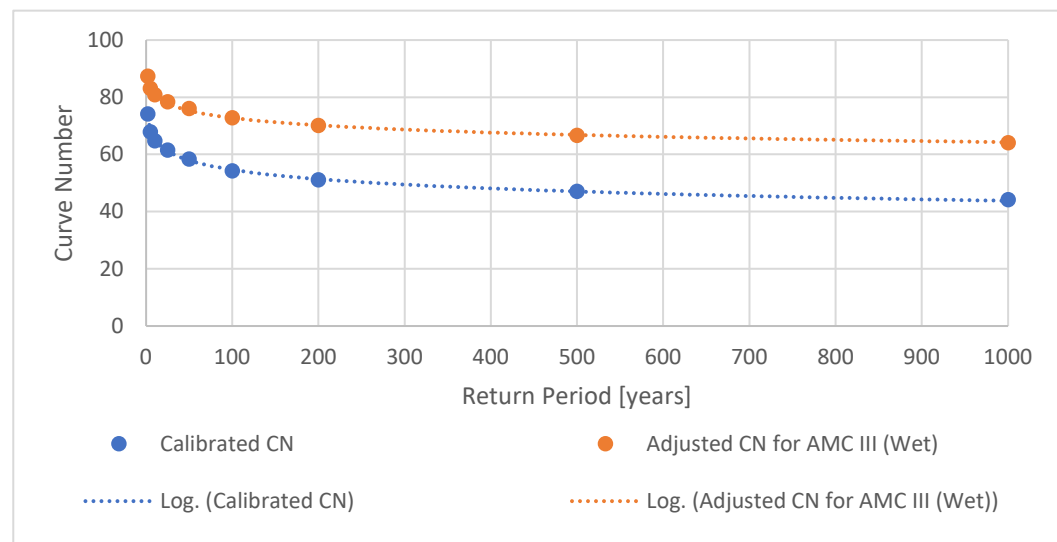


Figure 4-1: Calibrated CN Values and CN Values after Adjustments for AMC III Saturated Conditions for Return Periods

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Telkwa_CNFlowCalibration_rev07_sab.xlsm

Snowmelt is expected to govern the peak flows in this region and was therefore incorporated in addition to the rainfall distribution in hydrologic modeling for infrastructure designs. For storm events up to the 200-year return period, the average year maximum 24-hour snowmelt depth was included. For calculations related to the Inflow Design Flood and spillway routing, the return period for snowmelt depth was selected based on CDA (2007) recommendations. The shape of the snowmelt inflow hydrograph was developed using a sinusoidal distribution over a 13-hour period based on the average daylight hours for Smithers during April (NRC, 2020). The snowmelt depths presented in Table 2-7 were distributed, providing a peak snowmelt at mid-day.

Probable Maximum Flood (PMF) conditions were also modelled in HEC-HMS. Two PMF cases were compared, as per CDA (2007) recommendations:

- Spring rainfall-dominated flood: spring PMP rainfall event (Table 2-5) with a 1 in 100-year snowmelt event (Table 2-7)
- Summer rainfall-dominated flood: summer PMP rainfall event (Table 2-5)

CDA also recommends comparing the spring snowmelt-dominated event, which includes the probable maximum snow accumulation, often estimated as twice the 1 in 100-year snowpack (Alberta Transportation 2004) with a 1 in 100-year rainfall event. In this case, the sum of the 1 in 100-year rainfall event and two times the 1 in 100-year snowmelt is less than the spring PMP rainfall depth on its own and was therefore assumed to not be as critical as the rainfall-dominated flood.

Like other rainfall events, the spring and summer PMP's were distributed using the alternating block method, where duration factors from the 1 in 1,000-year event in the depth-duration-frequency analysis were applied to each PMP.

4.2 Diversion and Collection Channels

This section summarizes the design of channels and associated erosion protection for the Project. Culverts and road crossings are described separately in Section 4.9.

4.2.1 Engineered Channels

Channels that operate throughout the life of mine but are decommissioned at closure were designed to meet the following conditions:

- Convey the peak flow from the 1 in 10-year 24-hour return period flood event without destabilizing erosion protection
- Contain the 1 in 25-year 24-hour return period flood event while maintaining minimum freeboard conditions
- Contain the 1 in 100-year 24-hour return period flood event without overtopping

Channels that are expected to continue operating post-closure were designed to meet the following conditions:

- Convey the 1 in 200-year 24-hour return period flood event without destabilizing erosion protection and maintaining minimum freeboard

Peak flows incorporate climate change and contributions from snowmelt. The hydraulic analysis and sizing of channels were determined using Manning's equation under a steady state normal depth condition.

Given that riprap is not readily available at the Project, channel geometry incorporates the use of turf reinforcement mat (TRM) as erosion protection for channels, where feasible. Channels with steep grades and higher velocities or that are required post-closure will be lined with riprap sourced off-site. Channels that are breached in closure are expected to be operated within the design lifespan of the TRM.

Over time, vegetation is expected to become established within the channels. Therefore, channels were sized to convey the design peak flow with the addition of vegetation growth. Both TRM and riprap can promote vegetation growth.

Channels were also designed with typical geometry that reflect the size and capability of the anticipated construction equipment. Channels were designed to have a minimum base width of 2.0 m, side slopes of 2H:1V and a minimum freeboard of 0.3 m.

Channel excavated depths vary due to channel grade and type of erosion protection, and each channel is divided into segments of different grades. Minimum channel depths were designed to convey the design peak flow with the greater of the riprap Manning's roughness or the roughness value after vegetation is established.

Riprap sizing and Manning's roughness values were derived through model iterations. The Manning's equation was evaluated using initial roughness values and evaluated for each channel segment of different longitudinal slope. Where the Manning's roughness for the specified riprap was greater than the Manning's specified for vegetation, the riprap roughness was used for design depth calculations.

Riprap sizing for the closure conveyance channels was estimated using three expressions: Isbash (USACE 1991), Robinson et al. (1998), and Khan and Ahmad (2011) and the riprap thickness was specified as two times the D_{50} . Given a riprap size estimate, a Manning's roughness value (n) was then calculated based on three relationships by Strickler, Anderson, and Abt et al. (Abt. et al. 1988).

Alignments of channels were developed to maintain the target minimum design slope or reduce slopes overall. A minimum bend radius equal to 10 times the channel width should be considered as part of detailed engineering, along with additional erosion protection (i.e., larger riprap) in localized areas on the outer extents of any large bends.

A summary of the design criteria for engineered channels are presented in Table 4-2.

Table 4-2: Design Criteria for Engineered Channels

Parameter	Value	Reference and Justification
Capacity and Design Floods for Operations only Channels		
Design flood for erosion protection	1 in 10-year 24-hour event	SRK engineering judgement. Regular maintenance and repair will be conducted and consequence of failure during operations is low.
Design flood for channel capacity	Maximum of 1 in 25-year 24-hour event with freeboard or 1 in 100-year 24-hour event without freeboard	SRK engineering judgement
Capacity and Design Floods for Closure Channels		
Design flood for erosion protection	1 in 200-year 24-hour event	SRK engineering judgement
Design flood for channel capacity	1 in 200-year 24-hour event with freeboard	SRK engineering judgement
Geometry		
Minimum Freeboard	0.3 m	Best practices
Minimum Base Width	2.0 m	Constructability constraint
Minimum channel grade	0.5%	Constructability constraint
Channel side slopes	2:1 H:V	SRK engineering judgement
Thickness of topsoil layer below turf reinforcement mat	0.3 m	SRK engineering judgement
Erosion Protection		
Manning's roughness for Turf Reinforcement Mat	0.026	(Nilex, 2018)
Manning's roughness for vegetation	0.040	(Chow, 1994)
Manning's roughness for riprap	Varies, based on riprap size (0.030 to 0.046)	(Abt. et al. 1988)
Maximum permissible velocity for Turf Reinforcement Mat	2.9 m/s	(Nilex, 2018)
Riprap thickness	2 x D ₅₀	(BC MOE, 2000)
Geotextile	Non-woven, for riprap sections only	SRK engineering judgement

Source: developed in report

The mine plan for each year of operations was analyzed to determine the most critical condition in terms of catchment area contribution to each channel. Table 4-3 summarizes the various collection and diversion channels for the Project along with the most critical period in the mine life in terms of catchment contributions and the associated consequence of failure.

Table 4-3: Summary of Tenas Project Collection and Diversion Channels

Channel	Critical Phase	Description and Consequence of Failure	Drawing Reference
C-01	End of Year 2	Collects seepage and runoff from the eastern extents of the Project, including East Management Pond and North Management Pond dams and buttresses, discharging water to Tenas Control Pond. If overtopped, potentially high-sediment water would report to Four Creek. May result in short-term violation of discharge permit. Design assumes C-02 is discharging into C-01 towards Tenas Control Pond. This channel will be breached at closure, after buttresses are vegetated.	02, 03, 04, 05, 06
C-02	End of Year 5	Collects natural runoff upgradient of the East Management Pond and discharges to Four Creek, C01 and the EMP through a series of overflow weirs and culverts (see Section 4.2.2). If overtopped, undisturbed runoff would report to EMP or increased flows to Four Creek. May result in local erosion on East Pond rock cover once the facility is closed or local erosion in Four Creek. This channel will remain in-place after closure, discharging permanently towards Four Creek and the EMP. Channel includes a storage cell for potable water (see Potable Water Storage Pond design in Section 4.7.1).	07, 08, 09, 10, 11
C-03	End of Year 0	Collects runoff along the southern portion of the western mine access road, as well as seepage and runoff from the West Management Pond and North Management Pond dams and buttresses, discharging water to Tenas Control Pond. If overtopped during larger flood events than the design storm, potentially high-sediment water would report to Tenas Creek. This channel will be breached at closure, after buttresses are vegetated. Runoff along the northern portion of the western mine access road will be collected in roadside ditches, designed by others.	13, 14, 15, 16, 17
C-04	End of Year 10	Collects runoff and seepage along the toe of the Tenas Control Pond dam and will provide sediment control during the construction period. Consists of two channels, along the east and west portions of the dam, which meet near the outlet of the Tenas Control Pond spillway. Discharge back to Tenas Control Pond via a pump or directly to Goathorn Creek tributary. This channel will be breached at closure or during progressive reclamation in operations, after buttresses are vegetated.	18, 19, 20, 21
C-05	End of Year 10	Collects natural runoff upgradient of the southeastern portion of the open pit, discharging into Four Creek. If overtopped, non-contact water would report to open pit. This channel will continue to operate post-closure, diverting runoff to Four Creek.	22, 23

Source: developed in report

Catchment characteristics such as catchment areas, flow path and time of concentration were determined using available 2017 LIDAR and were generated in Global Mapper (developed by Blue Marble Geographic). Calibrated CN values (Section 4.1) were applied to natural catchment areas for the 10 and 25-year return period flood analyses, and the adjusted AMC III (saturated soils) CN values were applied for the 100-year and 200-year return period flood analysis. The climate change adjusted DDF precipitation data were incorporated into the HEC-HMS hydrologic model using the Alternating Block distribution method in addition to the snowmelt distribution for an average year maximum snowmelt event, as described in Section 4.1.

The Muskingum-Cunge routing method was used to transform a storm hydrograph within a catchment area along a channel reach. The input parameters for routing reaches of collection channels were based on standard channel geometries, as well as channel length, slope, and Manning's roughness.

Table 4-4 presents design peak flows for engineered channels.

Table 4-4: Engineered Channel Peak Flow Results

Channel	Catchment Area [km ²]	Outlet	Peak Flows[m ³ /s]			
			1 in 10-year event	1 in 25-year event	1 in 100-year event	1 in 200-year event ⁽⁴⁾
C-01	3.9 ⁽¹⁾	Tenas Control Pond	7.0	7.8	20.5	n.a.
C-02	3.2	C-01 or Four Creek ⁽²⁾	5.8	6.4	17.0	19.4
C-03	3.3	Tenas Control Pond	4.4	4.8	12.3	n.a.
C-04	1.2 ⁽³⁾	Goathorn Creek Tributary or Tenas Control Pond	2.0	2.2	5.8	n.a.
C-05	0.5	Four Creek	1.1	1.3	3.4	6.6

Source: \\Srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\ChannelDesign\ Telkwa_PeakFlows_1CT027-003_rev03_nf_sab.xlsx

Notes:

- ¹ C-01 catchment area presented above is the combined area into the channel, including area contributions from C-02
- ² C-02 is a collection channel that will operate with a flow control valve which will enable discharge to either the C-01 towards Tenas Control Pond or directly into Four Creek through a series of overflow weirs
- ³ C-04 catchment area presented above represents the combined area to the toe drainage ditches C-04a and C-04b along Tenas Control Pond Dam during construction of the dam. Discharge during dam construction will be to Goathorn Creek tributary, while discharge after Tenas Control Pond is constructed may be into the Tenas Control Pond via a trash pump
- ⁴ The 200-year peak flow is applied to channels in place beyond mine closure , which includes C-02 and C-05

Channel alignments were selected to minimize water diverted towards Project infrastructure to reduce the downstream effects on natural watercourses. The alignment, profile and typical cross sections are illustrated in the design drawings.

Hydraulic results are summarized in Table 4-5 for each channel section.

Table 4-5: Engineered Channel Design Results

Channel	Section	Grade [%]	1 in 10-year Velocity (m/s)	1 in 200-year Velocity (m/s)	Erosion Protection	Riprap D ₅₀ [m]	Base Width (m)	10-year Flow Depth [m]	25-year Flow Depth [m]	100-year Flow Depth [m]	200-year Flow Depth [m]	Design Depth [m]
C-01	1	3.50%	2.7	n.a.	Riprap ⁽²⁾	0.15	5	0.44	0.50	0.86	n.a.	0.90
	2	0.50%	1.3	n.a.	TRM ⁽³⁾	n.a.	5	0.82	0.86	1.46	n.a.	1.50
C-02	1	0.50%	1.5	1.7	Riprap	0.10	5	0.63	0.77	1.32	1.42	1.80
C-03	1	9.20%	3.2	n.a.	Riprap	0.30	2	0.47	0.49	0.79	n.a.	0.80
	2	1.00%	2.2	n.a.	TRM	n.a.	2	0.63	0.81	1.29	n.a.	1.30
C-04	1	3.40%	2.6	n.a.	TRM	n.a.	2	0.36	n.a.	n.a.	n.a.	0.70
C-05	1	1.0-15%	1.3	2.1	Riprap	0.10-0.30 ⁽⁴⁾	2	0.34	0.41	0.69	0.84	1.20

Source: \\Srk.ad\dfs\anal\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Channel Design\ Telkwa_ChannelDesign_Update_sab_rev06.xlsm

Notes:

- ¹ Rows highlighted in orange identify channels that will be in-place after mine closure. Riprap and channel design depth for closure channels were developed based on the 1 in 200-year return period event, as outlined in Table 4-2
- ² Riprap was selected for section 1 of C-01 and reflects a Manning's roughness of 0.035. The expected velocities with the lower Manning's roughness for TRM prior to establishment of vegetation exceeded the TRM threshold of 2.9 m/s
- ³ TRM = turf reinforcement mat
- ⁴ Riprap varies based on longitudinal slope. For slopes less than 10%, 0.10m D₅₀ can be used. For slopes greater than 10%, riprap D₅₀ of 0.30m required

4.2.2 Erosion Protection at Project Discharge Locations

Erosion protection was also specified for natural flow pathways downstream of Project discharges, including the discharge from channel C-02 into Four Creek and the discharge from Tenas Control Pond into Goathorn Creek tributary. Both systems will remain in place post-closure, and therefore the 200-year return period event with climate change was selected along with riprap and geotextile lining for erosion protection.

Channel C-02 and Four Creek

The alignment and discharge location of the C-02 diversion channel into Four Creek will result in a net increase to the existing catchment area in a portion of Four Creek (see SRK 2021b for details). As a result, the flow rates in this portion of the stream will increase, resulting in potential for increased erosion along the natural stream.

A control structure was designed for the outlet of C-02 channel to limit the inflows to Four Creek and thereby reduce the potential for increased erosion rates. A threshold velocity was estimated based on assumptions around the bedding material and vegetation in the creek. Based on United States Department of Agriculture guidance documentation, the permissible maximum velocity for gravel-based streams with moderate sediment transport and grasses, and a slope ranging from 5 to 10%, is 2.1 m/s (USDA 2007). The objective of the control structure is to limit the discharge rate from C-02 into Four Creek such that the combined flow rate during the design storm event from the natural catchment and the structure outflow produce an average channel velocity that is less than or equal to the threshold velocity of 2.1 m/s. The design storm event for this structure was selected to be the 1 in 10-year event.

The 1 in 10-year historical peak flow was calculated based on the existing catchment area at the proposed C-02 channel outlet, equal to 5.17 km², using the design peak flow methodology outlined in 4.1. Cross-sectional geometry, including longitudinal slope, channel side slopes and base width, were estimated from the available LiDAR data. The Manning's equation was used to estimate average velocity, based on a roughness value (n) back-calculated from manual flow measurements near the Four-Hydro station, calculated to be 0.09. This value corresponds to a stream with weedy reaches, deep pools, and underbrush, which is consistent with observations in baseline monitoring reports (ERM 2020). The acceptable combined peak flow was back-calculated to produce an average velocity equal to the threshold value. The additional flow rate from the control structure was calculated as the difference between the acceptable combined peak flow and the historical peak flow during the design event. Flows greater than the calculated maximum flow rate during the 10-year event will be conveyed to an alternate location to reduce the effect on Four Creek. Design criteria and assumptions are summarized in Table 4-6 and results are presented in Table 4-7.

Table 4-6: Four Creek Erosion Protection Design Criteria

Parameter	Unit	Value	Comment
Design Event for Control Structure	years	1 in 10-year, 24-hour storm	.
Design Event for Overflow Structure	Years	1 in 200-year, 24-hour storm	Consistent with closure infrastructure
Manning's n	unitless	0.09	Back-calculated based on eight manual flow measurements and stream surveys (ERM trip reports, 2020-2021)
Channel Threshold Velocity	m/s	2.1	Grass lined channels with 5 to 10% slope with coarse bedding (USDA 2007)

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\ChannelDesign\FourCreek_C02ErosionProtectionDesign_rev01_sab.xlsm

Table 4-7: C-02 Channel Discharge to Four Creek Design Flow Results

Parameter	Unit	Value	Comment
<i>Natural Channel</i>			
Catchment Area	km ²	5.15	Pre-mining catchment at the outlet of C-02 into Four Creek
10-year Peak Flow	m ³ /s	6.99	Based on calculated time of concentration 1 hour, CN of 80.8
Average Velocity during Design Event	m/s	1.97	
<i>Combined Conditions with Engineered Control Structure and Natural Channel</i>			
Maximum Acceptable Velocity	m/s	2.1	Based on threshold velocity in Table 4-6
Combined Design Flow	m ³ /s	8.50	Back-calculated using Manning's equation and maximum acceptable velocity
Design Flow from Control Structure to Four Creek	m ³ /s	1.51	Difference between combined design flow and natural channel peak flow

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\ChannelDesign\FourCreek_C02ErosionProtectionDesign_rev01_sab.xlsm

The control structure includes culverts to discharge flow from C-02 into Four Creek. Any flow from C-02 more than the design flow rate (1.51 m³/s) will be conveyed to C-01 during construction and operations phases until the EMP dam is constructed, at which point overflows will flow into the EMP directly. The overflow structure will consist of an overflow weir which will convey flows up to the 200-year return period event, consistent with design criteria for post-closure infrastructure. After culvert placement, backfill (Non-PAG and overburden) will be placed across culverts, forming an access road between channels C-02 and C-01. Erosion protection downstream of the culverts into Four Creek includes riprap since the structure will remain in-place through post-closure.

The Potable Water Storage Pond will be in the downstream extents of C-02 and will provide attenuation capacity during storm events. The culvert diameter and slope for the control structure were optimized along with geometry and elevation of the overflow weir such that the design flow of 1.51 m³/s is not exceeded during the 10-year event, and that events in excess of the 10-year event are safely conveyed to the appropriate downstream structure (C-01 in early operations and EMP for remaining operations and post-closure period). Culvert performance curves were developed in HY-8 and inflow hydrographs were developed in HEC-HMS. Overflow weir geometry (i.e., base width and spillway invert elevation) was adjusted to reduce velocities and associated riprap requirements. Erosion protection and design of the channel downstream of the control structure into Four Creek was also prepared based on the slopes of the valley.

The resultant configuration is presented in Table 4-8. Designs are presented in Figures 9 through 12 of Appendix D.

Table 4-8: C-02 Control Structure and Overflow Weir Design Results

Parameter	Unit	Value	Comment
<i>Control Structure to Four Creek</i>			
Culvert Diameter	mm	400	
Number of culverts	-	2	Provides contingency discharge during pipe maintenance or in case of blockage.
Culvert slope	%	2%	Selected to optimize discharge rate
Invert elev. relative to base of Potable Water Storage Pond	m	5.0	At top of Potable Water Storage Pond
Peak Outflow during Design Event (1 in 10-year 24-hour event)	m ³ /s	1.50	Less than threshold combined flow rate of 1.51 m ³ /s
<i>Overflow Weir to C-01 / EMP</i>			
Base width	m	15	Selected to reduce riprap requirements
Side slopes	H:V	10:1	For trafficability
Invert Elev. relative to base of Potable Water Storage Pond	m	6.0	1.0 m above top of Potable Water Storage Pond. Note top of C-02 channel is 6.8 m, providing ample freeboard.
Peak Outflow during Design Event (1 in 200-year 24-hour event)	m ³ /s	14.5	Consistent with closure design criteria
Riprap D₅₀	m	0.10	
Riprap layer thickness	m	0.20	2xD ₅₀ , (BC MOE, 2000)
<i>Channel from Control Structure to Four Creek</i>			
Base width	m	5	
Side slopes	H:V	2:1	
Maximum slope	%	11%	Ranges from 1% to 11% along edge of valley
Riprap D₅₀	m	0.15	
Riprap layer thickness	m	0.30	2xD ₅₀ , (BC MOE, 2000)

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\ChannelDesign\FourCreek_C02ErosionProtectionDesign_rev01_sab.xlsm

Detailed alignment selection of the channel extension from the control structure to Four Creek and appropriate energy dissipation systems will be required to reduce effects on the in-stream fish population. A field survey will be completed as part of detailed engineering.

Goathorn Tributary

The Goathorn Creek tributary erosion protection analysis included a review of historical flow conditions, and an evaluation of the expected Project discharge from Tenas Control Pond.

Two components were evaluated for the erosion protection along the Goathorn Creek tributary:

- The extent along the tributary that erosion protection is required
- The type and size of erosion protection

The extent to which erosion protection will be required along the tributary was reviewed based on the peak flows that would have been expected from the tributary's natural watershed in comparison to the peak flows discharging from the Project. Based on the magnitude of the Project catchment, the natural watershed area would be increased by up to 20 times the natural area at the upstream extents, and as high as 4 times at the confluence with Goathorn Creek. This change could result in increased rates of erosion of the natural creek bed, and increased sediment loading in Goathorn Creek. For this reason, erosion protection was included along the full length of the Goathorn Tributary to its outlet at Goathorn Creek (approximately 2.1 km length).

Since Project discharge will continue throughout closure, riprap was selected as the type of erosion protection for the Goathorn tributary. The riprap sizing was developed based on the expected Project discharge rates during a 1 in 200-year return period peak flow event.

The most critical period for discharge will be at closure of North Management Pond once the Tenas Control Pond has been decommissioned and removing the attenuation capacity within the pond. HEC-HMS was used to simulate the 200-year peak discharge, along with catchment characterization of the site at closure, like channel designs. The AMC III CN values were applied to natural catchment areas (5.1 km²) for the 200-year return period flood, which was routed through the East and West Management Pond spillways. The channel dimensions and slope were obtained from the site topography.

Erosion protection works at Goathorn tributary are summarized in Table 4-9.

Table 4-9: Summary of Erosion Protection Design at Goathorn Creek Discharge Location

Parameter	Goathorn Creek Tributary
200-year event design peak flow [m³/s]	5.2 ⁽¹⁾
Average slope [%]	3.8%
Design event velocity [m/s]	2.45
Riprap D ₅₀ [m]	0.20
Riprap thickness [m]	0.40
Length [m]	2,100

Source: Z:\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\TributaryGHC\GoathornTributary_FlowAnalysis_04292021_sab_rk.xlsm

Notes:

¹ Discharge from North Management Pond at closure, attenuated by the East and West Management Ponds

A road crossing will be installed along the tributary for the access road to the Explosives Magazine area (CU-05) and is described in Section 4.9.

4.3 Dam Classification and Design

Water containing structures were evaluated according to CDA Guidelines (2013) and BC Dam Safety regulations (BC 2016) and supported by the Global International Standards on Tailings Management (GISTM) (ICMM, 2020) and BC MOE Technical Guidance (BC MOE 2015).

The following sections summarize the dam classifications and methodologies for spillway design, as well as the freeboard calculations.

4.3.1 Classification

The consequence of failure was evaluated for each facility, including water storage ponds which will be constructed entirely in excavation and will not rely on a dam or embankment. Ponds are designed with an emergency spillway sized based on the associated dam classification, following guidance from CDA (2013).

A detailed dam break model was prepared for the major containment structures, including the Management Pond dams and the Tenas Control Pond dam (Appendix B). Consequence classification for the remaining five ponds were evaluated qualitatively.

Table 4-10 is a reproduction of the CDA-recommended consequence classification criteria. The classification systems are based on the incremental consequences of a dam failure. The incremental consequences of failure are defined as the total effects of a dam failure, beyond the effects that would have resulted from the condition that caused the failure (e.g., a large earthquake or a large flood event) had the dam not failed. During the dam classification process, each of the four hazard rating components in Table 4-10 are considered individually, and the overall dam hazard rating is defined by the component with the highest (i.e. most severe) rating.

Table 4-10: CDA (2013) Dam Classifications

Dam Class	Population at Risk ⁽¹⁾	Incremental Losses		
		Loss of Life ⁽²⁾	Environmental and Cultural Values	Infrastructure and Economics
Low	None	0	Minimal short-term loss	Low economic losses: area contains limited infrastructure or services
Significant	Temporary only	Unspecified	No long-term loss	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	No significant loss or deterioration of fish or wildlife habitat	High economic losses affecting infrastructure, public transportation, and commercial facilities
Very High	Permanent	100 or fewer	Loss of marginal habitat only	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Restoration or compensation in kind highly possible	Extreme losses affecting critical infrastructure or service (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)

Notes:

¹ Definitions for population at risk:

- None: There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure
- Temporary: People are only temporarily in the dam-breach inundation zone (e.g., seasonal cottage use, passing through on transportation routes, participating in recreational activities)
- Permanent: The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); Three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is conducted)

² Implications for loss of life:

- Unspecified: The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season

Table 4-11 presents the consequence classifications for excavated water storage ponds that do not rely on embankments for containment. In the case of each pond, overtopping is the only credible failure mode, which is expected to occur by design for events more than the 1 in 10-year return period inflow event, according to BC Ministry of Environment (BC MOE) Technical Guidance (BC MOE 2015). During these events, some sediment may be released to the environment, and erosion may occur in the downstream receiving environment. For this reason, a low consequence classification was assigned to excavated water storage ponds.

Table 4-12 summarizes consequence classification for embankments, as described in Appendix B.

Table 4-11: Consequence Classification for Excavated Water Storage Ponds

Pond	Description of Failure Effect / Comments	Population at Risk	Incremental Losses			Recommended Consequence Classification
			Loss of Life	Environmental and Cultural Values	Infrastructure and Economic Loss	
CPP Sedimentation Pond	Overtopping scenario would result in water from the pond flowing into Four Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e. repairs to and rebuild the dam if required) and are expected to be low (Low)	Low
Rail Infrastructure Sedimentation Pond	Overtopping scenario would result in water from the pond flowing into the Bulkley River	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e. repairs to and rebuild the dam if required) and are expected to be low (Low)	Low
Explosives Facility Sedimentation Pond	Overtopping scenario would result in water from the pond flowing into Tenas Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e. repairs to and rebuild the berm and pad if required) and are expected to be low (Low)	Low
Explosive Magazine Sedimentation Pond	Overtopping scenario would result in water from the pond flowing into Tenas Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e. repairs to and rebuild the berm and pad if required) and are expected to be low (Low)	Low
Potable Water Storage Pond	Overtopping scenario would result in water from the pond flowing into the East Management Pond, C-01 or into Four Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e. repairs to and rebuild the pond and channel if required) and are expected to be low (Low)	Low
Soil Treatment Facility Pond	Overtopping scenario would result in water from the pond flowing over the CPP and ultimately into the CPP Sedimentation Pond	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e. repairs to and rebuild the pond and channel if required or clean-out of the CPP Sedimentation Pond downstream) and are expected to be low (Low)	Low
Wastewater Storage Pond	Overtopping scenario would result in water from the pond flowing over the CPP and ultimately into the CPP Sedimentation Pond	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e. repairs to and rebuild the pond and channel if required or clean-out of the CPP Sedimentation Pond downstream) and are expected to be low (Low)	Low
Gravel Quarry Sedimentation Pond for Pits 1 and 2	Overtopping scenario would result in water from Pond into the Quarry and with enough water into Goathorn Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e., repairs to sedimentation pond and delay in gravel removal) and are expected to be low (Low)	Low

Pond	Description of Failure Effect / Comments	Population at Risk	Incremental Losses			Recommended Consequence Classification
			Loss of Life	Environmental and Cultural Values	Infrastructure and Economic Loss	
Rock Quarry Sedimentation Pond	Overtopping scenario would result in water from pond into Helps Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e., repairs to sedimentation pond and delay in gravel removal) and are expected to be low (Low)	Low
East TAC Sedimentation Pond	Overtopping scenario would result in water from pond flowing into Goathorn Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e., repairs to sedimentation pond and delay in gravel removal) and are expected to be low (Low)	Low
West TAC Sedimentation Pond	Overtopping scenario would result in water from pond flowing into Goathorn Creek	None (Low)	0 (Low)	Anticipate some minimal short-term loss and no permanent loss (Low)	Economic losses may include some limited infrastructure (i.e., repairs to sedimentation pond and delay in gravel removal) and are expected to be low (Low)	Low

Source: Prepared in text

Table 4-12: Consequence Classification for Management Ponds and Tenas Control Pond (Appendix B)

Dam	Description of Failure Effect / Comments	Population at Risk	Incremental Losses			Recommended Consequence Classification
			Loss of Life	Environmental and Cultural Values	Infrastructure and Economic Loss	
Tenas Control Pond	A dam failure scenario would result in water from the pond flowing into Tenas Creek and Goathorn Creek, creating a flood wave towards the Town of Telkwa	Permanent – likely to affect population inhabiting inundation zone (Very High)	100 or fewer (Very High)	The Tenas Control Pond breach will likely cause downstream erosion with temporary deterioration of aquatic and terrestrial habitat. Restoration or compensation in kind is highly possible (Very High)	Economic losses may include infrastructure (i.e., repairs to bridges, buildings, and roadways) and dam rebuilding if required (High)	Very High
North Management Pond	A failure of the dam could cause a cascading failure of the Tenas Control Pond. A dam failure scenario would result in water from the ponds to flow into Tenas Creek and Goathorn Creek	Permanent – likely to affect population inhabiting inundation zone (Very High)	100 or fewer (Very High)	The breach will contribute to downstream erosion with some loss or deterioration of aquatic and terrestrial habitat. Restoration or compensation in kind is highly possible (High)	Economic losses may include infrastructure (i.e., repairs to bridges, buildings, and roadways) and dam rebuilding if required (High)	Very High
East Management Pond	A failure of the dam could cause a cascading failure of the North Management Pond and Tenas Control Pond. A dam failure scenario would result in water from the ponds to flow into Tenas Creek	Permanent – likely to affect population inhabiting inundation zone and mine personnel occupying mine infrastructure complex (Very High)	100 or fewer (Very High)	The breach will contribute to downstream erosion with some loss or deterioration of aquatic and terrestrial habitat. Restoration or compensation in kind is highly possible (High)	Economic losses may include infrastructure (i.e., repairs to bridges, buildings, and roadways) and dam rebuilding if required (High)	Very High
West Management Pond	Depending on the location of the dam failure, the breached water could flow into Tenas Creek, Four Creek and Goathorn Creek. A failure of the dam may also cause a cascading failure of the North Management Pond and Tenas Control Pond	Permanent – likely to affect population inhabiting inundation zone (Very High)	100 or fewer (Very High)	The breach will contribute to downstream erosion with some loss or deterioration of aquatic and terrestrial habitat. Restoration or compensation in kind is highly possible (High)	Economic losses may include infrastructure (i.e., repairs to bridges, buildings, and roadways) and dam rebuilding if required (High)	Very High

Source: Appendix B

4.3.2 Selection of Inflow Design Flood

Given the above dam classifications, the recommended inflow design flood events for the spillways are summarized in Table 4-13. Both the wastewater storage pond and the soil treatment facility pond are located upstream of the CPP Sedimentation Pond, which includes these facilities as part of its catchment area. Spillway designs for the Wastewater Storage Pond are described in presented in Section 1-1 Description of Proposed Project in the EAC Application while the Soil Treatment Facility is described in SRK 2021c.

Table 4-13: Inflow Design Flood based on Dam Classification

Facility	Dam Classification	Minimum Design Flood	Selected Design Flood
Tenas Control Pond	Very High	2/3 Between 1 in 1,000 and PMF	2/3 between 1,000 year and PMF
North Management Pond	Very High	2/3 Between 1 in 1,000 and PMF	2/3 between 1,000 year and PMF
East Management Pond	Very High	2/3 Between 1 in 1,000 and PMF	2/3 between 1,000 year and PMF
West Management Pond	Very High	2/3 Between 1 in 1,000 and PMF	2/3 between 1,000 year and PMF
CPP Sedimentation Pond	Low	Between 1 in 100 and 1 in 1,000	200-year
Rail Infrastructure Sedimentation Pond	Low	1 in 100-year event	200-year
Explosive Facility Sedimentation Pond	Low	1 in 100-year event	200-year
Explosive Magazine Sedimentation Pond	Low	1 in 100-year event	200-year
Potable Water Storage Pond	Low	1 in 100-year event	200-year
Wastewater (Sanitary) Storage Pond	Low	1 in 100-year event	200-year
Gravel Quarry Sedimentation Pond for Pits 1 and 2	Low	1 in 100-year event	200-year
Rock Quarry Sedimentation Pond	Low	1 in 100-year event	200-year
East TAC Sedimentation Pond	Low	1 in 100-year event	200-year
West TAC Sedimentation Pond	Low	1 in 100-year event	200-year

Source: Prepared in text

For sedimentation ponds, the minimum design flood for overflow spillways is the 1 in 200-year design flood (MOE, 2015) which governs in the case of the Low consequence control ponds. Management Pond spillways as well as the spillway for the Tenas Control Pond were designed to convey the partial PMF.

4.3.3 Spillway Design

Each spillway channel was designed with trapezoidal cross-section and a minimum spillway width of 2 m with minimum side slopes of 2H:1V. The spillways will be lined with riprap for erosion protection. Spillway inverts and base widths were defined to provide sufficient minimum freeboard during the inflow design flood (IDF) as per CDA guidelines (CDA 2013). Spillway designs are presented with associated pond designs in the following sections.

Spillway sizing was performed by routing the IDF through respective ponds using the available pond capacity upstream of the spillway inlet (i.e., stage-storage curve) and the following trapezoidal spillway equation (Smith 1985):

$$Q = 1.70BH^{\frac{3}{2}} + 1.27H^{\frac{5}{2}} \tan\left(\frac{\phi}{2}\right) \quad \text{Equation 1}$$

Where B is the width of the spillway, H is the height of water over the spillway invert and $\phi/2$ is the angle of the side slope relative to the vertical, for which a channel with side slopes of 2:1, $\tan(\phi/2)$ will have a value of 2.

Spillway designs are discussed in subsequent sections in relation to the associated structures.

4.3.4 Freeboard Evaluation

A wind set-up and wave run-up analysis was performed to establish minimum freeboard requirements for the management pond dams and Tenas Control Pond dam during normal operating conditions and during the passage of the IDF (CDA 2013).

For a Very High dam consequence, CDA (2013) recommends:

- No overtopping by 95% of the waves generated by a 1:1,000-year sustained wind when the pond is at its maximum normal elevation (normal freeboard)
- No overtopping by 95% of waves generated by a 1:2-year sustained wind when the pond is at its maximum extreme level during the passage of the IDF

Wind-generated wave heights were evaluated for hourly winds recorded at the Environment Canada Smithers Airport station. A correction factor of 1.2 was applied to wind data to account for the difference between overland and overwater conditions (CDA, 2007). Wave generation and wave run-up were calculated following procedures presented in USACE (2003) and CDA (2007).

Table 4-14 summarizes the input parameters and calculated normal and minimum freeboard heights for each facility. Freeboard depths are applied to the maximum operating level and water level during

routing of the IDF in Section 4.6.2 for Management Pond dams and in Section 4.5.8 for the Tenas Control Pond dam.

Table 4-14: Tenas Project Dams Freeboard Summary

Pond	Scenario	Design Wind Speed (m/s)	Maximum Fetch Length [m]	Average Pond Depth [m]	Wave Runup [m]	Wind Setup (m)	Wind Freeboard (m)
Tenas Control Pond	1 in 1000 Wind at Max Operating Level	30.5	1000	3.5	0.74	0.05	0.79
	1 in 2 Wind During Passage of the IDF	15.1			0.36	0.02	0.38
West MP	1 in 1000 Wind at Max Operating Level	30.5	1440	3.5	0.91	0.08	0.99
	1 in 2 Wind During Passage of the IDF	15.1			0.45	0.02	0.47
East MP	1 in 1000 Wind at Max Operating Level	30.5	1860	2.7	1.1	0.1	1.2
	1 in 2 Wind During Passage of the IDF	15.1			0.52	0.03	0.56
North MP	1 in 1000 Wind at Max Operating Level	30.5	720	2.6	0.61	0.05	0.66
	1 in 2 Wind During Passage of the IDF	15.1			0.3	0.01	0.31

Source: Z:\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Freeboard\FreeboardAnalysis_TelkwaEA_04142021.xlsx

4.3.5 Dam Slope Stability

Slope stability for each water containment dam was evaluated in the Tenas Project Slope Stability Assessment Report (SRK 2021e), including each of the management pond dams and the Tenas Control Pond dam.

The results of the analyses showed that the TCP, NMP, EMP and WMP configurations meet or exceed Telkwa's internal requirement and CDA (2014) design criteria for dam stability factors of safety.

4.4 Sedimentation Ponds

There are ten sedimentation ponds at the Tenas Project. Each pond was designed in accordance with the BC Ministry of Environment (BC MOE) Technical Guidance (BC MOE 2015) to settle total suspended sediments and thereby improve water quality at the discharge point.

Table 4-15 presents the objectives for each pond and the routine and emergency discharge approach. Consequence classifications are summarized in Section 4.3.1.

Table 4-15: Sediment Control Pond Objective and Discharge Summary

Pond	Objective	Discharge Approach
Tenas Control Pond	Provides sediment retention for Project footprint from early mining to closure. Provides water supply to Coal Processing Plant (CPP) and to the Management Ponds year-round	Pumped discharge to Goathorn Creek tributary (PMP_SYS-01). Overflow via spillway to channel that connects to Goathorn Creek tributary. Pumping systems to CPP (designed by others) and to Management Ponds (PMP_SYS_02)
CPP Sedimentation Pond	Provides sediment retention for runoff from CPP area	Discharge via pumped pipeline or emergency spillway to unnamed tributary of Four Creek. Optional pumping system to Tenas Control Pond (PMP_SYS-04)
Rail Infrastructure Sedimentation Pond	Provides sediment retention for runoff from Rail Infrastructure area, which includes processed coal stockpiles and a topsoil stockpile on a pad	Discharge via pumped pipeline or emergency spillway to unnamed tributary of Bulkley River.
Explosive Facility Sedimentation Sump	Provides sediment retention for runoff from Explosive's pad. The pad itself will be a 100 m x 100 m raised rock area	Discharge via pumped pipeline or emergency spillway to unnamed tributary of Tenas Creek.
Explosive Magazine Sedimentation Sump	Provides sediment retention for runoff from Explosives Magazine pad. The pad itself will be of a 100 m x 50 m raised rock area	Discharge via pumped pipeline or emergency spillway to unnamed tributary of Tenas Creek.
Rock Quarry Sedimentation Pond	Provides sediment retention for runoff from the Gravel Quarry. During heavy rain events, quarry operation will cease.	Discharge via a pumped pipeline or emergency spillway to Helps Creek
Gravel Quarry 1 Sedimentation Pond	Provides sediment retention for runoff from the Gravel Quarry 1. During heavy rain events, quarry operation will cease.	Discharge via a pumped pipeline to Goathorn Creek
Gravel Quarry 1 Sedimentation Pond	Provides sediment retention for runoff from the Gravel Quarry 2. During heavy rain events, quarry operation will cease.	Discharge via a pumped pipeline to Goathorn Creek
East TAC Sedimentation Pond	Provides sediment retention for runoff from the TAC between Goathorn Creek Bridge and top of valley on the east side	Discharge via a discharge spillway to Goathorn Creek
West TAC Sedimentation Pond	Provides sediment retention for runoff from the TAC between Goathorn Creek Bridge and top of valley on the west side	Discharge via a discharge spillway to Goathorn Creek

Source: Prepared in text

TAC = Tenas Access Corridor

Rock pads are to be constructed with non-potentially acid generating (NAG) rock or overburden, and further treatment of discharge beyond total suspended solids (TSS) is not anticipated. Based on geochemical characterizations presented in the geochemistry report (SRK 2021d), the temporary coal stockpiles are not expected to oxidize and produce acidic and/or water with elevated metal

concentrations. Water quality predictions from the Rail Infrastructure Pond and the Tenas Control Pond are presented in the Water and Load Balance Report (SRK 2021b).

TSS is removed by gravity settling and/or addition of flocculent, if required. Flocculent addition is a contingency measure and has not been included in the design. Sediment will accumulate within the pond and will be removed periodically so that the ponds maintain full design capacity for effective treatment. Each pond includes a maintenance access ramp for regular sediment removal.

Sediment control ponds will discharge to the environment via an engineered spillway and a pumping system. The CPP Sedimentation Pond will also be equipped with a pumping system to route collected runoff to the Tenas Control Pond in case water quality is not suitable for direct discharge because of spills around the CPP. Excavated ponds will be lined with coarse gravel to isolate the moving water from exposed soils and reduce erosion and mobilization of sediment during operations.

Design criteria for pond design are summarized in Table 4-16.

Table 4-16: Design Criteria for Sediment Control Ponds

Parameter	Value
Minimum Inflow Design Flood (IDF) for ponds	24-hour duration 1 in 200-year flood event (rain + snowmelt)
Minimum particle for pond retention	10 micron
Minimum pond retention capacity	1 in 10-year 24-hour flood event
Particle settling velocity	2×10^{-5} m/s
Correction factor for non-ideal settling	1.2
Target length to width ratio	5:1
Minimum pond depth below orifice outflow	0.5 m
Minimum spillway base width	2.0 m
Spillway side slopes	2:1 H:V
Excavation side slopes	2:1 H:V
Minimum freeboard	0.5 m during IDF
Minimum pond depth	1.5 m

Source: Prepared in text

The following sections outline the design details for the sediment control ponds. The Tenas Control Pond is discussed separately in Section 4.5.

4.4.1 Sedimentation Pond Design Methodology

BC MOE recommends sedimentation ponds should be able to provide sufficient retention time to settle a 10-micron particle. The required retention time to settle a 10-micron particle with a settling velocity of 2×10^{-5} m/s can be evaluated given the following equation:

$$Tr = d/(3600V) \quad \text{Equation 2}$$

Where Tr is the retention time in hours, d is the pond depth in meters and V is the settling velocity in m/s. A settling velocity of 2×10^{-5} m/s was selected which coincides with a fine silt and is based on BC MOE recommendations where particle size information is not available. The minimum pond surface area was calculated based on Equation 3 below, which incorporates a correction factor of 1.2 (BC MOE 2015):

$$\text{Min Surface Area} = \frac{Q_{peak}^{10\text{-year event}}}{1.2} (V) \quad \text{Equation 3}$$

Sedimentation ponds will operate with a pumping system for normal operation and an emergency spillway. The pumping system will manage the discharge of water from the pond for events up to and including the 1 in 10-year event. The emergency spillway will convey events greater than the 1 in 10- year event up to the IDF, with a return period of 1 in 200-years.

Additional assumptions for sedimentation pond design are as follows:

- The 1 in 10-year flood event and the 1 in 200-year flood event were modelled in HEC-HMS using peak flow inputs defined in Section 4.1 with climate change-adjusted precipitation depths and snowmelt contributions. A CN of 98 was applied for mine facility pads, including the CPP area upstream of the CPP Sedimentation Pond, the Rail Infrastructure pad, the Explosives Magazine pad, and the Explosives Facility pad. Natural catchments incorporate CN values calibrated for each return period (Section 4.1) and account for AMC III ground conditions during the IDF
- Initial conditions during the 1 in 10-year flood event assume that the pond water level is equal to the pump invert elevation, which is set above a minimum depth of 0.5 m above the base of the pond for sediment accumulation
- Ponds are excavated in natural ground for containment without ponding water against a berm or dam
- The pond width is one fifth of the length
- The pond base length was adjusted to make the pond surface at the pump intake elevation greater than the minimum pond surface area for particle settling, as dictated by Equation 3
- Two baffles were included in the CPP Sedimentation Pond to better fit the topography and increase the flow length for particle settling
- The pumping rate and pond volume were established to meet the minimum retention time value from Equation 2, while reducing the total pond volume based on routing of the 1 in 10-year event. The spillway invert was set at the peak water level during the 1 in 10-year event. Initial conditions

for emergency spillway design assumed that the pond water level is at the spillway invert elevation at the onset of the IDF

- The top of excavation (or top of pond) elevation was selected based on the maximum water level through the spillway during the IDF plus the minimum freeboard of 0.5 m
- Each pond includes an 8H:1V access ramp to the base of the pond for maintenance access and may be drained as needed using the pumping system

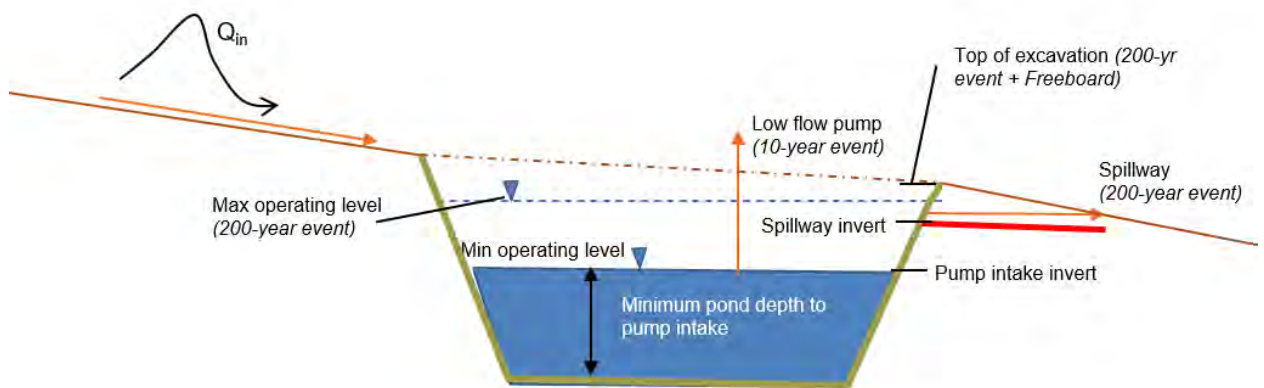


Figure 4-2: Typical Sediment Control Pond Geometry

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Ponds\Telkwa – Pond Designs_v04.pptx

4.4.2 Sedimentation Pond Design Results

The hydrologic analysis, pond geometry and sedimentation calculations and results are summarized in Table 4-17, Table 4-18, and Table 4-19, respectively. Catchment areas showing maximum areas contributing to the main ponds are presented in Figure 4-3, including the Rail Infrastructure, CPP and Explosives ponds, as well as the Tenas Control Pond. Catchment areas for the quarry ponds (rock quarry and gravel quarry 1 and 2) and TAC ponds 1 and 2 were provided by TCL.

Sedimentation pond operation and monitoring requirements are described in the Tenas Project Water Monitoring Plan and include:

- Daily water level readings for ponds
- Pumping rates should be measured and recorded with a totalizer. Pump controls may be manual or automatic, depending on the selected configuration and pump design
- Monthly water quality sampling for ponds for full suite of water quality parameters, with more frequent measurements of TSS concentrations proceeding storm events to ensure ponds are working as intended
- Annual sediment accumulation measurements

Plan view and typical sections of the CPP, Rail Infrastructure, Explosive Facility and Explosive Magazine Sedimentation ponds are presented in Drawings 14, 15 and 16. The geometry concepts presented for the Explosive Magazine and Explosive Facility Sedimentation ponds will be applied to other sedimentation ponds.

Table 4-17: Sedimentation Pond Hydrologic Analysis Summary

Pond	CPP Sedimentation Pond	Rail Infrastructure Sedimentation Pond	Explosive Facility Sedimentation Pond	Explosive Magazine Sedimentation Pond	Rock Quarry Sedimentation Pond	Gravel Quarry 1 Sedimentation Pond	Gravel Quarry 2 Sedimentation Pond	East TAC Sedimentation Pond	West TAC Sedimentation Pond
Maximum Catchment Area [m²]	203,500	77,900	10,000	5,000	50,000	15,000	80,000	20,000	10,000
1 in 10-year Peak Inflow [m³/s]	1.8	0.5	0.1	0.1	0.7	0.2	1.1	0.3	0.1
1 in 10-year 24-hour Storm Volume [m³]	19,550	7,440	960	480	4,820	1,450	7,720	1,930	960
1 in 10-year Max water level [m]	2.5	2.6	2.5	2.4	2.5	1.2	2.4	1.6	0.9
1 in 10-year Peak Outflow [m³/s]	0.090	0.030	0.008	0.006	0.010	0.010	0.010	0.010	0.010
1 in 200-year Peak Inflow [m³/s]	2.9	0.8	0.2	0.1	1.2	0.4	2.0	0.5	0.2
1 in 200-year 24-hour Storm Volume [m³]	29,000	10,960	1,440	720	7,190	2,160	11,510	2,880	1,440
1 in 200-year Max water level [m]	2.8	2.7	2.6	2.5	2.7	2.5	2.6	2.5	2.5
1 in 200-year Peak Outflow [m³/s]	2.6	0.8	0.2	0.1	1.0	0.2	1.9	0.4	0.0

Source: https://srk.sharepoint.com/sites/NA1CT027.003/Internal/400_Water_Management/Ponds/PondDesign_rev07_20220324_sab.xlsm

Table 4-18: Sediment Control Pond Geometry and Design Summary

Pond	CPP Sedimentation Pond	Rail Infrastructure Sedimentation Pond	Explosive Facility Sedimentation Pond	Explosive Magazine Sedimentation Pond	Rock Quarry Sedimentation Pond	Gravel Quarry 1 Sedimentation Pond	Gravel Quarry 2 Sedimentation Pond	East TAC Sedimentation Pond	West TAC Sedimentation Pond
Base length [m]	43	100	50	30	80	50	115	50	50
Number of baffles	2	0	0	0	0	0	0	0	0
Base width [m]	78	20	10	6	16	10	23	10	10
Depth from base of pond to pump intake [m]	0.5	0.5	2	2	0.5	0.5	0.5	0.5	0.5
Depth from base of pond to spillway [m]	2.6	2.7	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Depth from base of pond to top of excavation [m]	3.3	3.3	3.1	3.1	3.3	3.0	3.1	3.0	3.0
Spillway base width [m]	10	10	2	2	2	10	10	10	10
Spillway side slopes [H:V]	2	2	2	2	2	2	2	2	2
Pumping rate [m³/s]	0.090	0.030	0.008	0.006	0.010	0.010	0.010	0.010	0.010
Pond volume below pump intake [m³]	2,570	870	1,510	680	690	280	1,390	280	280
Pond volume below spillway [m³]	16,780	6,550	2,060	970	4,460	2,060	8,390	2,060	2,060
Total pond volume [m³]	22,290	8,400	2,800	1,330	6,310	2,690	10,910	2,690	2,690

Source: https://srk.sharepoint.com/sites/NA1CT027.003/Internal/400_Water_Management/Ponds/PondDesign_rev07_20220324_sab.xlsm

Table 4-19: Sediment Control Pond Sedimentation Calculation Summary

Pond	CPP Sedimentation Pond	Rail Infrastructure Sedimentation Pond	Explosive Facility Sedimentation Pond	Explosive Magazine Sedimentation Pond	Rock Quarry Sedimentation Pond	Gravel Quarry 1 Sedimentation Pond	Gravel Quarry 2 Sedimentation Pond	East TAC Sedimentation Pond	West TAC Sedimentation Pond
Minimum pond surface area for settling [m²]	5,400	1,800	480	360	600	600	600	600	600
Design pond surface area at orifice level [m²]	5,450	1,940	1,040	530	1,480	620	2,930	620	620
Minimum retention time [hr]	7	7	28	28	7	7	7	7	7
Design retention time during 1 in 10-year event [hr]	8	8	52	32	19	8	39	8	8

Source: https://srk.sharepoint.com/sites/NA1CT027.003/Internal/400_Water_Management/Ponds/PondDesign_rev07_20220324_sab.xlsm

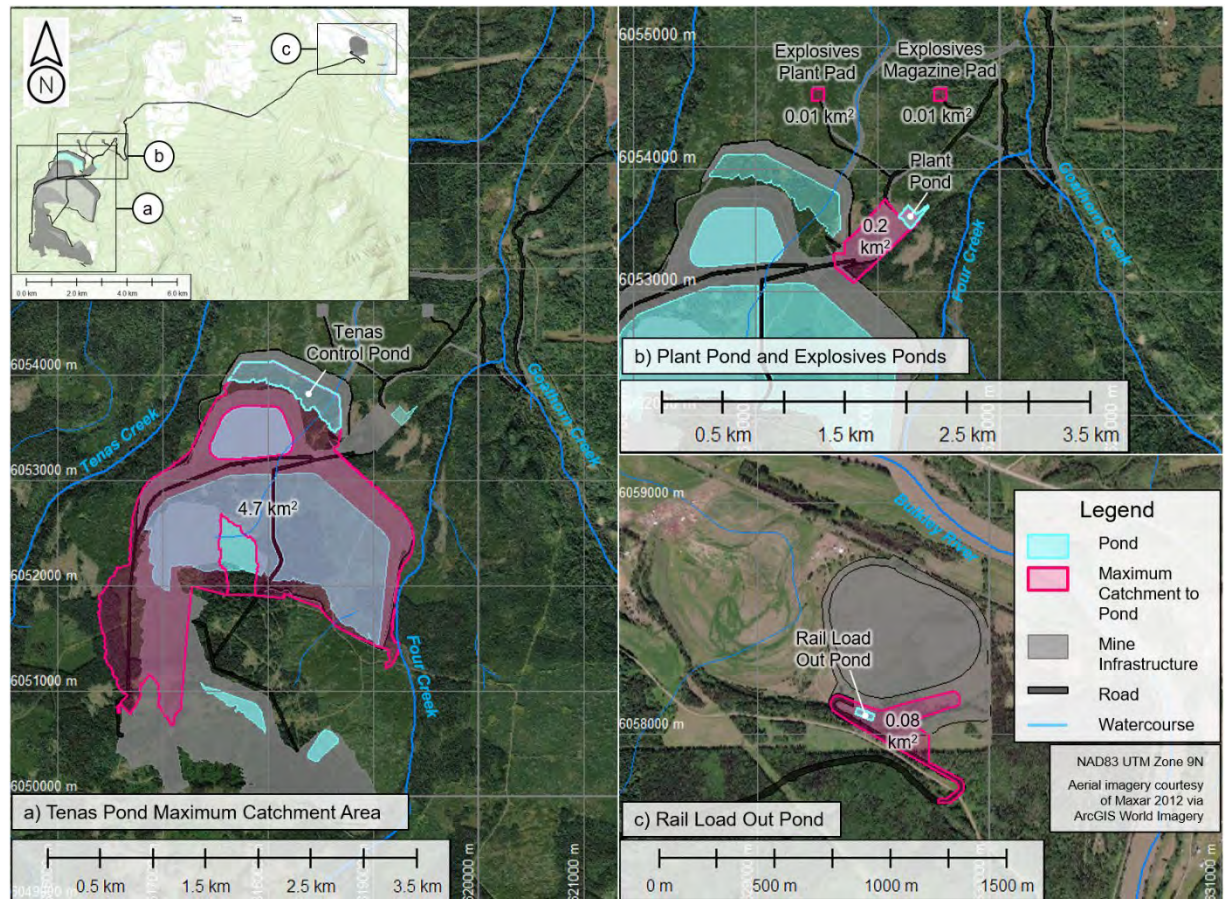


Figure 4-3: Sedimentation Pond Catchment Area Map for (a) Tenas Control Pond, (b) Plant and Explosives areas Sedimentation Ponds and (c) Rail Infrastructure Sedimentation Pond

4.5 Tenas Control Pond

The Tenas Control Pond will operate from construction through decommissioning and reclamation and for a period in post-closure, providing sediment control and water supply to the Tenas Project. During construction, Best Management Practices coupled with a seepage collection channel along the toe of the final dam will be in-place to manage sediment. Once reclamation of the site is complete, the Tenas Control Pond will be dewatered to Goathorn Creek, and the dam will be breached. The erosion protection at the toe of the dam will be extended south towards the outlet of the North Management Pond spillway to convey Project discharge post-closure through the former Tenas Control Pond area and towards Goathorn Creek (as described in Section 4.2.2).

Tenas Control Pond could also be used to provide temporary water storage in the event water quality is unacceptable for direct discharge, while a response is being mounted.

The stage-storage relationship for the pond is presented in Appendix A.

The following sections summarize the design of the Tenas Control Pond.

4.5.1 Conceptual Model and Design Criteria

In addition to sedimentation control, the Tenas Control Pond has the following design objectives:

- Provide water supply to the CPP, including maintaining a contingency storage volume equivalent to 7 months' operational demand during a 1 in 10-year dry year
- Provide water supply to the Management Ponds to maintain water covers over PAG materials, conservatively assuming there is no other source of flow to the ponds
- Maintain minimum discharge during the months of May through October to mix with treated effluent from the sewage treatment plant

The conceptual model and design criteria for the Tenas Control Pond are presented in Figure 4-4 and Table 4-20. Parameters were developed iteratively using a monthly water balance, flood routing and sedimentation design. The following sections describe the Tenas Control Pond water balance and associated designs.

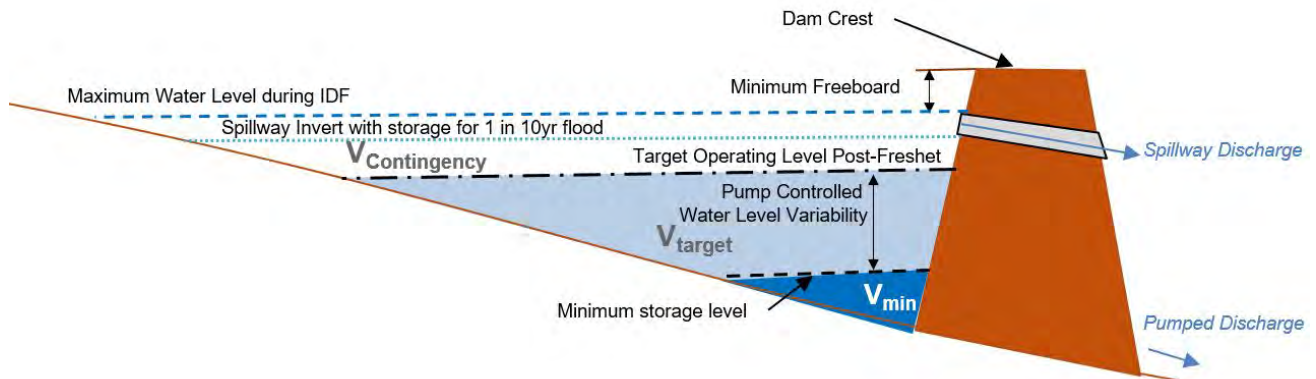


Figure 4-4: Tenas Control Pond Conceptual Design Schematic

Table 4-20: Tenas Control Pond Design Criteria

Criteria	Description
Minimum Storage Volume, V_{min}	7 months of process water supply + 2.5 m of storage for sediment accumulation (Section 4.5.3)
Maximum Target Operating Volume Post-Freshet, V_{Target}	Required water inventory as of July 1 st , as determined by 1 in 10-year dry year water balance (Section 4.5.4)
Contingency Storage, $V_{Contingency}$	Contingency storage capacity for 1 in 10-year 24-hour storm event above the normal maximum operating water level (Section 4.5.6) to provide operational flexibility in terms of discharge rates to Goathorn Creek
Pumping Rate	Mean annual runoff inflows – Annual process water demand, discharged over 5-month open-water period (Section 4.5.5). Considers flood recovery for 1:10-year inflow over 7-days (using backup pump system)
Spillway Invert	Based on stage-storage curve, providing storage volume greater than the sum of V_{target} and V_{10yr} (Section 4.5.7)
IDF Maximum Water Level	Based on routing of the Inflow Design Flood through the spillway (Section 4.5.8)
Minimum Freeboard	Based on freeboard evaluation (Section 4.3.4)
Dam Crest	Selected based on the minimum freeboard and IDF maximum water level
Sediment Removal	Calculated based on the retention time during the 1 in 10-year inflow and expected pond volume, and compared against the minimum retention time based on pond depth and particle settling velocity (Equation 2 in Section 4.4.1)
Pond Surface Area at V_{min}	Calculated from stage-storage curve and compared against minimum pond surface area for settling (Equation 3 in Section 4.4.1)

4.5.2 Operational Water Demand

The CPP water demand is approximately 15,000 m³/month based on the processing rate of 1.05 million tonnes per annum and a water demand of 130 litres per tonne, and an allowance of 3,625 m³/month for wash bays and dust suppression (rates provided by TCL).

A minimum discharge rate is required to mix with sewage treatment plant effluent from May through September, equal to 8,300 m³/month (50,000 m³/year).

Management ponds are required to maintain a minimum water cover above stored PAG material to reduce the risk of oxidation. Water supply requirements for the management ponds vary throughout the life of mine and are expected to increase with increasing PAG material volumes. The management

ponds collect flows from active open pit mining and natural runoff from undisturbed catchment areas. Any deficits are made up by pumping from the Tenas Control Pond.

The Tenas Control Pond design includes sufficient water supply to maintain submergence of PAG materials at the maximum projected placement rate, assuming no other sources of water are available. The maximum PAG placement rate coincides with Year 5 from the mine plan, which is expected to produce 1.98 Mm³ of PAG material and 0.18 Mm³ of processed rock, resulting in a total of 2.16 Mm³ of material into the East Management Pond. A void ratio of 0.3 is assumed for mine rock, which would require a volume of water equal to 0.65 Mm³ to saturate the added material. A factor of safety of 1.1 was applied to this volume, and this water volume was assumed to be required evenly throughout the year, resulting in a monthly water demand of approximately 60,000 m³/month.

4.5.3 Minimum Process Water Storage Volume

The minimum storage volume for the Tenas Control Pond was selected in order to provide contingency water supply to the CPP for a 7-month period, calculated as 105,000 m³ assuming no other sources of inflow are available to the CPP, in addition to an allowance of 2.5 m (approximately 5,000 m³) for sediment accumulation (Appendix A). The total minimum storage volume (V_{min}) selected for the Tenas Control Pond is 110,000 m³.

If sediment accumulation exceeds the volume threshold, floating dredge infrastructure would be acquired for sediment removal. Additional measures to reduce sediment accumulation at the upstream toe of the Tenas Control Pond dam include check dams at the outlets of C-03a and C-01 as they discharge towards Tenas Control Pond from the west and east Project extents respectively, acting as primary settling ponds for coarse sediment removal.

4.5.4 Monthly Water Balance for Target Operating Level

A monthly water balance was prepared for the Tenas Control Pond to establish the target operating post-freshet water level required to maintain the minimum process water storage volume throughout the year during a 1 in 10-year dry year. The water balance is based on the critical period with maximum management pond water demand (Year 5, see Section 4.5.2).

Monthly inflow rates to the Tenas Control Pond were calculated based on the Project monthly runoff distribution, as presented in Section 2.4, and a Year 5 catchment area of 5.92 km² which includes runoff from the management pond dams and buttresses, runoff into the management ponds themselves, runoff to the CPP pad and flows pumped from the CPP Sedimentation Pond and portions of the active open pit, as shown in Figure 4-5.

In addition to the operational water demands outlined in Section 4.5.2, additional losses from the system include evaporation and seepage. Project runoff rates account for losses to natural surfaces including evapotranspiration and groundwater infiltration. Additional evaporation losses were not included in the Tenas Control Pond water balance. However, the added head from Management Ponds and the Tenas Control Pond will increase groundwater infiltration losses over natural conditions.

Seepage losses were included based on results from the groundwater model and expected liner performance, which result in a maximum seepage loss of 1,600 m³/day (SRK, 2021b).

Monthly water balance results are presented in Table 4-21. A minimum post-freshet pond volume of 850,000 m³ at the beginning of July is required to maintain a minimum volume of 110,000 m³ throughout the 1 in 10-year dry condition. This volume becomes the target maximum operating volume after freshet each year, corresponding to a water level of 797.1 masl, to provide adequate water supply in the case of a dry year (V_{target}).

Since the minimum pond volume provides sufficient water to maintain process water demand for an additional 7 months, the water balance results and target operating conditions for the Tenas Control Pond are conservative, allowing for uncertainty in evaporation rates, seepage rates, or more extreme dry conditions. Any excess water will be discharged to the environment, mimicking the natural hydrograph where possible. A detailed Operation, Maintenance and Surveillance (OMS) plan will be developed for the Tenas Control Pond in the Detailed Engineering phase, which will assess the system under a wider range of conditions and look for opportunities to optimize the pond storage requirements.

Table 4-21: Tenas Control Pond Monthly Water Balance for 1 in 10-year Dry Condition in Mine Year 5, during Critical PAG Placement Period

Month	Monthly Runoff [mm]	Pond Flow Rates [m ³ /month]						End of Month Pond Volume [m ³]
		Runoff Inflow volume	CPP Water Demand	Management Pond Demand	Minimum Discharge	Seepage Loss	Net Inflow	
Jul	12	69,200	-15,000	-62,000	-8,300	-49,600	-67,400	782,600
Aug	9	52,400	-15,000	-62,000	-8,300	-49,600	-84,200	698,300
Sep	10	58,000	-15,000	-60,000	-8,300	-48,000	-75,000	623,300
Oct	14	81,800	-15,000	-62,000	-8,300	-49,600	-44,800	578,500
Nov	10	58,800	-15,000	-60,000	0	-48,000	-64,200	514,300
Dec	4	24,400	-15,000	-62,000	0	-49,600	-102,200	412,100
Jan	3	20,300	-15,000	-62,000	0	-49,600	-106,300	305,800
Feb	4	21,700	-15,000	-56,000	0	-44,800	-94,100	211,700
Mar	6	32,900	-15,000	-62,000	0	-49,600	-93,700	118,000
Apr	27	159,600	-15,000	-60,000	0	-48,000	36,600	154,500
May	60	355,600	-15,000	-62,000	-8,300	-49,600	219,000	373,600
Jun	19	112,700	-15,000	-60,000	-8,300	-48,000	-20,300	353,300

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Ponds\PondDesign1_rev05_sab.xlsm

Note: Initial pond volume at start of July is 850,000 m³

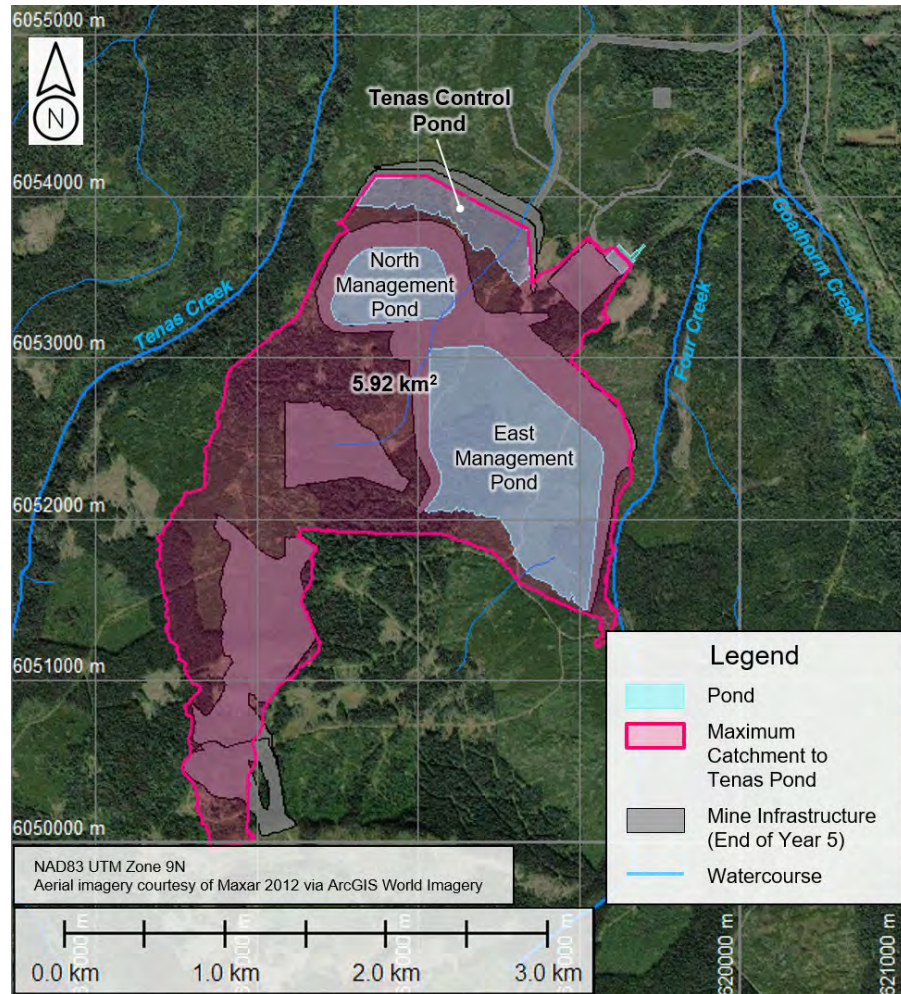


Figure 4-5: Tenas Control Pond Catchment Area, Year 5, during Critical PAG Placement Period

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\1040_AutoCAD\4.GMW\PondDesign\EA_PondDesign_CatchmentCompilation.gmw

4.5.5 Pumping Rate

The Tenas Control Pond includes two pumping systems to discharge water either to Goathorn Creek tributary (PMP_SYS-01) or to the upstream management ponds (PMP_SYS-02). Pumping systems are described in Section 4.8. The design flow rate for both systems was selected to discharge the mean annual runoff inflow over a 5-month period (May through September). This flow rate was calculated based on Equation 4:

$$Q_{\text{Pumping Rate}} = \frac{MAR \times \text{Area} - \text{Annual CPP Water Demand}}{5 \text{ months}} \quad \text{Equation 4}$$

Where the pond catchment area is presented in Figure 4-6, *MAR* is the mean annual runoff depth of 219 mm presented in Table 2-9, and the annual CPP water demand is 180,000 m³ based on the water balance in Table 4-21. The average pumping rate was calculated to be 267,350 m³/month, or 0.1 m³/s (including a factor of safety of 1.2). This discharge rate also meets the minimum discharge requirement stipulated by sewage treatment plant of 8,300 m³/month, as described in Section 4.5.2 and in (Radloff, 2021).

The pumping system must also meet the selected performance standard in relation to recovery after a flood event. A flood recovery target of 7 days was selected, during which sufficient pump capacity is required from the Tenas Control Pond to discharge the 1 in 10-year event inflow volume. Based on the event volume of 119,000 m³ and the pump capacity of 0.1 m³/s, two parallel pumping systems will be required to dewater the inflow volume with 7 days. Since two systems will be in-place throughout operations to send water to both the management ponds and to Goathorn Creek, the flood recovery performance standard will be achieved. In the event of a flood, the pumping system from the Tenas Control Pond to the management pond (PMP_SYS-02) would be temporarily directed to discharge to Goathorn Creek in parallel to PMP_SYS-01. It is recommended that a back-up system from Tenas Control Pond to Goathorn Creek (PMP_SYS-01) be available on-site throughout operations in case of pump malfunction.

This pumping rate excludes CPP water demand, which will operate on a separate system.

4.5.6 Sedimentation Capacity

The minimum available retention time in the Tenas Control Pond for sedimentation was estimated as 302 hours based on the minimum pond volume and typical pumped discharge rate (110,000 m³ and 0.1 m³/s, see Section 4.5.3 and 4.5.5), which exceeds the minimum time required based on the criteria described in Section 4.4.1 (Equation 1) of 84 hours, calculated using a depth of 6.1 m corresponding to the minimum storage volume. If the minimum storage volume is depleted, sedimentation criteria will still be met up to a minimum operating elevation of 789.2 masl (equivalent to approximately 20,000 m³ of storage).

4.5.7 Contingency Storage and Pond Spillway Invert

A contingency storage volume was selected to provide operational flexibility to control the rate of discharge to Goathorn Creek. The contingency storage volume was selected to be the volume associated with the 1 in 10-year flood event.

The inflow volume is based on the maximum direct pond catchment area of 4.69 km², which occurs in Year 1 during operations of the North Management Pond and construction of the East Management Pond (Figure 4-3). At this time, the C-02 channel will be constructed, diverting upstream runoff towards Four Creek, which was designed to convey up to the 200-year return period flows. This area is smaller than the catchment used for the water balance since it does not include runoff contributing to upstream management ponds or the open pit areas, which would attenuate flows during a short-duration runoff event but contribute to monthly inflow volumes with pumps in place.

Based on the maximum direct pond catchment area and the hydrology assumptions described in Section 4.1, the 1 in 10-year flood volume was calculated to be 119,000 m³.

The spillway invert elevation was selected as the sum of the 1 in 10-year flood volume and the target operating post-freshet volume, calculated to be 850,000 m³ (Section 4.5.4). Based on the combined storage volume of 969,000 m³, the corresponding water level from the stage-storage curve in Appendix A is 797.6 masl. A spillway invert of 798 masl was selected, which increases the contingency storage to 210,000 m³ and providing a combined total storage capacity of 1,060,300 m³.

4.5.8 Spillway Sizing and Freeboard

The emergency spillway was designed to convey the IDF based on the Tenas Control Pond dam classification while maintaining the minimum freeboard requirements defined in Section 4.3.4. The IDF was selected to be 2/3 between the 1,000-year flood and the PMF, based on a CDA (2013) dam consequence classification of Very High. Both the spring snowmelt-dominated PMF and the summer rainfall-dominated PMF events were modelled in HEC-HMS, as described in Section 4.1. It was assumed that upstream infrastructure, including the C-02 channel, have failed in the event of the IDF. The 200-year return period calibrated CN value of 70.1, for AMC III conditions, was applied to natural catchments (Table 4-1), and a CN of 99 was applied any potentially ponded area and a CN of 98 for pads.

Two scenarios were considered to estimate the IDF – End of Year 1 (EOY01) and End of Mine (EOM), as shown in Figure 3-4 and Figure 3-11 respectively. The catchment area for the EOY01 phase during operations is 8.25 km² (including North management Pond), which assumes C-02 has failed, where direct runoff occurs from the sub-catchments except the North Management Pond area which is attenuated by the pond itself. This phase resulted in higher peak flow results compared to EOM and was considered the critical phase for the Tenas Control Pond spillway sizing. The resulting IDF was routed assuming that the initial water level is at the target maximum operating level of the Tenas Control Pond (797.1 mas).

Figure 4-6 presents the inflow hydrograph with the spring IDF, which produces a marginally higher peak and volume than summer IDF, as well as the outflow hydrograph through the spillway and water level of the Tenas Control Pond.

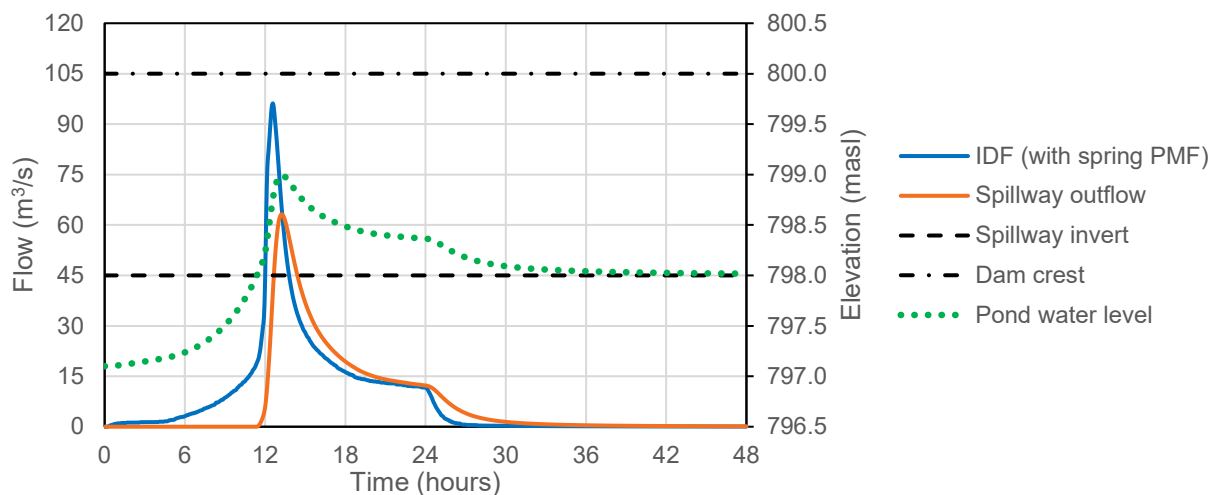


Figure 4-6: Routing of the IDF through the Tenas Control Pond Spillway

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Spillways\Telkwa_SpillwayDesign_rev08_rk

A spillway base width of 30 m was selected to achieve the minimum freeboard requirements with a dam crest elevation of 800 masl.

Erosion protection for the spillway was evaluated with a 1% longitudinal slope through the dam, and a 33% (3:1 H:V) slope along the downstream face of the embankment. Channel velocities were calculated iteratively based on calculated Manning's roughness values and the selected riprap size, as described in Section 4.2.1 for engineered channels.

Results for spillway design, including riprap, along with freeboard evaluation are presented in Table 4-22.

Table 4-22: Tenas Control Pond Spillway and Freeboard Results

Parameter	Result
IDF Peak Inflow [m ³ /s]	96.2
IDF Peak Outflow through spillway [m ³ /s]	63.2
Spillway Base Width [m]	30
IDF Maximum Water Level [masl]	799.0
Minimum Freeboard [m]	0.38
Available Freeboard during IDF [m]	1.0
Velocity during IDF through Spillway [m/s]	2.22
Spillway Riprap D ₅₀ [m]	0.15
Velocity during IDF along Embankment Channel [m/s]	5.30
Embankment Channel Riprap D ₅₀ [m]	0.65

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Spillways\Telkwa_SpillwayDesign_rev08_rk

4.5.9 Summary of Tenas Control Pond Design

Table 4-23 summarizes design for the Tenas Control Pond.

Table 4-23: Summary Tenas Control Pond Design

Hydrologic Design			
1 in 10-year Peak Inflow [m ³ /s]	6.4	IDF Peak Inflow [m ³ /s]	97.0
1 in 10-year Storm Volume [m ³]	118,890	IDF Storm Volume [m ³]	1,448,730
1 in 10-year Max Water Level [m]	793.0	IDF Max Water Level [m]	798.8
1 in 10-year Peak Outflow [m ³ /s]	0.10	IDF Peak Outflow [m ³ /s]	74.2
Sedimentation Design			
Required Retention Time [hrs]	84	Available Retention Time [hrs]	302
Required Pond Surface Area ⁽¹⁾ [m ²]	6,067	Available Pond Surface Area [m ²]	61,400
Pond Operating Volumes			
Minimum Storage Volume, V _{min} [m ³]	110,000	Maximum Target Operating Volume, V _{Target} [m ³]	850,000
Contingency Storage Volume, V _{10yr} [m ³]	210,300	Maximum Target Operating Level, [masl]	797.1
Pump Intake Elevation [masl]	792	Pumping Rate [m ³ /s]	0.10
Spillway Invert [masl]	798	Total Storage Capacity to Spillway [m ³]	1,060,300
Dam Crest Elevation [masl]	800	Total Capacity to Dam Crest [m ³]	1,611,100

Hydrologic Design			
Spillway Design			
Spillway Base Width [m]	30.0	Spillway Side Slopes (H:V)	10:1
Spillway Riprap D ₅₀ [m]	0.15	Embankment Channel Riprap D ₅₀ [m]	0.65
Minimum Required Freeboard during IDF [m]	0.38	Available Freeboard during IDF [m]	1.0

Notes:

¹ Minimum and available pond surface areas are based on the lowest expected pond levels, which coincide to the minimum pond volume of 110,000 m³ at the pump intake elevation of 792 masl

4.6 Management Ponds

4.6.1 Design and Objectives

The purpose of the management ponds is to mitigate ML/ARD at the Project by maintaining a water cover above the impounded processed and unprocessed mine rock elevation, reducing the potential for oxidation of PAG materials. The effectiveness of sub-aqueous disposal of the PAG material is described in the water quality model (SRK 2021b) and the ML/ARD management plan (SRK 2021i).

There are three management ponds for the Tenas Project: North Management Pond, East Management Pond, and West Management Pond. Stage-storage relationships are presented in Appendix A.

The management ponds are partially lined storage facilities impounded by rockfill and overburden embankments (Section 4.3.5). The base of each facility along with the upstream dam face will be lined, reducing seepage loss to groundwater. Buttresses will be constructed along the toe of each dam over time as suitable mine rock becomes available (Section 1.1 Description of Proposed Project in the EAC Application).

Management ponds are constructed in stages, beginning with the North Management Pond (NMP) in Year 0, providing containment for PAG material during year 1 and 2 of operations. The NMP dam will be a ring-dyke, located directly south of the Tenas Control Pond and north of the subsequent management ponds. As placement of PAG material proceeds in the NMP, the East Management Pond (EMP) dam is constructed, which will provide containment on three sides, with a separation dam from the final management pond to the west. PAG placement will continue in the EMP from Year 2 through 7. The West Management Pond (WMP) will consist of both an excavated cell and an embankment, encompassing the northern extents of the open pit. Staging of open pit mining will be developed such that PAG backfill and flooding in the northern area may begin without impacting mining sequence farther south. PAG placement will continue in the WMP from Year 7 through 21 (end of mining).

Processed rock consists of material that has gone through the Coal Processing plant but is not processed coal. Processed rock will follow the deposition schedule of PAG material into the management ponds, designed to maintain water cover over PAG materials.

Once PAG placement is complete within each management pond, a minimum of 1.0 m layer of Non-PAG rock will be placed, followed by a minimum of 0.3 m of overburden and topsoil. The Non-PAG rock, overburden, and soil covers are intended to reduce evaporative losses from the facilities, while initiating progressive closure by returning the covered area to a similarly forested land use to pre-mining conditions. Soil covers will be graded towards the spillway locations, with engineered infiltration channels designed to maximize water flowing through the mine rock and soil, promoting long-term PAG saturation. Appendix C provides additional details on the soil cover design and objectives.

Spillways were designed for management ponds based on the dam classifications presented in Section 4.3.1 and described in Appendix B. In all cases, the IDF was selected to be 2/3 between the 1,000-year return period flood and the PMF. Normal and minimum freeboards were calculated for each facility in accordance with CDA guidelines (see Section 4.3.4).

During operations, the spillway alignments of the EMP and WMP will intersect the main haul road from the open pit to the CPP. Based on the design of the management ponds, it is intended that water levels reach the spillway inverts and discharge by gravity instead of relying on pumps to facilitate drainage towards Tenas Control Pond. As such, a series of culverts will be required beneath the haul road to maintain access during normal flow conditions. Culverts are described in Section 4.9.

Once saturated, both PAG and NAG rock are assumed to be 30% water by volume while processed rock is assumed to be 15% water by volume. Table 4-24 presents the deposition schedule for PAG and processed rock, and Table 4-25 presents the deposition volumes and associated elevations for each management pond.

Table 4-24: Management Pond Deposition Schedule

Management Pond	PAG and Processed Rock Deposition (start/end)
North Management Pond	Q3 Year -1 to Q2 Year 2
East Management Pond	Q2 Year 2 to Q1 Year 7
West Management Pond	Q1 Year 7 to Q4 Year 21

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\500_Water_Balance\Inputs\MasterInputs_TelkwaEA_Rev08_sab_ccm.xlsm

Table 4-25: Management Pond Design Volumes and Elevations

Parameter	NMP	EMP	WMP
Total Saturated PAG Material Volume [m ³]	2,386,000	11,561,000	27,432,000
Total Saturated Processed Rock Volume [m ³]	344,000	986,000	3,006,000
Top of PAG Elevation [masl]	825.3	847.4	847.4
NAG Cover thickness [m]	1.5	1.5	1.5
NAG Cover Volume [m ³]	427,000	1,580,000	1,441,000
Top of NAG Elevation [masl]	826.8	848.9	848.9
Overburden Cover Thickness [m]	1.2	1.1	1.1
Overburden Cover Volume [m ³]	338,000	1,222,000	1,138,000
Dam Crest Elevation [masl]	830.0	852.0	852.0

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\500_Water_Balance\Inputs\MasterInputs_TelkwaEA_Rev08_sab_ccm.xlsm

4.6.2 Spillway Sizing and Freeboard

Management ponds will operate with an emergency spillway throughout operations and post-closure, designed in accordance with CDA requirements (Section 4.3.3). Spillway alignments were selected to facilitate water conveyance from south to north across the Project footprint, routing water towards the Tenas Control Pond. The North Management Pond spillway discharges directly into the Tenas Control Pond, while the East and West management pond spillways discharge into the North Management Pond.

Culverts will convey overflows from the EMP and WMP towards the NMP, crossing the haul road. Flood events more than the capacity of the culverts and up to the IDF will be conveyed via open-channel spillways, which will result in water flowing over the haul roads under extreme conditions. Riprap will be placed across the length of the spillway channels, with additional compacted gravel along the haul road sections to allow vehicle access. After extreme flood events, some re-grading may be required along the spillway sections over the haul road, however any mobilized material will be captured in either the NMP or the Tenas Control Pond downstream.

The management pond spillways were designed for both operational and closure conditions. Freeboard was evaluated based on CDA (2013) recommendations, as described in Section 4.3.4.

The operational spillway configuration for the EMP and WMP will consist of culverts (see Section 4.9) and an overflow channel to convey the IDF (Drawings 17 and 18). Overflow channel inverts were selected to maintain the minimum freeboard requirements while maximizing depth of flow through the culverts.

At the end of operations, the management ponds will be reclaimed with soil covers and vegetated. The haul road will be decommissioned, and culverts removed, and spillway invert elevations for both EMP and WMP will be lowered to the top of overburden. A closure-condition IDF analysis was prepared based on final catchment contributions and final spillway requirements were evaluated.

A summary of the spillway designs is presented in Table 4-26.

Table 4-26: Design Criteria for Management Pond Spillways

Parameter	North MP	East MP	West MP	Reference
Dam Classification	Very High			Dam breach assessment, Appendix B
Inflow Design Flood	2/3 between 1,000 return period event and PMF.			CDA (2007)
Freeboard	1 in 1000 wind at max operating level, and 1 in 2 wind during passage of the IDF			CDA (2007)
Operational Spillway				
Minimum Overburden Thickness between NAG Cover and Culverts	n.a.	0.5	0.5	SRK and TCL judgement
Minimum spillway base width	2.0 m	2.0 m	2.0 m	SRK judgement and for ease of construction and maintenance
Spillway side slopes	10:1 H:V	10:1 H:V	10:1 H:V	SRK judgement and to allow traffic during dry conditions
Longitudinal slope through dam	1%	1%	1%	SRK judgement
Longitudinal slope of embankment channel	33%	33%	33%	Initial dam configuration, prior to buttress construction
Closure Spillway				
Minimum spillway base width	2.0m	2.0m	2.0m	SRK judgement and for ease of construction and maintenance
Spillway side slopes	10:1 H:V	10:1 H:V	10:1 H:V	SRK judgement and to allow traffic during dry conditions
Longitudinal slope through dam	1%	1%	1%	SRK judgement
Longitudinal slope of embankment channel	33%	33%	33%	Initial dam configuration, prior to buttress construction
Erosion Protection				
Manning's roughness for riprap	Varies, based on riprap gradation (0.03 to 0.05)			(Abt. et al. 1988)
Riprap thickness	2 x D ₅₀			(BC MOE, 2000)
Geotextile	Non-woven, for riprap sections only			SRK judgement

Source: Prepared in text

A HEC-HMS model was developed in accordance with inputs and assumptions outlined in Section 4.3.2 for the operational and closure inflow design floods as well as the lower return period culvert design floods for the EMP and WMP. Both an operational critical catchment condition and a closure catchment condition were simulated. The CN selected for the IDF assume AMC III conditions and conservatively use the calibrated results for the 200-year flood instead of higher return periods (Table 4-1). Time of concentration for each natural catchment was calculated and a minimum time of concentration of 10 minutes was applied to any ponded area. Catchment areas are presented in Figure 4-7. Note that upstream diversion or collection infrastructure are assumed to fail in the event of the IDF, which exceeds the structures' design criteria.

During passage of the IDF in operations, it was assumed that culverts were not functional (e.g., because of blockage). Both the spring and summer PMF cases were considered for operational and closure conditions. The inflow hydrograph is presented in Figure 4-8, Figure 4-9 and Figure 4-10 for the NMP, EMP and WMP, respectively, along with the final outflow hydrograph based on selected spillway configuration. In the case of the NMP, the operational spillway is more critical in terms of the IDF, and no changes will be required at closure. Spillway and freeboard results for operational conditions are presented in Table 4-27 while closure conditions are presented in Table 4-28.

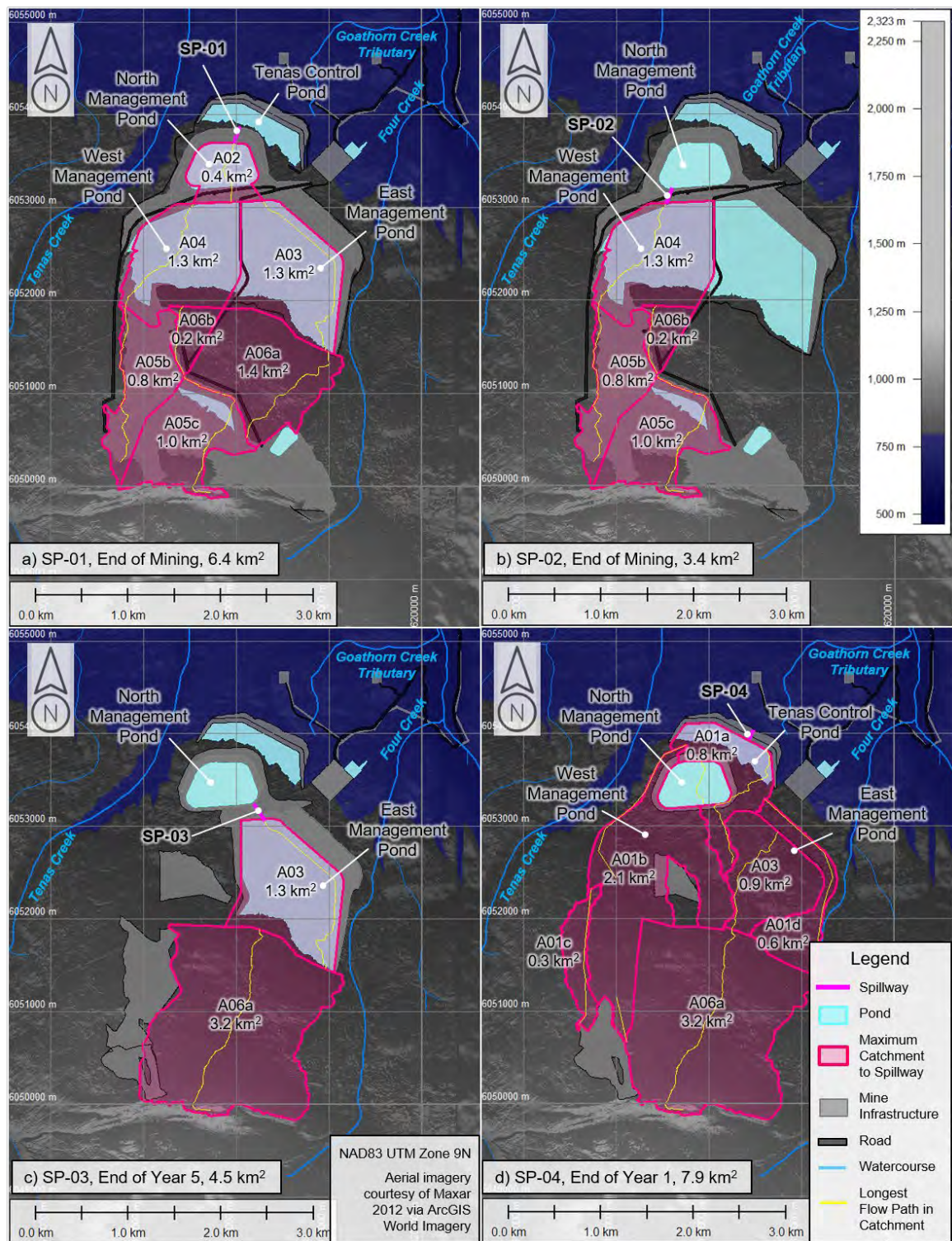


Figure 4-7: Management Pond Spillway Catchment Map

Table 4-27: Management Pond Operational Spillway and Freeboard Results

Parameter	NMP	EMP	WMP
Spillway ID	SP-01	SP-03	SP-02
Critical Catchment Area [km ²]	6.4	4.5	3.4
Average Catchment time of Concentration [hr]	3.8	0.8	1.5
25-year Peak Inflow [m ³ /s]	n.a.	21.9	23.9
Diameter and No. of culverts required	n.a.	1.0 m dia., n = 14	1.0 m dia., n = 14
Culvert Invert Elevation [masl]	n.a.	849.4	849.4
25-year Maximum Water Level [masl]	n.a.	850.75	850.86
Overflow Channel Invert Elevation [masl]	828.0	850.8	851.0
IDF Peak Inflow [m ³ /s]	22.8 ⁽¹⁾	73.7	70.6
IDF Peak Outflow through Overflow Channel [m ³ /s]	18.6	12.5	6.2
Spillway Base Width [m]	19.0	26.0	11.5
IDF Maximum Water Level [m]	828.6	851.2	851.4
Minimum Freeboard [m]	0.31	0.56	0.47
Available Freeboard during IDF [m]	1.40	0.80	0.60
Velocity during IDF through Spillway [m/s]	1.71	1.38	1.32
Spillway Riprap D ₅₀ [m]	0.10	0.10	0.10
Velocity during IDF along Embankment Channel [m/s]	4.12	3.36	3.34
Embankment Channel Riprap D ₅₀ [m]	0.40	0.25	0.25

1- Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Spillways\Telkwa_SpillwayDesign_rev08_rk

2 Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Spillways\SpillwayCulverts\Telkwa_SpillwayCulvert_DesignHY8_rev03_rk.xlsm

Notes:

¹ includes attenuated flow from EMP and WMP

Table 4-28: Management Pond Post-Closure Spillway and Freeboard Results

Parameter	NMP	EMP	WMP
Spillway ID	SP-01	SP-03	SP-02
Critical Catchment Area [km ²]	6.4	3.0 ⁽¹⁾	3.4
Average Catchment time of Concentration [hr]	3.7	0.7	1.5
IDF Peak Inflow [m ³ /s]	23.5	67.5	70.6
IDF Peak Outflow through spillway [m ³ /s]	17.2	7.9	8.8
Spillway Base Width [m]	25.0	26.0	22.5
IDF Maximum Water Level [m]	828.50	850.30	850.35
Minimum Freeboard [m]	0.31	0.56	0.47
Available Freeboard during IDF [m]	1.50	1.70	1.65
Velocity during IDF through Spillway [m/s]	1.56	1.17	1.27
Spillway Riprap D ₅₀ [m]	0.10	0.10	0.10
Velocity during IDF along Embankment Channel [m/s]	3.73	2.82	3.09
Embankment Channel Riprap D ₅₀ [m]	0.35	0.25	0.25

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Spillways\Telkwa_SpillwayDesign_rev08_rk

Notes:

¹ corresponds to End of Mining catchment area

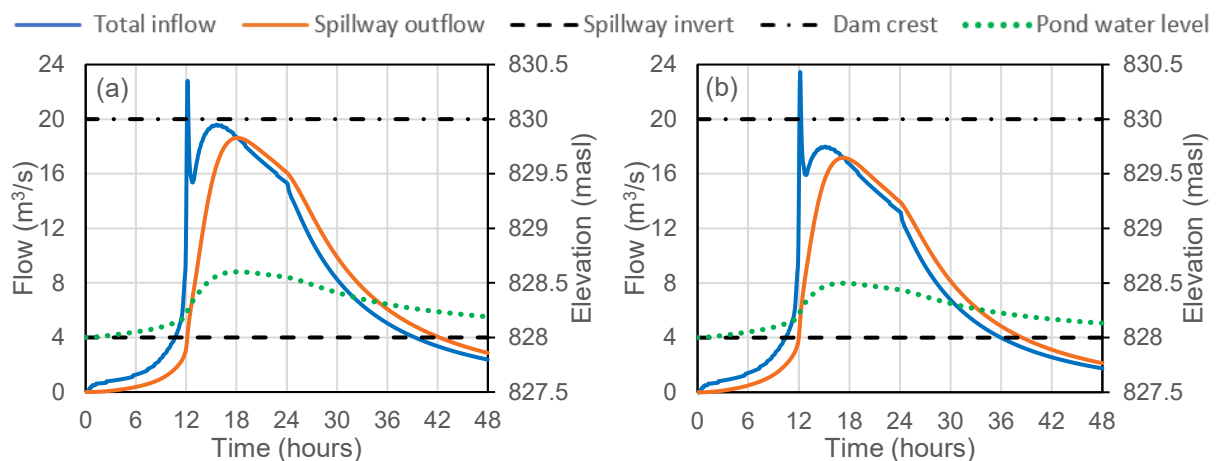


Figure 4-8: North Management Pond Inflow Design Flood Hydrographs. (a) Operational Conditions Hydrographs, (b) Closure Conditions Hydrographs

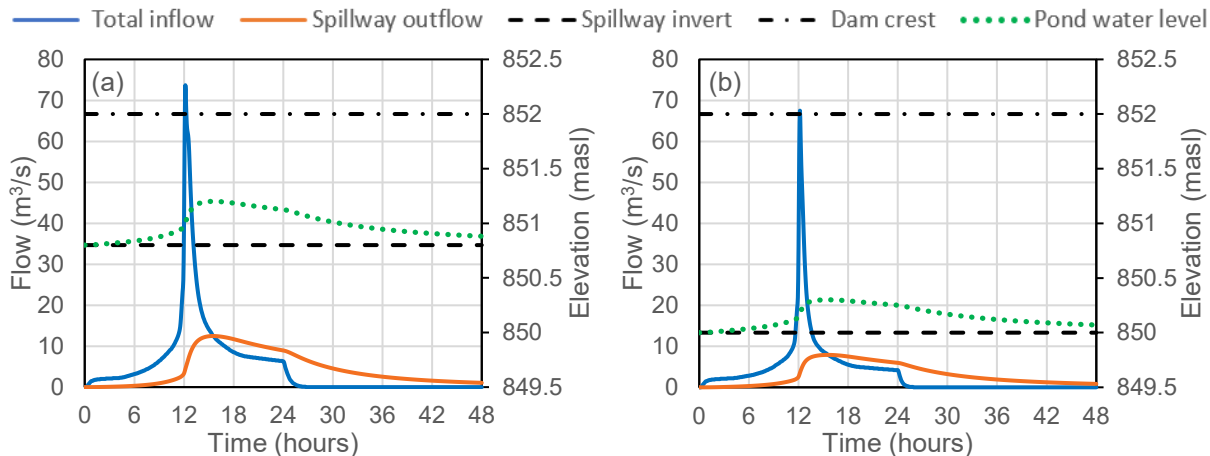


Figure 4-9: East Management Pond Inflow Design Flood Hydrographs. (a) Operational Conditions Hydrographs, (b) Closure Conditions Hydrographs

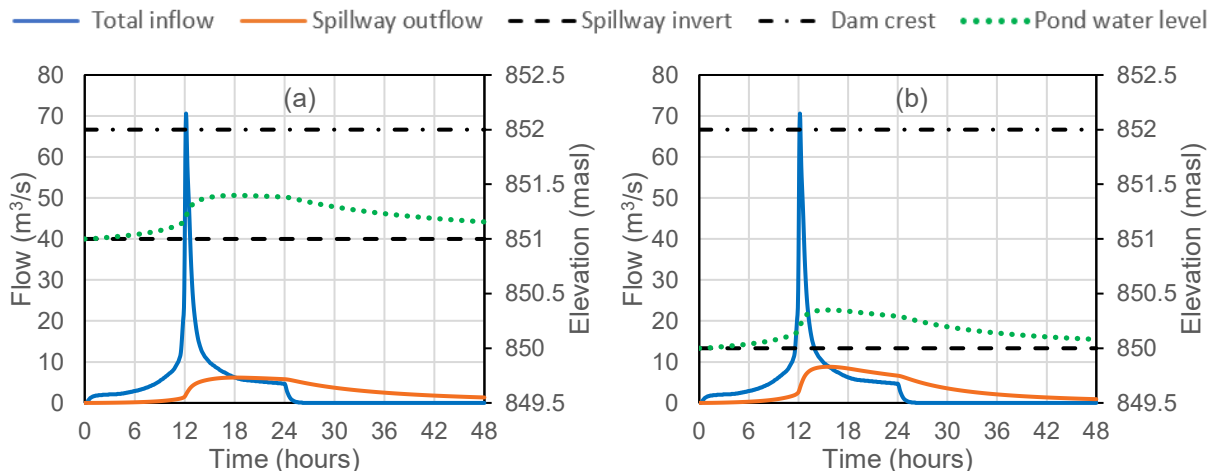


Figure 4-10: West Management Pond Inflow Design Flood Hydrographs. (a) Operational Conditions Hydrographs, (b) Closure Conditions Hydrographs

4.6.3 Management Pond Operations

Management ponds will require a minimum water cover thickness above placed PAG and processed rock to prevent oxidation and risk of ML/ARD. However, an excess of stored water may pose an increased consequence of dam failure and may also impede mine rock deposition. For this reason, operating criteria for the management ponds have been developed to control water levels in each facility.

Objectives of the management pond controls are as follows:

- Achieve and maintain the minimum 1.0 m water cover on top of PAG material within 1-year of rock being blasted within the open pit
- Minimize potential dam breach failure consequence and enable PAG deposition by discharging excess water via the Tenas Control Pond

Pumping systems will be in-place throughout operations to control water levels within each management pond. Pump controls will be dictated by regular surveys of PAG levels and daily water level readings. Pumping systems will operate as follows:

- Activate pump from Tenas Control Pond to any given management pond when that ponds' water cover depth is less than 1.0 m above PAG material
- Activate pump from any given management pond to the Tenas Control Pond when water cover depth exceeds 1.2 m above PAG material

During snowmelt, or extreme precipitation events, it is expected that water cover depths in any given management pond may temporarily exceed the pump activation level of 1.2 m above top of PAG. For dam breach modeling, it was assumed that 2.0 m of water depth would be in-place, with additional assumptions for conservative modeling.

PAG deposition and placement strategies are detailed in Section 1.1 Description of Proposed Project in the EAC Application.

4.7 Other Project Infrastructure

The Project will consist of three additional water storage facilities: the Potable Water Storage Pond, Wastewater Storage Pond, and Soil Treatment Facility Pond. Water management in the quarry areas is also discussed in the following sections.

4.7.1 Potable Water Storage Pond

The Potable Water Storage Pond will serve as an intake to the potable water treatment plant for mine operations. The plant will require 17.3 m³/day year-round, from a source with reliable water quality. Potable water plant design was prepared by Radloff Inc. (2021), including the pump intake and pipeline design from the Potable Water Pond to the plant, which will be located near the CPP.

The Potable Water Storage Pond will be constructed along the alignment of channel C-02 and will consist of an over-excavated cell, starting at the southeast extent of C-02. During winter months, it was assumed that up to 1.0 m of water in the cell could freeze (BC MOAL, 2006), reducing the available capacity and available water to meet potable water demand. The geometry was selected to maintain a minimum of 8 months of potable water demand (approximately 4,250 m³ of water) beneath the maximum ice cover in the cell.

Cell geometry is presented in Figure 9 of Appendix D. Total capacity to the spill point (i.e., invert of C- 02) in the cell, without ice, will be approximately 7,200 m³, sufficient to provide 415 days of potable water plant operations.

The Potable Water Storage Pond cell will discharge into the C-02 channel, which is designed for the 1 in 200-year event. An overflow weir will also be constructed as part of the Potable Water Storage Pond to convey excess flows to the EMP. Design criteria and objectives of the overflow weir are described in Section 4.2.2.

It is anticipated that the cell will remain near-full during the open water season based on the expected average runoff yield of 600 to 3,300 m³/day. Inflows are based on the water balance outputs (SRK 2021b) for channel catchment area, which ranges from 3.2 km² at the beginning of operations to a final value of 1.6 km² towards the end of operations as the open pit footprint expands farther east.

4.7.2 Wastewater (Sanitary) Storage Pond

The Wastewater Storage Pond is a 5,000 m³ storage facility located within the Mill Infrastructure Complex (MIC) pad, within the catchment of the CPP Sedimentation Pond. The design of the Wastewater Pond is described in Section 1-1 Description of Proposed Project in the EAC Application. In summary, it has been designed to hold 12 months of effluent from the Sanitary Treatment Plant or 5,000 m³ to allow discharge to only occur in the open water season. Discharge occurs via a pumped pipeline to the tributary of Goathorn Creek and mixes with pumped outflows from the Tenas Control Pond (refer to Tenas Control Pond water demand in Section 4.5.2).

4.7.3 Soil Treatment Facility Pond

A soil treatment facility (STF) is required for treatment of petroleum hydrocarbon (PHC) contaminated soil. STF design is described in SRK 2021c. BC Ministry of Environment guidance does not specify criteria for minimum storage capacity for stormwater and snowmelt. The design of the STF adopted the criterion based on Northwest Territory guidelines (LWBMV 2020): The facility is designed to contain the 24-hour, 1 in 10-year return period event plus the snowmelt equivalent to the average annual snow accumulation.

The area is situated between the sanitary treatment plant and the run-of-mine stockpiles, within the catchment area of the CPP Sedimentation Pond. Any overflows from the STF cells will therefore be captured by the CPP Sedimentation Pond and managed up to the 1 in 200-year event.

4.7.4 Gravel and Rock Quarry Development

Best Management Practices will be included during gravel and rock quarry development to control sediment and runoff. Upstream runoff will be diverted around working areas to reduce water into the quarry using temporary diversion berms. The base of the quarry floor will include a low point to capture runoff from the working area. Overflows from the low point will be discharged to the environment when acceptable. A minimum storage volume will be maintained to facilitate sedimentation up to the 1 in 10-year event. Storage details and discharge options will be developed in Detailed Engineering.

4.8 Pumping Systems and Pipelines

The water management plan includes six types of pump and pipeline systems to circulate water throughout life of mine. Table 4-29 summarizes the pumping systems for the Tenas Project.

Table 4-29: Tenas Project Pumping System Description

Pumping System	Description
PMP_SYS-01	Tenas Control Pond discharge to environment (Goathorn tributary). Activated when Tenas Control Pond exceeds maximum target operating level of 797.1 masl
PMP_SYS-02	Tenas Control Pond discharge to Management Ponds to maintain minimum water cover. Activated when water cover in any given pond is less than 1.0 m.
PMP_SYS-03	Management Ponds discharge to Tenas Control Pond to maintain maximum target water cover depth. Activated when water cover in any given pond exceeds 1.2 m.
PMP_SYS-04	CPP Sedimentation Pond discharge to Tenas Control Pond
PMP_SYS-05	Sedimentation Pond discharges to environment
PMP_SYS-06	Open Pit to the active management pond, designed to dewater the 1 in 2-year flood over a 24 hour duration in 2 days from the open pit area at the time of the flood

As noted in Section 1.3, pumping systems from the Tenas Control Pond to the CPP, and pumping infrastructure associated with water treatment plants for potable water and sewage treatment are not included in this design scope.

The following sections summarize conveyance requirements and pump and pipeline sizing for each system outlined in Table 4-29.

4.8.1 Pump Design Details

The design flow rate for each pumping system is summarized in the following bullet points:

- PMP_SYS-01 and PMP_SYS-02 have the same design capacity, developed based on the annual average inflows to the Tenas Control Pond minus the CPP demand (Section 4.5.5). The flow rate represents the expected discharge volume over a 5-month open water season, and was calculated to be 0.1 m³/s, or approximately 9,000 m³/day

- PMP_SYS-03 represents a mobile system that will be situated in the active management pond, as dictated by the current PAG deposition location. While each management pond is active, water more than the maximum target water cover depth (1.2 m) will be pumped to Tenas Control Pond for discharge off-site. This rate was selected to be the same as the flow rate from PMP_SYS-01, or 9,000 m³/day, which was validated through the water and load balance modeling (SRK 2021b)
- PMP_SYS-04 is a contingency system to discharge water collected in the CPP Sedimentation Pond to the Tenas Control Pond. The CPP Sedimentation Pond was designed to provide sufficient retention time to remove 10-micron particles up to the 1 in 10-year inflow event, however based on the proximity of the pond to plant and soil treatment facilities, there is a risk that this water may be in contact with other contaminants. The PMP_SYS-04, therefore, provides a back-up solution to route runoff from the CPP area towards the Tenas Control Pond, and controls the Project discharge towards Goathorn Creek instead of flowing by gravity into the smaller Four Creek (downstream of the CPP Sedimentation Pond). The pumping rate was sized to be 0.09 m³/s, or approximately 7,800 m³/day, based on sedimentation requirements outlined in Section 4.4.1
- PMP_SYS-05 will be in-place at each sedimentation pond, including the Explosives Magazine Sedimentation Pond, Explosives Facility Sedimentation Pond, Rail Infrastructure Sedimentation Pond, CPP Sedimentation Pond, Rock Quarry Sedimentation Pond, Gravel Quarry Sedimentation Ponds for Pit 1 and Pit 2, and the East and West TAC Sedimentation Ponds, enabling discharge to the downstream environment. The minimum pumping rates for each pond vary from 0.006 m³/s to 0.03 m³/s. Based on the low head requirements (less than 3.0 m of static head), systems were designed equally, providing versatility and redundancy to operators, and enabling purchase of standard back-up pumps. The design rate for this system was selected to be 2,600 m³/day based on the maximum of the three sedimentation ponds (Rail Infrastructure Pond) design rate
- PMP_SYS-06 will be a mobile pumping station that follows the active mining location during operations. Mining methods will result in natural depressions and low points throughout pit floor, providing temporary storage for rainfall events. Progressive backfilling of the pit will also alter the flow of runoff. The worst-case period for pump flow rate design was identified to be in the deepest portion of the pit, in the footprint of the WMP, when backfill in this area is minimized. The flow rate was calculated based on a 1 in 2-year return period 24-hour rainfall event, assuming 80% of precipitation is collected at the pit floor, and is dewatered in a 2-day period, which resulted in a flow rate of 10,900 m³/day (0.13 m³/s)

For pump characterization, the following additional assumptions and calculations were completed for each system:

- Proposed pipeline alignments were selected to follow planned mine or haul roads, where possible, and reduce corners and bends
- Static head was calculated based on the maximum elevation difference along the proposed pipeline alignment
- Pipeline length was selected based on the selected alignment with an additional 20% contingency
- Pipeline diameter was selected to facilitate a maximum expected velocity of 2 m/s based on the design flow rate

- Head losses in the system were calculated based on Hazen Williams, with an assumed Hazen Williams roughness value (C) of 120, representative of HDPE pipelines. Additional singular losses value of 5 was selected, which accounts for entrance/exit losses and bends in the pipeline
- Pump efficiency was assumed to be 70%
- The number of stages along the pumped line were selected based on calculated pressure during the design flow, where an additional stage was incorporated when calculated pressure exceeded the pressure rating, assuming DR17 pipes

Pump characterization results are summarized in Table 4-30, including the design flow rate, pipeline length, and the calculated total head and power requirements. A third party will develop detailed design of pumping systems and pipelines.

Table 4-30: Pump Design Summary

Pump ID	Intake Location	Discharge Location	Design Flow Rate [m ³ /day]	Total Head [m]	No. of Stages	Power [HP]	Pipe Length [m]
PMP_SYS-01	Tenas Control Pond	Goathorn Tributary	9,000	25	1	50	340
PMP_SYS-02	Tenas Control Pond	Management Ponds	9,000	76	2	75	1650
PMP_SYS-03	Management Pond	Tenas Control Pond	9,000	75	2	100	1650
PMP_SYS-04	CPP Sedimentation Pond	Tenas Control Pond	7,800	14	1	25	880
PMP_SYS-05	Sedimentation Pond	Downstream Environment	2,600	5	1	1	30
PMP_SYS-06	Open Pit	Management ponds	10,900	114	2	150	1200

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Pumping Systems\Pump_Summary_rev1_20210419.xlsx

4.9 Road Crossings Design

During the pre-mining and operations phases of the project, haul roads and access roads will be constructed over low lying points and collection channels. To facilitate traffic and allow drainage, conveyance culverts are proposed beneath service roads. Two types of culverts were considered for the road crossing:

- Channel culverts – including a culvert crossing at the C-01 channel and the C-02 channel as well as crossing Goathorn Creek Tributary
- Spillway culverts – from East and West Management Ponds to North Management Pond

Figure 1 and 19 in Appendix D illustrate the location of required road crossings along access roads which were positioned to maintain required drainage patterns, and the typical section for road crossings.

The BC Health, Safety and Reclamation Code specifies that road crossings over stream be designed to convey the 1 in 100-year return period peak flow event (MEM, 2017). Only the crossing at the Goathorn Creek tributary is considered a natural stream. Design criteria were selected based on consequence of failure of each system.

- The channel C-01 will cross the main haul road from the CPP to the management ponds and open pit. This crossing, identified as culvert CU-01, is not expected to be overtopped, due to the elevation of the haul road, however backwater could result in overtopping of the C-01 channel towards the CPP. This could result in uncontrolled discharge to the environment, and for this reason, the 100-year return period peak flow was used for CU-01 design
- The channel C-02 will cross the mine road upstream (south) of the intersection dam between the EMP and WMP (CU-02). An undersized culvert would result in overflows from C-02 into the open pit or into the WMP, depending on the timing within the mine life. Local erosion could occur, but water would be controlled within downstream infrastructure. For this reason, a 25-year return period peak flow was used for sizing of the CU-02 culvert
- Both the EMP and WMP are expected to discharge excess water, once they have reached their maximum operating levels, towards the NMP. This flow pathway will cross the haul road connecting the CPP to the open pit. In the event that the discharge rate exceeds the capacity of these culverts (CU-03 from EMP and CU-04 from WMP), water would be conveyed within the emergency spillways designed for the IDF (refer to Section 4.6.2). Minor erosion may be expected across the emergency spillway; however, the riprap will ensure structural stability is maintained. Since overflows will be controlled, the design event for these two road crossings was selected to be the 25-year return period event
- The access road to the Explosives Facility will cross the Goathorn Creek Tributary which functions as the discharge conveyance channel from the Tenas Control Pond. Discharge more than the culvert capacity could result in erosion or potential wash-out of the Explosives access road, which could result in loss of access for a temporary period and release of associated material sediment into Goathorn Creek. A 100-year return period was selected for the CU-05 design event. At closure, this road will no longer be required, and the culvert and road may be removed and replaced by a riprap swale

Culverts were sized to maintain the maximum headwater depth to culvert diameter ratio (HW/D) below 1.5 (Federal Highway Administration, 2019) during the design flood event.

The BC Forests, Lands and Natural Resource Operations Engineering Manual requires that roads at 6-9% grade and with a slight erosion hazard have culverts placed every 250 m (FLNRO Engineering Manual 2018). Additional culverts can be included as needed along access roads within the Project footprint.

A summary of the design criteria culvert designs is presented in Table 4-31.

Table 4-31: Design Criteria for Road Crossings

Parameter	Description	Reference
CU-01, CU-05 Culvert Conveyance Capacity	1 in 100-year flood	SRK judgement
CU-02, CU-03 and CU-04 Culvert Conveyance Capacity	1 in 25-year flood	SRK judgement
Culvert slope	Variable, based on inlet and outlet conditions	SRK judgement
Maximum headwater depth to culvert diameter ratio	1.5	U.S. Department of Transportation (Federal Highway Administration, 2019)

Culvert crossings beneath service roads within the Tenas Project were sized using HY-8 software developed by U.S. Department of Transportation (Federal Highway Administration, 2019). Inputs to the culvert hydraulics model includes the slope of the culvert, the dimensions, type of material, design inflow, and expected tailwater conditions. Circular culverts were optimized with a headwater constraint such that maximum headwater depth to culvert diameter ratio does not exceed 1.5 and the flow regime is governed by inlet control. The design flood was obtained using HEC-HMS with calibrated CN values (AMC II conditions) for associated return periods, assuming functional upstream diversion and collection channels. Description of the culvert inputs and design are detailed in Section 4.9.

The CU-01, CU-02 and CU-05 culverts were sized as straight culverts using corrugated steel pipes (CSP). The diameter for these culverts were 1600 mm, 1400 mm, and 1200 mm, respectively. The slope and length of the culvert was based on available topography. The upstream water surface elevation and the inlet geometry represent the major flow control, i.e., inlet control. Critical depth occurs near the inlet and the flow regime immediately downstream is supercritical.

The EMP and WMP spillway culverts (CU-03 and CU-04 respectively) were analyzed as broken-back culverts as an alternative to steeply sloped culvert towards the NMP. Double breaks were considered where the upper section (1% slope) collects flow, the steep section (33% slope) conveys flow across the embankment buttress and the runout section (1% slope) discharges water into NMP (Drawing 19). The flow was inlet controlled for this configuration, but flat upstream slope with limited head resulted in considerable number of barrel requirements for 1000 mm diameter culverts. Inlet elevations for both culverts were selected to provide a minimum of 0.5 m above the top of NAG cover to reduce water level fluctuations within NAG material and potential release of loading from drying and re-wetting of rock.

In addition, culverts will be placed through the division berm between East and West management ponds. These culverts would not be required to convey storm flows, which would be routed through the main discharge culverts, but would serve as a gravity flow mechanism between management ponds to facilitate water cover. Culverts would be controlled by a valve that would be left closed during large flood events to prevent erosion of downstream covers.

The culverts within the Project footprint are required during pre-mining and operations and are decommissioned for post-closure. Given that roads will be kept through post-closure to provide permanent access to the Project for monitoring purposes, CU-01, CU-02 and CU-05 culverts will be replaced with armored swale crossings. Swales were designed to allow light-vehicle access with a

minimum base width of 3 m, side slopes of 5H:1V and a maximum longitudinal slope of 0.5% to allow for road surfacing material to be stable post closure. Culverts CU-03 and CU-04 from East and West management ponds, respectively, will be removed at closure and replaced with trapezoidal channel spillways, as described in Section 4.6.2.

The outlet velocities for both straight and broken-back culverts are high and erosion protection at the culvert outlet will be needed to avoid scour. In addition, the broken-back culvert on steep slopes can develop sections of sub-atmospheric (negative) and excess positive internal pressure that can affect structural condition and joints. Concrete boxes can be an alternative to this and require consideration in the detailed engineering design phase. Table 4-32 summarizes the culvert details for the road crossings.

Table 4-32: Road Crossings Culvert Details

Parameter	CU-01	CU-02	CU-03	CU-04	CU-05
Upstream Structure	Channel C01	Channel C02	EMP	WMP	Goathorn Creek Tributary
Design Flood (Return Period)	100-year	25-year	25-year	25-year	100-year
Design Peak Flow [m ³ /s]	20.5	6.4	21.9	23.9	3.7
Culvert Type	Straight	Straight	Broken-back	Broken-back	Straight
Culvert Inlet Invert Elevation [masl]	803.6	870.1	849.4	849.4	778.14
Culvert Outlet Elevation [masl]	801.4 ⁽¹⁾	869.8	828.0 ⁽²⁾	828.0 ⁽²⁾	777.23
Longitudinal Slope [m/m]	1.8%	1.0%	1%, 33%, 1% ⁽³⁾	1%, 33%, 1% ⁽³⁾	3.0%
Diameter of Circular Culvert [mm]	1600	1400	1000	1000	1200
Number of Culvert Barrels Required	4	2	14	14	2
Flow Control	Inlet control	Inlet control	Inlet control	Inlet control	Inlet control
Maximum Headwater Elevation [masl]	805.77	871.77	850.75	850.86	779.38
Maximum Headwater Depth to Culvert Diameter Ratio	1.35	1.19	1.27	1.40	1.03
Tailwater Depth [m]	1.46	0.57	n.a.	n.a.	0.17
Outlet Velocity	2.66	2.88	2.58	2.69	3.25
Maximum Velocity at Steep Section	n.a.	n.a.	7.56	7.75	n.a.
Culvert Length [m]	120	30.0	135	135	30.0

Source: Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Culverts\Culvert Sizing_Rev05_20210618_rk_sab.xlsx, Z:\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\400_Water_Management\Spillways\SpillwayCulverts\Telkwa_SpillwayCulvert_DesignHY8_rev03_rk.xlsm

Notes:

¹ Corresponds to downstream channel invert

² Corresponds to overburden layer elevation of North Management Pond

³ Indicates slopes for upper, steep and runout sections respectively

4.10 Care and Maintenance

Should the Project go into temporary Care and Maintenance, water monitoring and regular maintenance will continue as in regular operations. The following measures related to water management will continue:

- Active monitoring of management ponds and available water covers will continue throughout Care and Maintenance, to control water levels
- Keep pumping systems in-place at the open pit
- Continue to discharge off-site to reduce total stored water
- Regular dam and channel inspections
- Channel and spillway maintenance after large storm events

4.11 Closure and Reclamation

Progressive reclamation of water management features will occur throughout the life of mine where possible, including:

- Installation of covers across each management pond upon completion of PAG material placement which will consist of variable thicknesses of NAG rock, overburden, and soil. The covers will be graded and vegetated, maximizing infiltration into the saturated PAG and NAG rock zones, and enabling gravity drainage towards the spillway inverts
- Vegetation will be established within permanent engineered channels that are lined with turf reinforcement mats
- Buttresses and dams will be vegetated after completion of mine rock placement at each facility. Forestry will not be established on dam portions of the management ponds
- Toe drainage channels along each dam will remain in-place until monitoring shows that water quality is acceptable for direct discharge to the environment. It is expected that channel C-04 along the toe of the Tenas Control Pond may be decommissioned prior to end of operations after the dam slopes have been sufficiently vegetated to control erosion
- Open pit walls will be progressively covered with NAG rock backfill and overburden material. At the end of operations, a defined end-pit lake area will remain in the southwest portion of the open pit, however original open pit walls will be covered with NAG rock and overburden material to prevent ARD

Once mining is complete, decommissioning and reclamation will continue for approximately four years, as described in (SRK 2021f). The following decommissioning activities are related to water management features:

- The operational spillways at the EMP and WMP will be decommissioned and replaced by the closure spillways (Section 4.6.2). Armoring will be extended into the downstream NMP and across the NMP soil cover to the NMP spillway
- Channels C-01 and C-03 along the toe of the East and West management ponds will be breached once monitoring shows that erosion and sediment migration from vegetated slopes has been controlled. Downgradient berms will be breached at regular intervals, preventing channelized flow, and reducing risk of erosion on the natural slopes towards Tenas Creek and Four Creek. Associated road crossings will be removed
- The Tenas Control Pond will be dewatered to the Goathorn tributary during open water season months of May through October. Dewatering rates will be dictated based on pond water quality at the end of operations which will support a detailed decommissioning plan. Once dewatered, sediment collected throughout operations in the TCP will be allowed to drain and dry out and will be form the basis of the wetland. The dam will be breached and an armored channel section will be constructed from the outlet of the NMP spillway embankment channel to the former outlet of the Tenas Control Pond spillway embankment channel, which will tie-into the erosion protection along the Goathorn Creek tributary
- Other sedimentation ponds will be dewatered and backfilled or the dams breached to allow these ponds to naturally drain. Backfill surfaces will be covered with overburden and vegetated, returning to pre-development conditions where possible
- Pumping systems will be decommissioned and removed from site

The following water management systems will remain in-place post-closure

- The potable water pond cells within the C-02 channel will remain in-place, along with the C-02 channel. Water from the catchment between the EMP and the final open pit extents will continue to be diverted towards Four Creek
- The C-05 channel south of the open pit will remain in-place to continue diverting water around the pit lake and towards Four Creek
- Each management pond will discharge surface runoff and excess water from the soil covers through the closure spillways. Long-term monitoring and inspections of dams will be required

5 Conclusions

5.1 Risk and Opportunities

Below are several water management risks that have been identified during the study:

Table 5-1: Water Management Risks and Mitigation Measures

Risk	Mitigation or Path Forward
Water quality in sedimentation ponds does not meet guidelines and cannot be discharged	Sedimentation ponds are governed by pump-controlled systems. Contingency pumping to Tenas Control Pond has already been allotted for in the CPP Sedimentation Pond and can be implemented at the Explosives Magazine and Explosives Facility sedimentation ponds, as-needed. Rail Infrastructure Sedimentation Pond is too far from the Tenas Control Pond for pumping to be viable. In-situ treatment would be required, including addition of flocc or other settling agents which has been included as a contingency.
Water quality in Tenas Control Pond does not meet guidelines and cannot be discharged	Additional treatment may be required. Based on water and load balance estimates of discharge and in-stream water quality, this is not anticipated throughout operations or post-closure phases of the Project.
Water quantity (pattern and flow rates) in downstream environments is disturbed, affecting fish and aquatic life	The EA documents consider the potential effects of changes to flow regime and magnitude on fish and aquatic life. Diversion channels such as C-02 and C-05 have been incorporated into designs to mitigate changes to Four Creek catchments.
Dry years or higher groundwater seepage could result in insufficient water cover in Management Ponds and deposited PAG material or processed rock becomes acidic	Contingency water sources, including water available in the Tenas Control Pond, will be available to supplement water covers. Furthermore, 750,000 m ³ of water from Goathorn Creek has been included in the Environmental Assessment Application as a contingency water supply source. The C-02 channel can be controlled to discharge towards C-01 and ultimately Tenas Control Pond, instead of Four Creek. This may have a temporary effect on Four Creek flows, and a trade-off of the effect to Four Creek vs. pond water quality may be required. The water and load balance tests the system under a range of dry conditions.
Larger than expected flood events occur within Project area, resulting in overtopping and failure of engineered systems	Events that exceed the selected design criteria are possible within the mine life. Regular inspection and maintenance is anticipated after large flood events to repair erosion protection. It is expected that water quality during high flow events will be sufficiently diluted and have minor effects on downstream environments. Dams and water storage facilities are designed with emergency spillways designed in accordance with CDA guidelines. The IDF for management ponds is 2/3 between the 1,000-year event and the PMF.

The following additional opportunities to optimize and refine water management designs and associated costing have been identified:

- Further trade-off studies for channel erosion protection, including keeping TRM in closure instead of using riprap, given that synthetic products and haulages of riprap from off site are expensive
- Optimization of spillway configurations to reduce the cost of construction for these structures
- Optimization and review of high pressures expected within broken-back culverts (CU-03 and CU-04) between management ponds

Limitations

- Hydrologic analysis to date to support peak flow modeling has been primarily based on Goathorn Creek Water Survey of Canada flow data. Ongoing monitoring of Tenas Creek and Four Creek, the smaller watersheds in proximity to the Project, is required to improve and validate peak flow estimates. A continuous rainfall-runoff model at Four Creek and Tenas Creek would be beneficial to assess peak flow responses for smaller watersheds
- Climate change considerations for the PMP and 1 in 1,000-year event were not considered for this study. Final designs should incorporate climate change into design flood, estimates
- Culverts during operations for EMP and WMP will experience high pressure flows during an IDF event. Detailed design should consider selection of appropriate materials for culvert construction

Closure

This report, Tenas Project – Water Management Design Technical Report - FINAL, was prepared by

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All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

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Appendix A Stage-Storage Relationships

Table A- 1: Management Pond Stage-Storage-Elevation Relationships

Elevation [masl]	Cumulative volumes [m3]			Surface Area [m2]		
	North MP	East MP	West MP	North MP	East MP	West MP
753.0	-	-	1	-	-	-
753.5	-	-	158	-	-	-
754.0	-	-	906	-	-	-
754.5	-	-	2,332	-	-	3,085
755.0	-	-	4,404	-	-	4,363
755.5	-	-	7,105	-	-	5,670
756.0	-	-	10,512	-	-	7,150
756.5	-	-	14,868	-	-	9,240
757.0	-	-	20,458	-	-	11,650
757.5	-	-	27,329	-	-	14,210
758.0	-	-	36,011	-	-	16,740
758.5	-	-	46,879	-	-	22,650
759.0	-	-	59,579	-	-	26,000
759.5	-	-	73,865	-	-	29,350
760.0	-	-	89,696	-	-	32,330
760.5	-	-	106,984	-	-	35,040
761.0	-	-	125,555	-	-	37,650
761.5	-	-	145,444	-	-	40,240
762.0	-	-	166,661	-	-	42,880
762.5	-	-	189,282	-	-	45,680
763.0	-	-	213,354	-	-	48,560
763.5	-	-	238,832	-	-	51,300
764.0	-	-	265,706	-	-	54,100
764.5	-	-	293,983	-	-	56,800
765.0	-	-	323,643	-	-	59,500
765.5	-	-	354,661	-	-	62,100
766.0	-	-	387,023	-	-	64,800
766.5	-	-	420,676	-	-	67,200
767.0	-	-	455,502	-	-	69,500
767.5	-	-	491,462	-	-	71,700
768.0	-	-	528,531	-	-	74,000
768.5	-	-	566,688	-	-	76,100
769.0	-	-	605,936	-	-	78,300
769.5	-	-	646,276	-	-	81,900
770.0	-	-	687,695	-	-	85,500
770.5	-	-	730,173	-	-	87,900
771.0	-	-	773,697	-	-	90,100
771.5	-	-	818,248	-	-	92,100
772.0	-	-	863,781	-	-	94,100

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Appendix A: Stage-Storage-Elevation Relationships

Elevation [masl]	Cumulative volumes [m3]			Surface Area [m2]		
	North MP	East MP	West MP	North MP	East MP	West MP
772.5	-	-	910,295	-	-	96,100
773.0	-	-	957,801	-	-	98,200
773.5	-	-	1,006,315	-	-	100,200
774.0	-	-	1,055,844	-	-	102,300
774.5	-	-	1,106,399	-	-	104,400
775.0	-	-	1,157,984	-	-	106,500
775.5	-	-	1,210,602	-	-	108,700
776.0	-	-	1,264,266	-	-	110,800
776.5	-	-	1,322,177	-	-	113,000
777.0	-	-	1,381,203	-	-	115,100
777.5	-	-	1,441,272	-	-	117,100
778.0	-	-	1,502,357	-	-	119,100
778.5	-	-	1,564,448	-	-	121,100
779.0	-	-	1,627,554	-	-	123,100
779.5	-	-	1,691,688	-	-	125,100
780.0	-	-	1,756,845	-	-	127,100
780.5	-	-	1,823,031	-	-	129,100
781.0	-	-	1,890,246	-	-	131,100
781.5	-	-	1,958,490	-	-	133,200
782.0	-	-	2,027,767	-	-	135,200
782.5	-	-	2,098,078	-	-	137,200
783.0	-	-	2,169,433	-	-	139,200
783.5	-	-	2,241,912	-	-	141,500
784.0	-	-	2,315,618	-	-	143,900
784.5	-	-	2,390,689	-	-	146,900
785.0	-	-	2,467,148	-	-	149,600
785.5	-	-	2,545,068	-	-	152,600
786.0	-	-	2,624,532	-	-	155,600
786.5	-	-	2,705,621	-	-	158,900
787.0	-	-	2,788,365	-	-	162,200
787.5	-	-	2,872,732	-	-	165,300
788.0	-	-	2,958,701	-	-	168,500
788.5	-	-	3,046,286	-	-	171,800
789.0	-	-	3,135,510	-	-	175,100
789.5	-	-	3,226,391	-	-	181,100
790.0	-	-	3,319,017	-	-	186,900
790.5	-	-	3,413,568	-	-	191,000
791.0	-	-	3,510,406	-	-	194,800
791.5	-	-	3,609,548	-	-	201,700
792.0	-	-	3,711,020	-	-	206,800
792.5	-	-	3,815,026	-	-	212,700

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Appendix A: Stage-Storage-Elevation Relationships

Elevation [masl]	Cumulative volumes [m3]			Surface Area [m2]		
	North MP	East MP	West MP	North MP	East MP	West MP
793.0	-	-	3,921,701	-	-	218,000
793.5	-	-	4,030,771	-	-	222,300
794.0	-	-	4,141,625	-	-	225,700
794.5	-	-	4,254,107	-	-	229,000
795.0	-	-	4,368,103	-	-	232,000
795.5	-	-	4,483,523	-	-	234,800
796.0	-	-	4,599,998	-	-	236,900
796.5	-	-	4,717,374	-	-	238,800
797.0	-	-	4,835,632	-	-	240,600
797.5	-	-	4,954,783	-	-	242,300
798.0	-	-	5,075,065	-	-	244,000
798.5	-	-	5,196,508	-	-	246,700
799.0	-	-	5,319,055	-	-	248,700
799.5	-	-	5,442,668	-	-	250,600
800.0	-	-	5,567,360	-	-	252,500
800.5	-	-	5,703,541	-	-	254,500
801.0	-	-	5,840,754	-	-	256,400
801.5	-	-	5,978,984	-	-	258,400
802.0	-	-	6,118,222	-	-	260,300
802.5	-	-	6,258,458	-	-	262,200
803.0	-	-	6,399,688	-	-	264,200
803.5	-	-	6,541,941	-	-	266,300
804.0	-	-	6,685,320	-	-	268,400
804.5	-	-	6,829,891	-	-	271,000
805.0	-	-	6,975,848	-	-	273,400
805.5	-	-	7,123,275	-	-	276,200
806.0	-	-	7,272,139	-	-	278,700
806.5	-	-	7,422,418	-	-	281,400
807.0	-	-	7,574,057	-	-	283,900
807.5	-	-	7,727,015	-	-	286,500
808.0	-	-	7,881,221	-	-	288,900
808.5	-	-	8,036,607	-	-	291,100
809.0	-	-	8,193,067	1,623	-	293,400
809.5	466	-	8,350,557	3,093	-	301,900
810.0	1,123	-	8,509,021	5,470	-	310,200
810.5	6,738	-	8,668,394	12,480	-	313,600
811.0	14,942	-	8,828,629	20,760	-	315,500
811.5	25,958	-	8,989,676	37,090	-	317,100
812.0	40,193	-	9,151,508	59,600	-	318,500
812.5	76,609	-	9,322,278	72,200	-	319,900
813.0	118,602	-	9,493,790	85,900	-	321,300

Tenas Project – Water Management Design Technical Report

Appendix A: Stage-Storage-Elevation Relationships

Elevation [masl]	Cumulative volumes [m3]			Surface Area [m2]		
	North MP	East MP	West MP	North MP	East MP	West MP
813.5	166,062	-	9,666,027	97,600	-	322,700
814.0	218,864	-	9,838,972	112,400	-	323,900
814.5	282,474	-	10,012,613	127,800	-	325,200
815.0	350,375	-	10,186,927	140,500	-	326,500
815.5	422,631	-	10,361,900	152,100	-	327,800
816.0	499,164	-	10,537,565	160,600	-	329,100
816.5	583,548	-	10,713,986	169,400	-	330,700
817.0	671,440	-	10,891,236	180,300	-	332,300
817.5	763,219	-	11,069,348	191,700	-	333,900
818.0	859,037	-	11,248,364	201,200	-	335,600
818.5	965,501	-	11,428,354	211,800	-	337,600
819.0	1,076,237	-	11,609,434	224,100	-	339,900
819.5	1,190,980	-	11,791,627	234,400	-	342,500
820.0	1,309,466	-	11,974,937	241,900	-	346,000
820.5	1,434,190	42	12,159,426	249,100	337	351,000
821.0	1,561,375	320	12,345,385	255,800	1,451	359,400
821.5	1,690,649	1,051	12,533,323	259,400	3,827	368,800
822.0	1,821,642	2,547	12,724,378	262,100	8,610	378,000
822.5	1,953,991	10,772	12,920,878	264,600	16,980	389,400
823.0	2,087,586	23,020	13,121,001	267,100	28,270	398,200
823.5	2,222,431	42,014	13,325,122	269,600	47,530	407,200
824.0	2,358,531	69,014	13,533,487	272,100	66,400	421,800
824.5	2,495,890	108,864	13,773,078	274,600	83,200	432,700
825.0	2,634,515	155,702	14,017,326	277,100	101,600	444,900
825.5	2,774,409	209,671	14,266,365	279,600	118,300	456,600
826.0	2,915,579	270,911	14,520,354	282,200	133,200	468,200
826.5	3,058,027	343,147	14,782,602	284,700	147,800	479,000
827.0	3,201,761	422,036	15,049,146	287,300	163,100	490,400
827.5	3,346,783	507,460	15,320,163	289,900	177,800	501,000
828.0	3,493,100	599,289	15,595,810	292,400	192,400	513,000
828.5	3,640,717	703,243	15,879,423	295,000	208,400	527,000
829.0	3,789,638	813,949	16,167,983	297,600	224,300	541,000
829.5	3,939,867	931,380	16,461,317	300,200	241,100	564,000
830.0	4,091,411	1,055,505	16,759,159	304,100	258,100	587,000
830.5	4,091,411	1,192,641	17,063,436	304,100	275,600	600,000
831.0	4,091,411	1,336,892	17,372,266	304,100	295,200	613,000
831.5	4,091,411	1,488,259	17,685,485	304,100	314,000	626,000
832.0	4,091,411	1,646,749	18,003,104	304,100	332,600	638,000
832.5	4,091,411	1,824,665	18,325,865	304,100	354,900	649,000
833.0	4,091,411	2,011,258	18,653,252	304,100	384,600	660,000
833.5	4,091,411	2,206,866	18,985,405	304,100	406,100	671,000

Tenas Project – Water Management Design Technical Report
Appendix A: Stage-Storage-Elevation Relationships

Elevation [masl]	Cumulative volumes [m3]			Surface Area [m2]		
	North MP	East MP	West MP	North MP	East MP	West MP
834.0	4,091,411	2,411,944	19,322,219	304,100	426,200	682,000
834.5	4,091,411	2,638,892	19,664,854	304,100	453,800	694,000
835.0	4,091,411	2,875,363	20,012,850	304,100	481,300	706,000
835.5	4,091,411	3,121,632	20,366,250	304,100	511,000	718,000
836.0	4,091,411	3,378,045	20,724,973	304,100	538,000	731,000
836.5	4,091,411	3,657,995	21,103,457	304,100	560,000	744,000
837.0	4,091,411	3,945,452	21,488,639	304,100	581,000	761,000
837.5	4,091,411	4,240,427	21,880,228	304,100	604,000	777,000
838.0	4,091,411	4,542,927	22,277,967	304,100	626,000	795,000
838.5	4,091,411	4,868,144	22,684,794	304,100	650,000	810,000
839.0	4,091,411	5,200,592	23,095,603	304,100	682,000	823,000
839.5	4,091,411	5,540,912	23,510,114	304,100	705,000	834,000
840.0	4,091,411	5,889,745	23,928,082	304,100	732,000	846,000
840.5	4,091,411	6,268,822	24,349,506	304,100	760,000	853,000
841.0	4,091,411	6,655,128	24,774,109	304,100	784,000	860,000
841.5	4,091,411	7,049,047	25,201,772	304,100	808,000	868,000
842.0	4,091,411	7,450,979	25,632,372	304,100	831,000	875,000
842.5	4,091,411	7,879,514	26,066,229	304,100	855,000	882,000
843.0	4,091,411	8,316,596	26,502,750	304,100	880,000	890,000
843.5	4,091,411	8,762,626	26,941,945	304,100	904,000	899,000
844.0	4,091,411	9,218,008	27,384,013	304,100	931,000	909,000
844.5	4,091,411	9,692,452	27,829,546	304,100	948,000	918,000
845.0	4,091,411	10,172,779	28,278,292	304,100	962,000	927,000
845.5	4,091,411	10,659,065	28,730,361	304,100	975,000	936,000
846.0	4,091,411	11,151,404	29,185,855	304,100	988,000	945,000
846.5	4,091,411	11,652,757	29,645,829	304,100	1,003,000	955,000
847.0	4,091,411	12,160,557	30,109,510	304,100	1,017,000	964,000
847.5	4,091,411	12,674,872	30,576,987	304,100	1,030,000	974,000
848.0	4,091,411	13,195,774	31,048,346	304,100	1,044,000	985,000
848.5	4,091,411	13,724,465	31,534,080	304,100	1,058,000	996,000
849.0	4,091,411	14,259,242	32,023,612	304,100	1,071,000	1,008,000
849.5	4,091,411	14,800,183	32,517,015	304,100	1,084,000	1,023,000
850.0	4,091,411	15,347,364	33,014,397	304,100	1,096,000	1,040,000
850.5	4,091,411	15,902,340	33,517,598	304,100	1,110,000	1,051,000
851.0	4,091,411	16,463,011	34,025,340	304,100	1,121,000	1,063,000
851.5	4,091,411	17,029,425	34,538,001	304,100	1,132,000	1,075,000
852.0	4,091,411	17,601,633	35,055,618	304,100	1,145,000	1,089,000

Tenas Project – Water Management Design Technical Report
Appendix A: Stage-Storage-Elevation Relationships

Table A- 2: Tenas Control Pond Stage-Storage-Elevation Relationship

Elevation [masl]	Cuml. Volume [m³]	Surface Area [m²]
785.5	-	-
786.0	87	202
786.5	534	837
787.0	1,321	1,751
787.5	2,541	3,043
788.0	4,450	5,052
788.5	8,418	9,207
789.0	14,799	15,886
789.5	24,228	21,913
790.0	36,986	29,776
790.5	55,181	38,774
791.0	77,681	49,509
791.5	105,167	59,643
792.0	137,796	71,412
792.5	178,238	83,315
793.0	224,187	96,529
793.5	275,891	108,818
794.0	333,501	122,351
794.5	399,936	135,701
795.0	472,619	150,138
795.5	551,708	164,821
796.0	637,362	180,687
796.5	733,838	195,054
797.0	836,404	210,125
797.5	945,189	224,448
798.0	1,060,335	239,564
798.5	1,187,144	256,691
799.0	1,321,141	274,806
799.5	1,462,424	290,253
800.0	1,611,092	306,428

Table A- 3: Pit Lake Stage-Volume Relationship

Elevation [masl]	Volume [m³]	Elevation [masl]	Volume [m³]	Elevation [masl]	Volume [m³]
823	-	861	703,627	899	4,817,737
824	1,013	862	740,918	900	5,031,452
825	3,080	863	779,333	901	5,252,152
826	6,375	864	819,038	902	5,478,455
827	10,887	865	860,129	903	5,710,202
828	16,526	866	902,719	904	5,947,304
829	23,203	867	947,026	905	6,189,626
830	30,886	868	993,339	906	6,437,193
831	39,526	869	1,041,923	907	6,689,938
832	49,059	870	1,095,005	908	6,947,581
833	59,429	871	1,153,134	909	7,209,925
834	70,600	872	1,214,687		
835	82,554	873	1,279,964		
836	95,292	874	1,349,137		
837	108,821	875	1,422,472		
838	123,122	876	1,500,607		
839	138,177	877	1,583,875		
840	153,974	878	1,672,615		
841	170,595	879	1,766,753		
842	187,854	880	1,866,647		
843	205,785	881	1,972,436		
844	224,453	882	2,083,348		
845	243,890	883	2,199,315		
846	264,108	884	2,320,391		
847	285,126	885	2,446,467		
848	306,981	886	2,577,434		
849	329,726	887	2,713,361		
850	354,571	888	2,854,404		
851	381,678	889	3,000,718		
852	409,754	890	3,155,048		
853	438,772	891	3,317,654		
854	468,710	892	3,485,835		
855	499,551	893	3,659,558		
856	531,296	894	3,838,809		
857	563,937	895	4,023,575		
858	597,486	896	4,213,854		
859	631,936	897	4,409,654		
860	667,315	898	4,610,917		

Appendix B Dam Breach Inundation Model Memo

FINAL

Tenas Project Dam Breach Assessment Report

Tenas Project, Telkwa, BC, Canada
Telkwa Coal Limited



SRK Consulting (Canada) Inc. ■ 1CT027.003 ■ April 2021



FINAL

Tenas Project Dam Breach Assessment Report

Tenas Project, Telkwa, BC, Canada

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File Name:

Telkwa_DamBreach_Report_FINAL_1CT027-003_04072021_revF_nf_mjm_sab.docx

Suggested Citation:

SRK Consulting (Canada) Inc. 2021. Tenas Project Dam Breach Assessment Report. FINAL. Prepared for Telkwa Coal Limited: Telkwa, BC. Project number: 1CT027.003. Issued April. 2021.

Cover Image(s):

Proposed North Management Pond area during drilling. Credit: Kyle Scale, February 17, 2021.

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Appendix F	East Dam Breach, Rainy Day Scenario

Useful Definitions

This list contains definitions of symbols, units, abbreviations, and terminology that may be unfamiliar to the reader.

CDA	Canadian Dam Association
EA	Environmental Assessment
EMP	East Management Pond
GISTM	Global Industry Standard on Tailings Management
NAG	Non-acid generating
NMP	North Management Pond
PAG	Potentially acid generating
PMF	Probably maximum flood
Project	Tenas Project
Q200	1-in-200-year flood
SRK	SRK Consulting (Canada) Inc.
SRTM	Shuttle Radar Topography Mission
TCL	Telkwa Coal Limited
TCP	Tenas Control Pond
CPP	Coal Processing Plant
WMP	West Management Pond

Executive Summary

An inundation model was developed to simulate the effects of a dam breach event on areas downgradient of the Tenas Project's Management and Tenas Control Ponds, particularly the Village of Telkwa and the Maintenance Infrastructure Complex containing the Coal Processing Plant, Maintenance Facility and Mine Administration Building. A dam breach may also affect natural areas close to the site, including Tenas, Goathorn, and Four Creeks, and the Telkwa and Bulkley Rivers.

A cascade failure of the embankments corresponding to the West and North Management Ponds and the Tenas Control Pond was investigated, as well as the individual failure of the East Management Pond's dam. Incremental breach effects were assessed under fair-weather conditions (sunny-day), as well as under flood conditions (rainy-day). Flood depths, water velocities, and arrival times are provided, and the hazard for populations at risk is estimated.

Model inputs were developed based on the mine plan as of December 2020, topography, surface roughness estimates, regional hydrology, and methods from literature. There is much uncertainty surrounding inundation predictions, as the model does not account for sediment and debris transport, scouring, and hydraulic structures and infrastructure that may affect flow direction. Uncertainty and variability in predictions must be considered if using the results of the analysis for emergency response and action planning.

Results of the analysis estimate that up to 200 additional people may be put at risk due to the potential breach scenarios described, as compared to a no-breach scenario. The sunny-day breach scenarios have marginally higher incremental effects, as the total effects from a flood-induced scenario are largely driven by the flood itself. Effects on buildings, roadways, and bridges may be minor to moderate, and may result in temporary economic losses and reparation costs. Some damage to aquatic and terrestrial habitats will occur, but restoration is possible and this damage will be temporary in nature.

Based on the analysis, all four dams considered have been given a hazard classification of 'Very High', in accordance with Canadian Dam Association guidelines.

1 Introduction

Telkwa Coal Limited (TCL) is proposing to develop the Tenas Project (the Project) in northwest British Columbia. The site is located approximately 25 kilometres (km) southwest of the town of Smithers within the Telkwa River watershed and will comprise of an open pit, backfill, external stockpiles, management ponds, control ponds, the Mine Infrastructure Complex and rail infrastructure pad. The Tenas Project's total mine life is approximately 25 years including construction, operations, decommissioning, and reclamation phases.

SRK Consulting (Canada) Inc. (SRK) has been retained to conduct a dam break assessment to evaluate the potential downstream flooding conditions resulting from failure of the dams associated with the management ponds. Results will support water management designs being completed for the Environmental Assessment Certificate (EAC) application, inform dam classification according to the Canadian Dam Association (CDA) guidelines, and will be incorporated into the Emergency Response Plan.

1.1 Site Description and Plan

The dam breach assessment extents encapsulate areas directly downgradient of the potential breach locations, areas along streams leading to the key receptors, and areas containing the key receptors. Figure 1-1 provides an aerial view of the project area, showing the site water management facilities, as well as the locations of the streams along which water released from the dams may travel. There are three creeks of interest, as well as the Telkwa River and Bulkley River. The nearby Village of Telkwa is located at the confluence of the Telkwa and Bulkley Rivers, approximately 11 km northeast of the Project's mine site.

Figure 1-1: Dam Breach Analysis Site Plan



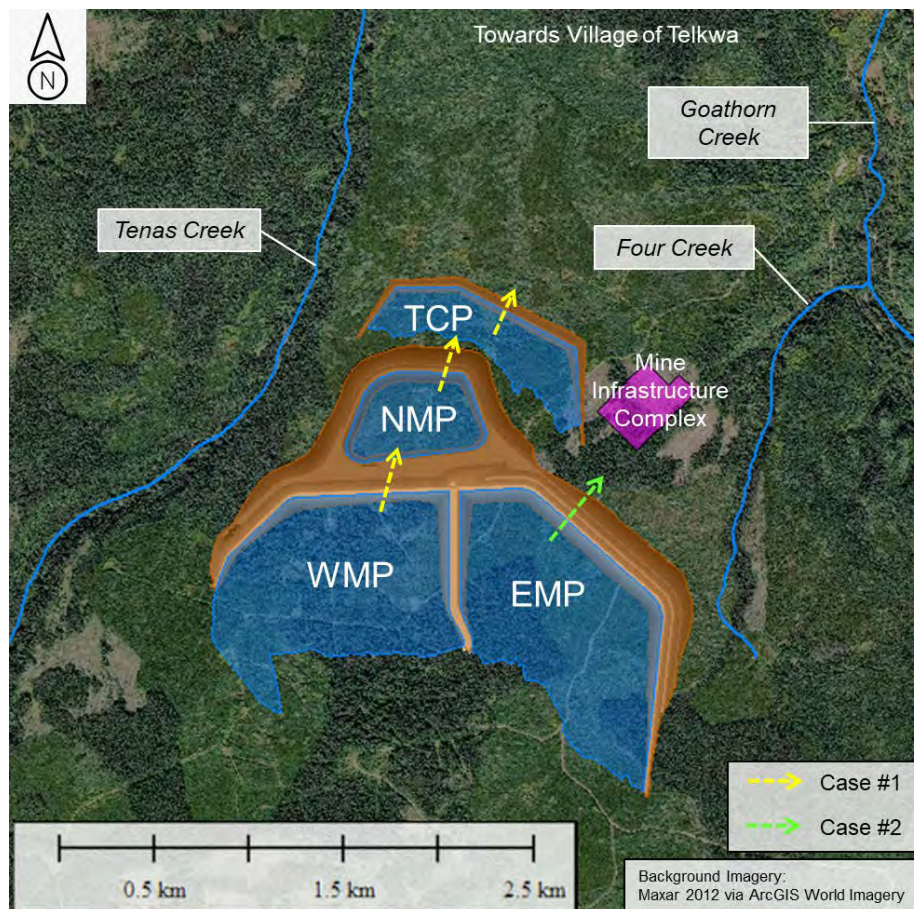
Sources: \\srk.ad\dfs\in\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\1080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

There are four water management facilities of interest to the Dam Breach analysis as shown in Figure 1-2:

1. Tenas Control Pond (TCP)
2. North Management Pond (NMP)
3. East Management Pond (EMP)
4. West Management Pond (WMP)

The three management ponds serve as potentially acid generating (PAG) rock and processed rock storage facilities with approximately 2 metres (m) of water cover maintained during operations, decommissioning and reclamation and post closure phases of the Project. Non-acid generating (NAG) rock will be placed over the PAG rock as cover, followed by overburden material to an elevation above the proposed water level. The NMP is scheduled to be constructed, filled, and covered early in the life of mine, followed by the EMP, and then by the WMP. The TCP is located downstream of the management ponds and collects overflow from the ponds and surface runoff from upslope mine infrastructure.

Figure 1-2: Management and Control Pond Site Plan



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

1.2 Scope of Work

The purpose of the dam breach assessment is to estimate the incremental consequences to key receptors downstream of the ponds resulting from embankment failure, the scope of which includes:

- identification of key risk receptors,
- assessment of credible failure modes,
- calculation of dam breach parameters and hydrographs,
- simulation of breach scenarios and production of inundation and hazard maps, and
- assessment of results to determine impacts of the potential dam breach.

The following two critical failure cases were assessed in the detailed dam breach study, and are shown previously by Figure 1-2:

1. Cascade failure of the West Dam, North Dam, and Tenas Dam, for which the key receptor is the Village of Telkwa and is shown by the yellow line on Figure 1-2.
2. Failure of the East Dam northeastward towards Four Creek, for which the key receptor is the Mine Infrastructure Complex, with some consideration given to the Village of Telkwa and is shown by green line on Figure 1-2.

The breach scenarios are determined by evaluating the mine plan for the worst-case conditions for failure during the life of mine.

In recent years, there have been a large number of long runout flow slides from Rocky Mountain coal mine waste dumps. The waste dumps are constructed as end-dumped fills with an angle of repose of 38°. Dump heights range between 100 and 400 m. The dumps are normally founded on mountain slopes that are covered with a thin veneer of granular colluvial and dense stony till materials (Dawson et al., 1998). Given the design heights of waste rock within the Management Ponds, the relatively flat ground beneath the ponds themselves, and the containment within an earthen dam, the liquefaction of neither the rock stored in the ponds nor the rock used in the dam fill is likely to occur.

The rock material stored in the ponds is therefore not expected to significantly affect inundation predictions. Additionally, the quantity of overburden from the dam that can mobilize during a breach event is considered insignificant compared to the quantity of water. Therefore, sediment transport modelling was not included in the inundation study, as the effects of a dam breach will be principally governed by the release of water, rather than eroded overburden from the embankments or stored rock material.

Water in the ponds may either be pore water that saturates the voids in the waste material, or free water which covers the mine rock. In the event of a breach, the pore water is expected to flow out relatively slowly compared to the free water and will therefore have a negligible effect on the peak flow from the breach and incremental flood effects. The breach parameters were therefore calculated based only on the free water present during the worst-case scenarios.

Sensitivity analyses were also completed to understand the effects of changes to breach and modelling parameters on inundation predictions.

2 Dam Breach Inputs

2.1 Dam Information and Geometry

The dams are to be constructed with coarse NAG waste rock and locally sourced overburden which comprises well-graded sands, silts, and gravels near the surface, with larger quantities of fine sands at greater depths. The embankment foundation consists of similar soils, and the North, East, and West dams will be supported by buttresses (SRK 2021a).

The embankment geometry for each dam at the critical failure scenario was compiled based on mine plan drawings and topographic data from the 2018 LiDAR survey of the Project area, and are summarized in Table 2-1. Storage volumes were estimated based on pond stage-storage relationships, developed using site topography and embankment surfaces provided by TCL. Summary elevation-storage tables are provided in Appendix A.

Table 2-1: Water management pond dam embankment geometry

Parameter	Unit	Critical Failure Case #1			Critical Failure Case #2
		West Dam	North Dam	Tenas Dam	East Dam
Crest length	m	1511	1173	1876	3225
Typical crest width	m	9	14.5	10	9
Overall upstream slope	-	2.5H:1V	2.5H:1V	2.5H:1V	2.5H:1V
Overall downstream slope	-	3.0H:1V	3.0H:1V	3.0H:1V	3.0H:1V
Maximum embankment height	m	37	27	18	40
Outlet elevation	masl	851	830	801	851
Crest elevation	masl	852	831	802	852
Rock fill elevation in worst-case scenario	masl	847	828	N/A	845
Total storage to outlet invert	m ³	35,151,000	3,800,800	1,936,600	16,379,300
Total storage to dam crest	m ³	36,211,900	4,119,000	2,292,100	17,514,600
Rock fill volume in worst-case scenario	m ³	31,108,800	3,239,600	N/A	10,120,000

Sources: \\srk.ad\dfs\alvan\Projects\01_SITES\Telkwa Coal\1CT027-003_2020 EA Water Management\800_Dam_Break\Inputs\Telkwa_DamBreach_Inputs_1CT017-003_rev04_nf.xlsm

Note: Final mine plan designs have since reduced the dam crest elevations to 850 masl for the EMP and WMP and 800 masl for the TCP. Dam breach assessments continue to use higher dam crests, resulting in a more conservative dam breach hydrograph for the purpose of dam classification.

2.2 Topography

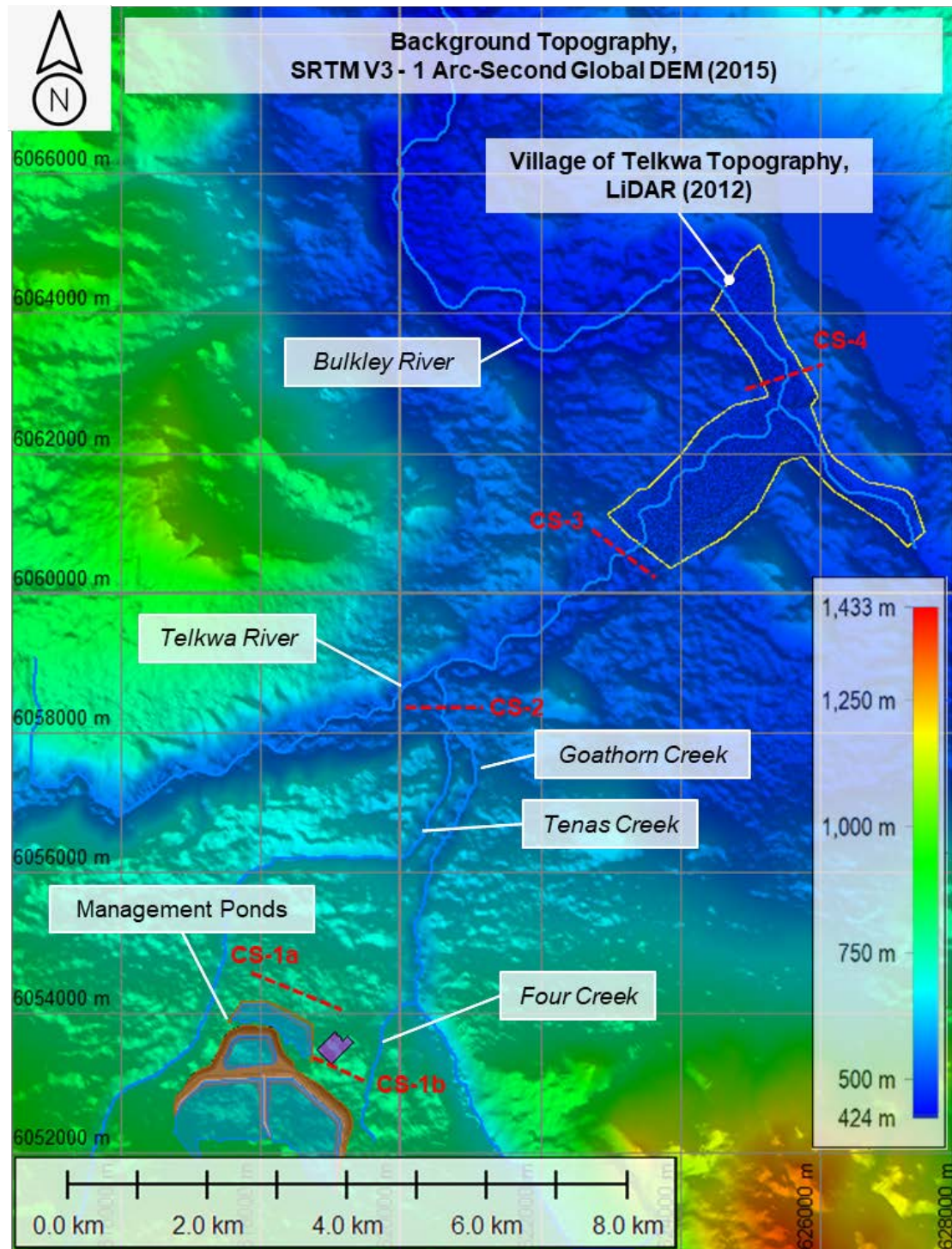
Detailed 2020 LiDAR topography for the site was provided by TCL, with a resolution of 1 m, which was used to develop pond storage-elevation curves and determine fill elevations in the facilities. To guarantee

full coverage of the computational area, the SRTM (Shuttle Radar Topography Mission) 1 arc-second global digital elevation model (DEM) Version 3.0 was used for the majority of the study area, including areas immediately downgradient of the breach locations and along streams flowing towards the Village of Telkwa (USGS, 2020).

For added resolution within the Village of Telkwa, the SRTM surface was merged with 2012 topography data which was used in a flood inundation study completed on behalf of the village by Golder Associates Inc. (2019), which has a resolution of 1 m.

The SRTM and 2012 topography data (Village of Telkwa, 2020) were reasonably well-matched, but some minor feathering was required to smoothly transition between the two surfaces. No other erroneous topographical features that would adversely affect the modelling were perceived within the data. The grid cell size chosen for the model was 50 m by 50 m, in order to accommodate the resolution of the SRTM data.

Figure 2-1: SRTM and 2012 Digital Elevation Data Extents



Sources: \\srk.ad\dfs\l\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

2.3 Hydrology

Peak stream flows are summarized in Table 2-2, and represent the reasonable worst-case natural flows in the five streams affected by the breaches. For streams that have tributaries that are also modelled in the list, the tributary flows are subtracted from the post-confluence total flow values, making the flows in Table 2-2 additive. This way, the streams may combine within the model without double-accounting for any flows.

Flows corresponding to the probable maximum flood (PMF) values are given for Four Creek, Goathorn Creek, and Tenas Creek, and were determined through rainfall-driven runoff modelling in HEC-HMS, consistent with methodologies outlined in the Tenas Project – Mine Site Water Management Design Technical Report (SRK 2021b) and Hydrometeorological Baseline Report (SRK 2021c). The 1-in-200 year flood events are used for the reasonable worst-case in the Telkwa and Bulkley Rivers in lieu of the PMF, and were determined with a frequency analysis of historical Water Survey of Canada records (SRK 2021c).

Table 2-2: Stream flow estimates for worst-case flood conditions

Return Period	Peak Flow (m ³ /s)
Tenas Creek: PMF	168.0
Goathorn Creek: PMF	335.0
Four Creek: PMF	30.0
Telkwa River: 1-in-200 year	561.0
Bulkley River: 1-in-200 year	982.4

Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam Break\Inputs\Inflow Hydrographs\DamBreak_inflowHydrographs.xlsx

In addition to the in-stream flow conditions, the catchment areas and associated runoff contributions to the WMP and EMP were calculated based on inputs and assumptions consistent with SRK 2021b and 2021c, and added to the operating water cover to create the total stored water volume under worst-case conditions. Table 2-3 summarizes additional hydrological inputs applied to the breach configurations.

Table 2-3: Pond hydrological inputs

Parameter	Unit	Critical Failure Case #1 West Dam	Critical Failure Case #2 East Dam	Assumptions/Comments
1-in-100 wet year precipitation	mm	822		1-in-100 wet year precipitation
Worst-case contributing catchment area	km ²	2.4	4.2	Assumes upstream hydraulic structures have failed.
Conservative runoff coefficient	-	1	0.8	All pit runoff reports to the WMP. EMP receives majority of runoff from upstream natural catchments. Natural

Parameter	Unit	Critical Failure Case #1 West Dam	Critical Failure Case #2 East Dam	Assumptions/Comments
				runoff coefficient calculated to be 0.36 (SRK 2021c)
1-in-100 wet year inflow volume	m ³	1,600,000	2,900,000	Assumes volume from 1-in-100 wet year precipitation is added to total stored water in ponds.

Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Telkwa_DamBreach_Inputs_1CT017-003_rev04_nf.xlsm

2.4 Key Downstream Receptors

Table 2-4 provides descriptions for specific downstream receptors of particular concern for the dam breach analysis, categorized according to CDA (2013) guidelines.

Table 2-4: Key receptors assessment

Category	Receptor	Critical Failure Case	Comments
Population	Residents of the Village of Telkwa and surrounding areas	1, 2	The Village of Telkwa is located approximately 11 km downstream of the water management ponds. It is the closest township to the site, and likely to be at risk due to inundation from a breach. The village is situated at the confluence of the Telkwa and Bulkley Rivers, with buildings located within the river floodplains.
	TCL Personnel	2	TCL personnel are anticipated to operate within the Mine Infrastructure Complex area at all times, and would be at risk should the East Dam fail in its northeastern section. Approximately 80 people are expected to occupy the area at the worse case during shift change.
Environmental	Trees/Habitat	1, 2	The undeveloped inundation area outside of the creek channels generally consists of forested terrain. Restoration of the habitat would be possible.
	Fish and Aquatic Habitat	1, 2	The creeks downslope of the mine site, as well as the Telkwa and Bulkley Rivers, provide a habitat for various fish and other aquatic species.
Cultural	Residents of the Village of Telkwa and surrounding areas	1, 2	Regional heritage sites and tourism in the Village of Telkwa may be affected by a breach event.
Infrastructure	Road bridge over Bulkley River (Telkwa Coalmine Rd to Riverside St crossing)	1, 2	A bridge crosses the Bulkley River and is located close to the junction of the Telkwa and Bulkley Rivers. It is the traversing location for vehicular traffic within the Village of Telkwa.

Category	Receptor	Critical Failure Case	Comments
	Railway bridge over Telkwa River	1, 2	A railway bridge crosses the Telkwa River close to the junction of the Telkwa and Bulkley Rivers. The railway serves both freight and public transport trains.
	Aveling/Telkwa Coalmine Rd	1, 2	Access road running alongside the Telkwa River to the Village of Telkwa.
	Highway 16	1, 2	Freeway alongside the Bulkley River, through the Village of Telkwa.
	Town roads	1, 2	Non-major roadways within the Village of Telkwa.
	Buildings and residential homes	1, 2	Encompasses the residential, commercial, and industrial edifices near and within the Village of Telkwa.

Sources: Compiled in text.

3 Dam Breach Model Development

The dam breach assessment was modelled using FLO-2D software (FLO-2D, 2019). The software applies the finite difference technique for overland flow routing by means of mass and momentum conservation. The outputs from the model can also be exported as shapefiles that can be viewed in GIS programs.

The development of the model comprised the following steps detailed in this section:

- Determination of the failure modes and breach locations for the critical cases
- Estimation of the volume released in the breach
- Valuation of the breach parameters and outflow hydrographs
- Routing of the downstream flood

3.1 Failure Modes and Breach Locations

In accordance with the CDA Guidelines (2013), the dam breaches and the associated discharge hydrographs are evaluated under two hydrologic conditions:

- **“Sunny-day” failure** – “a sudden dam failure that occurs during normal operations”, which may be “caused by internal erosion, piping, earthquakes, mis-operation leading to overtopping, or another event”.
- **Flood-induced or “rainy-day” failure** – “a dam failure resulting from a natural flood of a magnitude that is greater than what the dam can safely pass”.

There are several credible failure modes for embankment dams, the most common of which are overtopping failures due to inadequate spillway capacities and/or large flood event, piping failures due to concentrated internal erosion of the dam, and foundation-related failures such as slippage or settlement. The failure modes have been assumed based on knowledge of the dam structures, construction schedules, and engineering judgement and previous dam failures that have occurred.

Along the lengths of the embankments, the breach locations were chosen to achieve the highest incremental effects for the key receptors. For the West, North, and Tenas Dams, the breach location must be in the north to north-east portions to simulate the cascade failure case that eventually reports to the Village of Telkwa. The breach location for the East Dam was chosen for the greatest impact to the MIC, which is the northeastern section of the dam.

3.1.1 Sunny-Day Breach

A sunny-day failure occurs during normal operations, with the most likely failure mode for the dams being piping failure. The likelihood of foundational failures in the management pond dams is further mitigated by the structural support from the buttresses, and the spillways are to be designed to prevent overtopping from flood events that are credibly expected to occur during operations.

The most likely failure location is typically near the dam crest, and/or where the embankment slopes are steepest, as these areas would have the maximum hydraulic gradient for free water retained in the facilities. The breach is not expected to extend significantly below the elevation of the rock fill in the ponds, therefore the maximum breach height is approximated by the depth of the free water in the pond.

The sunny day breach scenarios assume that mean annual flow conditions are occurring in the catchments with flow in the streams, whose flood extents are accounted for in the topography.

3.1.2 Rainy-Day Breach

The rainy-day breach occurs under conditions of exceptional flow due to large storm events. In the event of flows exceeding the spillway design criteria, the ponds would overtop the embankments resulting in overtopping failure. Piping failure is also a possibility, but the breach in either case would originate at or close to the dam crest.

In order to assess the incremental effects of a breach, two rainy day scenarios are modelled – both will have PMF or 1-in-200 year flows (summarized in Table 2-1). One model includes the breach while the other does not. Comparing the two sets of results allows separation of breach-related effects from storm event-related effects.

3.2 Breach Volumes

The critical failure conditions for each dam in the two main failure cases are summarized in this subsection, which includes cross-sectional diagrams of the dams with free water depths relative to the dam heights.

3.2.1 Critical Failure Case #1

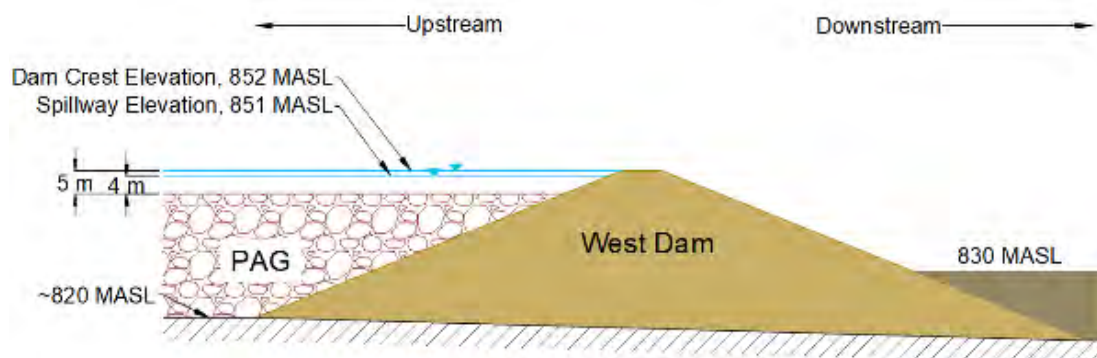
West Dam

Construction of the WMP is not expected to begin until later in the mine life, after which PAG filling and dam raising will occur incrementally until the dam is completed. According to the mine plan, the maximum volume of free water in the pond occurs after the dam has been raised to the maximum elevation.

Under typical operating conditions, the completed WMP (Figure 3-1) is not expected to have more than 2 m of water cover over the PAG waste rock, and lengthy periods of time without pump-controlled discharge from the pond is unlikely to occur. The maximum volume with the highest hydraulic gradient occurs when the stored water is at the spillway elevation. For the purpose of modelling critical conditions, an additional volume of water equal to the total runoff from upslope catchments during a 1-in-100 wet year was included in the facility addition to the 2 m typical water cover.

The total breach height in the West Dam with the additional 100-year wet runoff contributions (Table 2-3) is 4 m for the sunny day failure, corresponding to approximately 4.0 million cubic metres (Mm^3) of free water in the WMP. The breach height for the rainy-day overtopping failure is 5 m (5.1 Mm^3 stored), which occurs when the water level reaches and overtops the dam crest elevation during a large flood event.

Figure 3-1: West Dam cross-section in the critical case



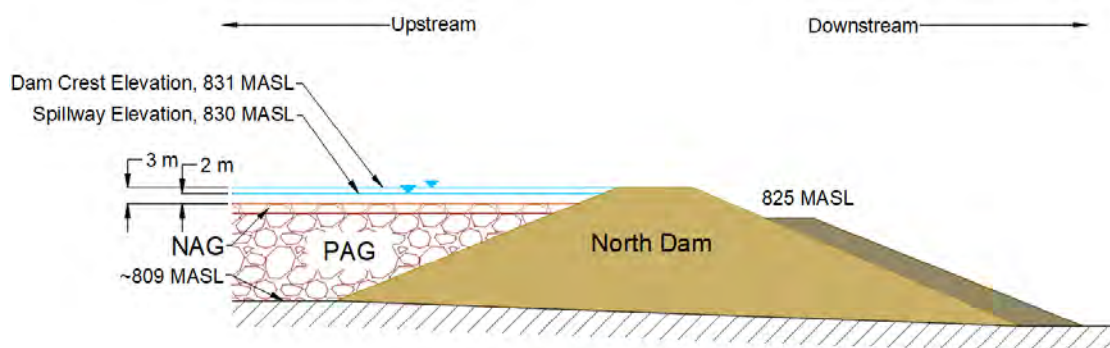
Sources: \\srk.ad\dfs\na\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\!080_Deliverables\Dam Breach Assessment Memo\040_Figures\Dam Cross-Sections

North Dam

At the time of completion for the WMP, the filling of the NMP will have already been completed, including NAG rock cover over the PAG waste rock. For the failure case, it is assumed that the planned overburden cover will not have been placed in the facility, allowing a larger volume of water to be stored in the NMP and therefore released during the breach scenario.

The breach height for the sunny-day failure will comprise the water cover of 2 m (0.56 Mm^3), which will be 3 m (0.88 Mm^3) for the rainy-day failure.

Figure 3-2: North Dam cross-section in the critical case



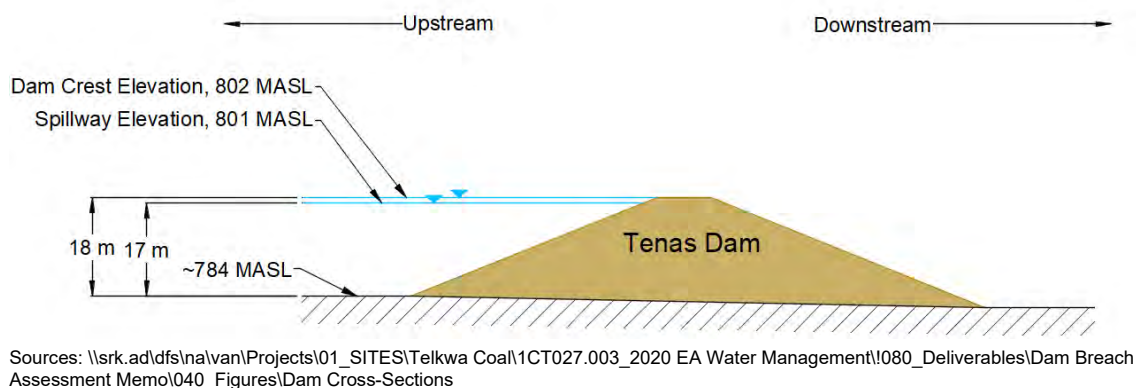
Sources: \\srk.ad\dfs\na\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\!080_Deliverables\Dam Breach Assessment Memo\040_Figures\Dam Cross-Sections

Tenas Control Pond Dam

It is assumed that the water level in the TCP is up to the spillway elevation at the start of a sunny day failure. Since TCP is a settling pond, there will be no rock storage resulting in maximum breach height equal to the entire height of the dam.

The sunny and rainy-day breach heights are therefore 17 m (1.9 Mm³) and 18 m (2.3 Mm³) respectively.

Figure 3-3: Tenas Dam cross-section in the critical case



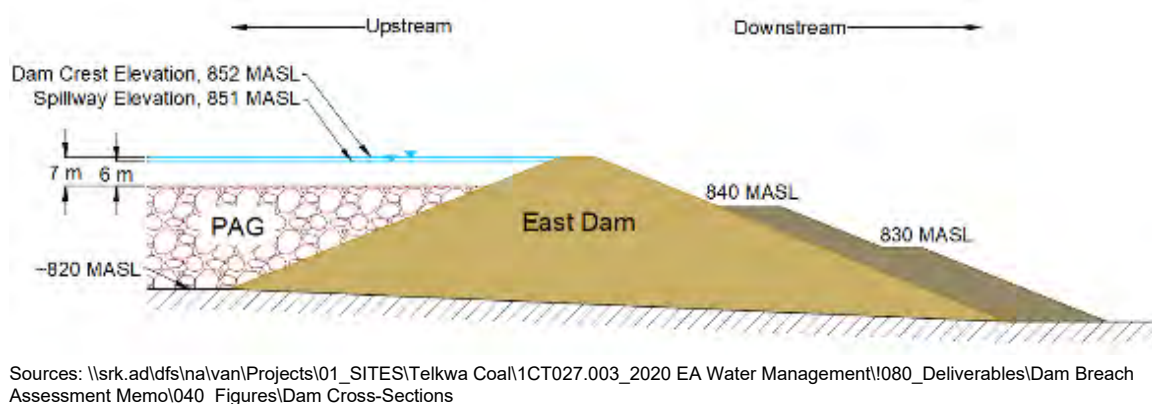
3.2.2 Critical Failure Case #2

East Dam

The second failure case assesses the failure of the East Dam towards the MIC. The approach to determining the critical failure conditions for the East Dam is the same as for the West Dam, where 2m of water cover and one year of 1-in-100 wet year runoff is stored at the time of breach (Table 2-3).

It then follows that the sunny and rainy-day breach heights are 6 m (6.3 Mm³) and 7 m (7.4 Mm³) respectively.

Figure 3-4: East Dam cross-section in the critical case



3.3 Dam Breach Hydrographs

The dam breach hydrographs are determined from parameters calculated using empirical equations that were developed through the assessment of case histories from past water-retaining embankment breaches. The equations from Froehlich (1995) and Froehlich (2008) were applied, and the resulting breach parameters for the sunny day breach are presented in Table 3-1, and the rainy day breach parameters are presented in Table 3-2.

The peak flows are relatively high due to the large volumes of water released. The formation times are slower (up to 2 hours) for breaches with relatively high volumes but relatively low breach heights, and faster for facilities with relatively low volumes with relatively high breach heights. The Tenas Dam can be expected to empty in as little as 12 to 30 minutes. The rainy-day peak flows are higher than the sunny-day flows, with a shorter breach formation time.

Table 3-1: Sunny day dam breach parameters

Parameter	Unit	Critical Failure Case #1			Critical Failure Case #2
		West Dam	North Dam	Tenas Dam	East Dam
Dam Type		Rock-filled	Rock-filled	Rock-filled	Rock-filled
Failure mode		Piping	Piping	Piping	Piping
Breach height	m	4	2	17	6
Free water volume	m ³	4,042,200	561,200	1,936,600	6,259,300
Individual dam breach peak flow	m ³ /s	300	71	1500	570
Total breach case peak flow ¹	m ³ /s	1871			570
Formation Time ²	hr	1.8 (2.8)	1.1 (2.1)	0.2 (0.5)	1.5 (2.3)

Sources: \\srk.ad\dfs\l\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Telkwa_DamBreach_Inputs_1CT017-003_rev04_nf.xlsm

Notes:

¹ Peak flow for cascade failure assumed to equal peak flow from all contributing dams.

² Approximate value from fitting peak flow and volume to hydrograph is presented; calculated value from empirical equations in parenthesis.

Table 3-2: Rainy day dam breach parameters

Parameter	Unit	Critical Failure Case #1			Critical Failure Case #2
		West Dam	North Dam	Tenas Dam	East Dam
Dam Type		Rock-filled	Rock-filled	Rock-filled	Rock-filled
Failure mode		Overtopping	Overtopping	Overtopping	Overtopping
Breach height	m	5	3	18	7
Free water volume	m ³	5,103,100	879,400	2,292,100	7,394,600
Individual dam breach peak flow	m ³ /s	430	130	1600	720

Parameter	Unit	Critical Failure Case #1			Critical Failure Case #2
		West Dam	North Dam	Tenas Dam	East Dam
Total breach case peak flow ¹	m ³ /s	2160			720
Formation Time ²	hr	1.6 (2.5)	0.9 (1.8)	0.2 (0.5)	1.4 (2.2)

Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Telkwa_DamBreach_Inputs_1CT017-003_rev04_nf.xlsm

Notes:

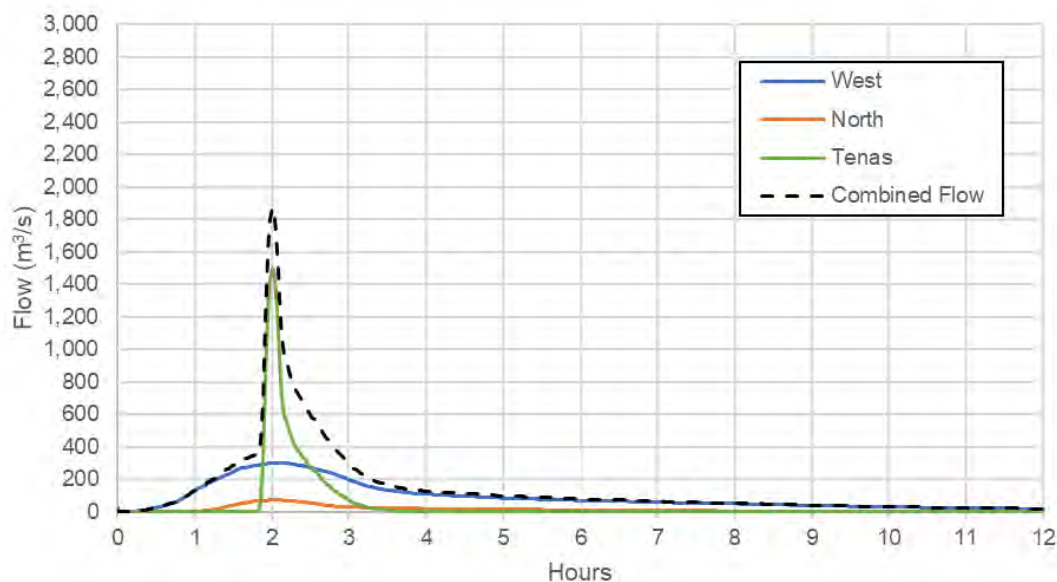
¹ Peak flow for cascade failure assumed to equal peak flow from all contributing dams.

² Approximate value from fitting peak flow and volume to hydrograph is presented, calculated value from empirical equations in parenthesis.

The breach hydrographs were prepared using an approximate log-normal distribution fitted to match the total volume released and calculated peak flow. The hydrographs representing the cascade failures were obtained by super-imposing the hydrographs from the contributing dams with the peaks aligned (Figure 3-5 for sunny-day; Figure 3-6 for rainy-day).

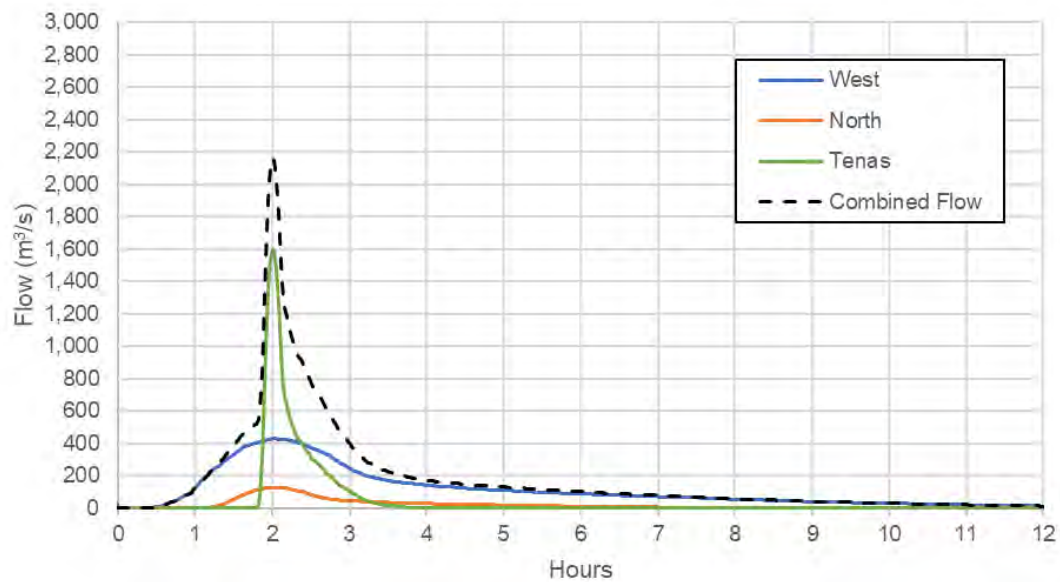
The hydrographs for the sunny and rainy-day failures for the East Dam are plotted in Figure 3-7.

Figure 3-5: Sunny day breach outflow hydrograph for Critical Failure Case #1



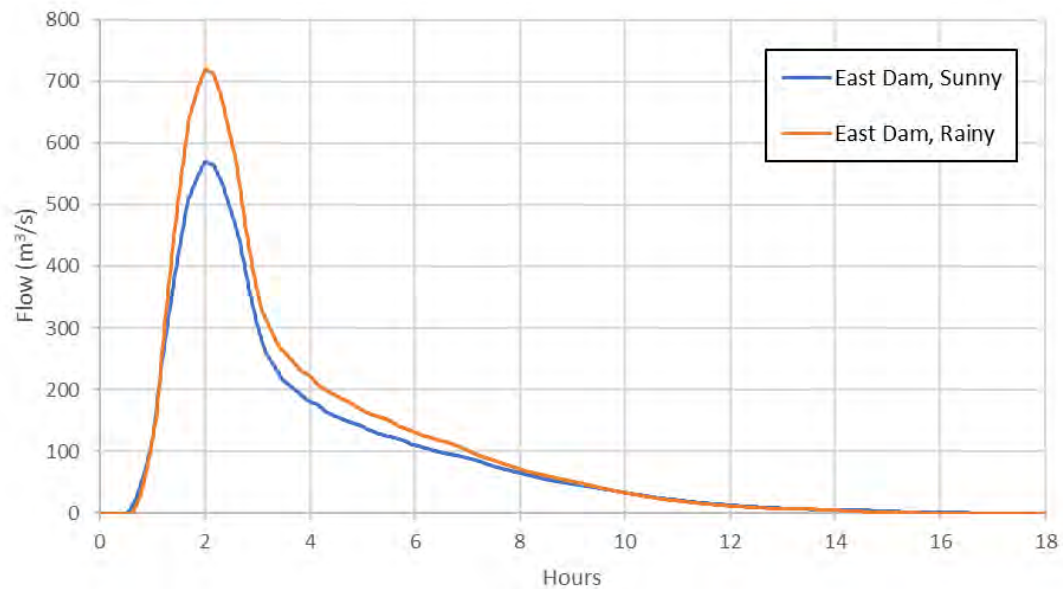
Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Breach Hydrographs\Base_Case\Telkwa_BC_Breach_Spreadsheet_1CT027-003_rev04_nf.xlsx

Figure 3-6: Rainy day breach outflow hydrograph for Critical Failure Case #1



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Breach Hydrographs\Base_Case\Telkwa_BC_Breach_Spreadsheet_1CT027-003_rev04_nf.xlsx

Figure 3-7: Sunny and rainy day breach outflow hydrographs for Critical Failure Case #2



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Breach Hydrographs\Base_Case\Telkwa_BC_Breach_Spreadsheet_1CT027-003_rev04_nf.xlsx

3.4 Flood Route Modelling

3.4.1 Boundary Conditions

Preliminary simulations were run in order to estimate inundation limits, from which the size of the computational area could be adjusted. Overall, the model extents allow for the flood flows from the mine site to reach the Village of Telkwa, as well as provide buffer distance for the stream inflows to be applied. The inundation extents within the modeled area are generally determined by topography, with some effects from surface roughness. FLO-2D 'Outflow elements' are assigned to the downstream of the Bulkley River, past the Village of Telkwa.

3.4.2 Hydraulic structures

Hydraulic structures are not explicitly included in the analysis for the flood route modelling. The spillways are omitted in order to prescribe the breach locations along the dams. Bridges located in the Village of Telkwa are considered only when assessing breach effects on infrastructure.

3.4.3 Surface Roughness

The areas along the inundation pathway were classified into zones for surface roughness (Figure 3-8), quantified by Manning's n values. The values were defined using recommendations from the FLO-2D PRO Reference Manual (2019) and are summarized in Table 3-3. The overland flow roughness coefficients are considerably higher than those used for open channel flow.

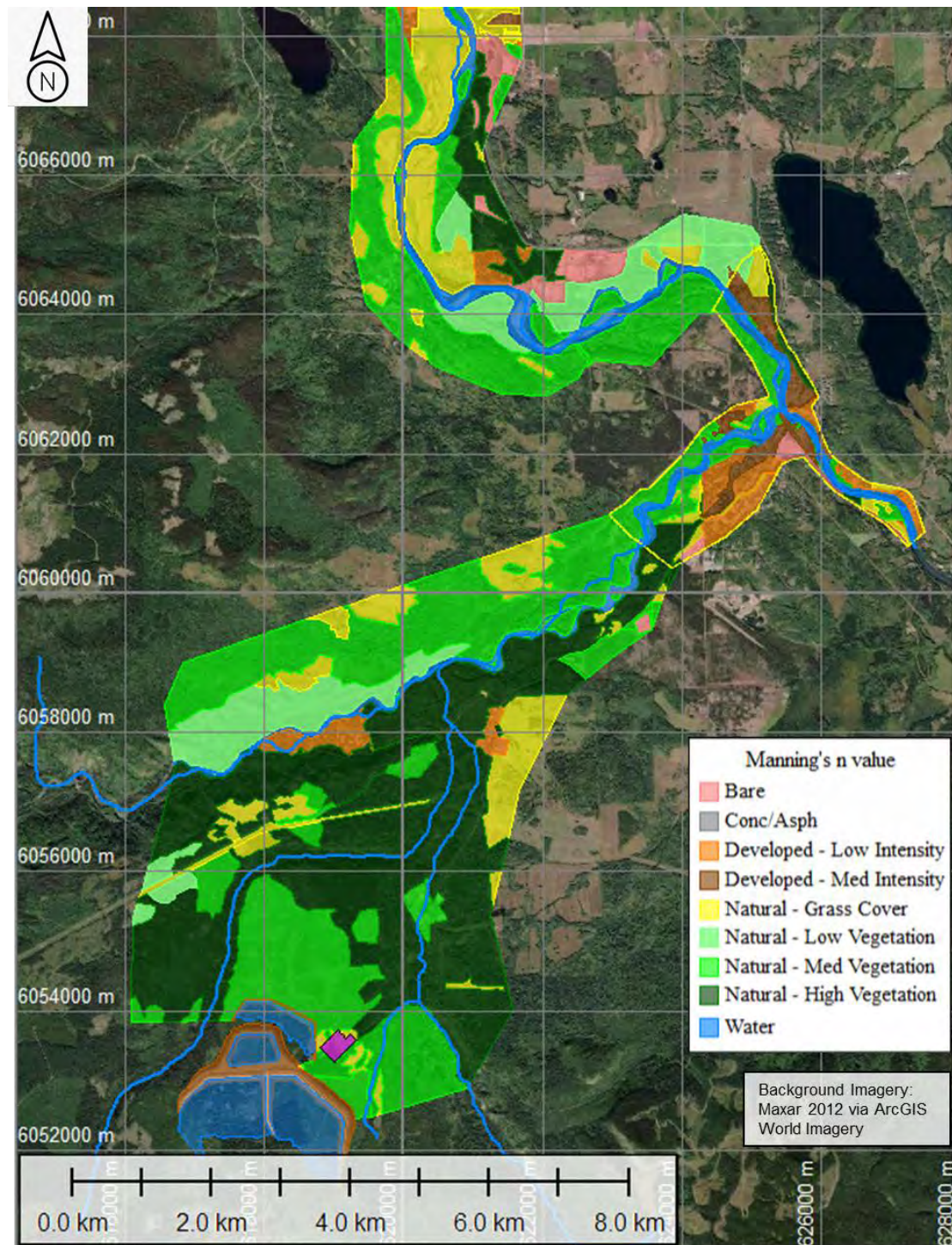
Sensitivity analyses were performed by varying the Manning's n values by +/- 50% and showed that surface roughness has little impact on the results compared to other modeled factors. The results of the sensitivity analysis are not reported in this document for brevity.

Table 3-3: Manning's n values used in the dam breach analysis

Zone Description	Manning's n value
Open ground, no debris	0.04
Concrete/Asphalt	0.02
Developed - Low Intensity	0.30
Developed - Medium Intensity	0.35
Natural - Low Vegetation	0.20
Natural - Medium Vegetation	0.30
Natural - High Vegetation	0.35
Natural - Grass Cover	0.16
Water	0.04

Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Inflow Hydrographs\DamBreak_inflowHydrographs.xlsx

Figure 3-8: Surface roughness zone assignment in computational area



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\1080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

3.4.4 Model Inflow Hydrographs

The combined breach hydrograph for the cascade failure (Figure 3-5 and Figure 3-6) were applied to the northern section of the Tenas Dam. The breach hydrographs for the East Dam failure (Figure 3-7) was applied to the northeastern section of the dam.

The stream peak inflow was applied as constant values for each creek or river, at locations upstream of any confluences.

3.4.5 Control Cross-Sections

Five cross-sections (dashed red lines in Figure 2-1) were created to assess the downstream effects of the flood wave. The cross-sections are detailed in Table 3-4 and are numbered upstream to downstream (south to north). CS-1a, CS-2, CS-3, and CS-4 are used for Critical Failure Case #1, and CS-1b is used for Critical Failure Case #2. Time series plots showing flow width and depth at each cross-section for each simulation scenario are provided in the appendices with the maximum flow depth contour plots. Time series for flow velocity and discharge rates are shown with the maximum flow velocity contour plots.

Table 3-4: Cross-section IDs and locations

Cross-Section ID	Location Description	Distance Downstream of Breach Location
CS-1a	Directly downgradient of Tenas Dam breach location	0 km
CS-1b ¹	Directly downgradient of East Dam breach location	0 km
CS-2	Tenas Creek, before confluence with Telkwa River	5.7 km
CS-3	Telkwa River, at south end of Village of Telkwa	9.5 km
CS-4	Bulkley River, after confluence with Telkwa River	13.1 km

Sources: Compiled in text.

Notes:

¹ Cross-section includes Four Creek inflow.

4 Results

The results from the following scenarios are discussed in this section:

1. Critical Failure Case #1
 - a. Rainy day (PMF in creeks, 200-year flood in rivers) with no breach
 - b. Sunny day with cascade breach at the northern embankment of Tenas Dam
 - c. Rainy day with cascade breach at the northern embankment of Tenas Dam
2. Critical Failure Case #2
 - a. Sunny day breach at the northeastern embankment of East Dam
 - b. Rainy day breach at the northeastern embankment of East Dam

Inundation maps for each scenario are provided in Appendix A through E, which include the following:

- Maximum flow depth – relative to starting condition
- Maximum flow velocity
- Flood hazard maps
- Arrival time (sunny day scenarios) - relative to the start of the model, and not peak flow or breach formation.

The flood hazard maps were produced for the purpose of assessing risk to human life. The hazard levels were assigned based recommendations from the Professional Practice Guidelines for flood mapping provided by the Association of Professional Engineers and Geoscientists BC (APEGBC, 2017). The hazard ratings (HRs) are a function of the depth and velocity of flood waters (Equation 4-1), and the categories are summarized in Table 4-1. Additional risks posed by the inclusion of debris in the flood waters is uncertain but assumed to be negligible for this study. The presence of debris could result in a significant increase in hazard rating.

$$HR = d * (v + 0.5) + DF \quad \text{Equation 4-1}$$

Sources: Equation derived from HR Wallingford (2006), provided by APEGBC (2017).

Notes: *HR* = flood hazard rating
d = depth of flooding (m)
v = velocity of floodwaters (metres per second (m/s))
DF = debris factor (0, 0.5, or 1 depending on probability that debris will lead to significantly greater hazard – 0 used)

Table 4-1: Flood hazard level to people based on hazard rating

Hazard Rating (HR)	Hazard to People Classification
< 0.75	Very Low Hazard (Caution)
0.75 – 1.25	Danger for Some (includes children, the elderly, and the infirm)
1.25 – 2.00	Danger for Most (includes the general public)

Hazard Rating (HR)	Hazard to People Classification
> 2.00	Danger for All (includes emergency services)

Sources: Classification system developed by Surendran et al. (2008), provided by APEGBC (2017).

4.1 Critical Failure Case #1

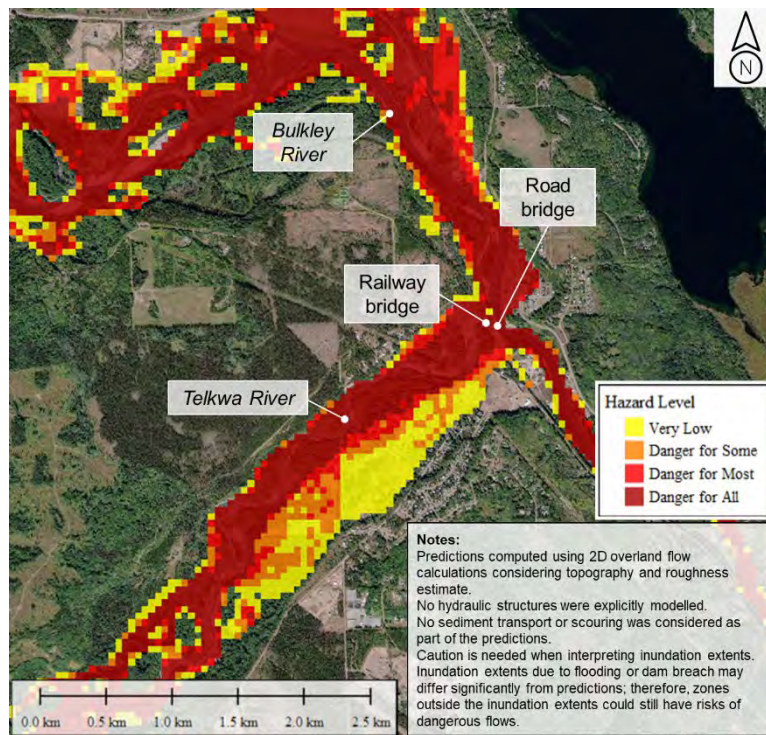
4.1.1 No Breach, Flood Event Scenario

Appendix A (Figures A.1 to A.3) provides inundation maps for maximum flow depths, maximum flow velocities, and flood hazard resulting from PMF conditions in the creeks and 200-year flood in the rivers (PMF/Q200). Maps for both the full study area and specifically the Village of Telkwa are shown. The CPP area is unaffected by stream flow during a rainy-day scenario with no breach.

According to the prediction as shown in Figure 4-1, the southern portion of the Village of Telkwa would be largely inundated during a PMF/Q200 event, but depth predictions do not generally exceed 1 m, nor do the predicted flows exceed 0.2 m/s, making the hazard mostly 'Very Low' or a 'Danger for Some' along the Telkwa River section. The risk to humans and infrastructure in the northern section of Telkwa is greater, where flows are predicted to be as high as 5 m, though velocities are relatively low at below 0.3 m/s in most areas.

Flows depths in the rivers near both the road bridge and railway bridge are predicted to exceed average depths by up to 9 m, with velocities close to 4 m/s. Therefore, bridge damage may occur during a PMF/Q200 event without a dam breach.

Figure 4-1: Rainy day, no breach maximum flood hazard in the Village of Telkwa



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

Notes: Background Imagery courtesy of Maxar 2012 via ArcGIS World Imagery

The inundation maps in Appendix A show that despite significantly higher flows in the streams, the flow is generally confined to within the existing creek or riverbanks in areas with less human development, such as towards the proposed mine site. There is some minor flooding in the clearing close to the confluence of Goathorn Creek with Tenas Creek, where there is some development and infrastructure

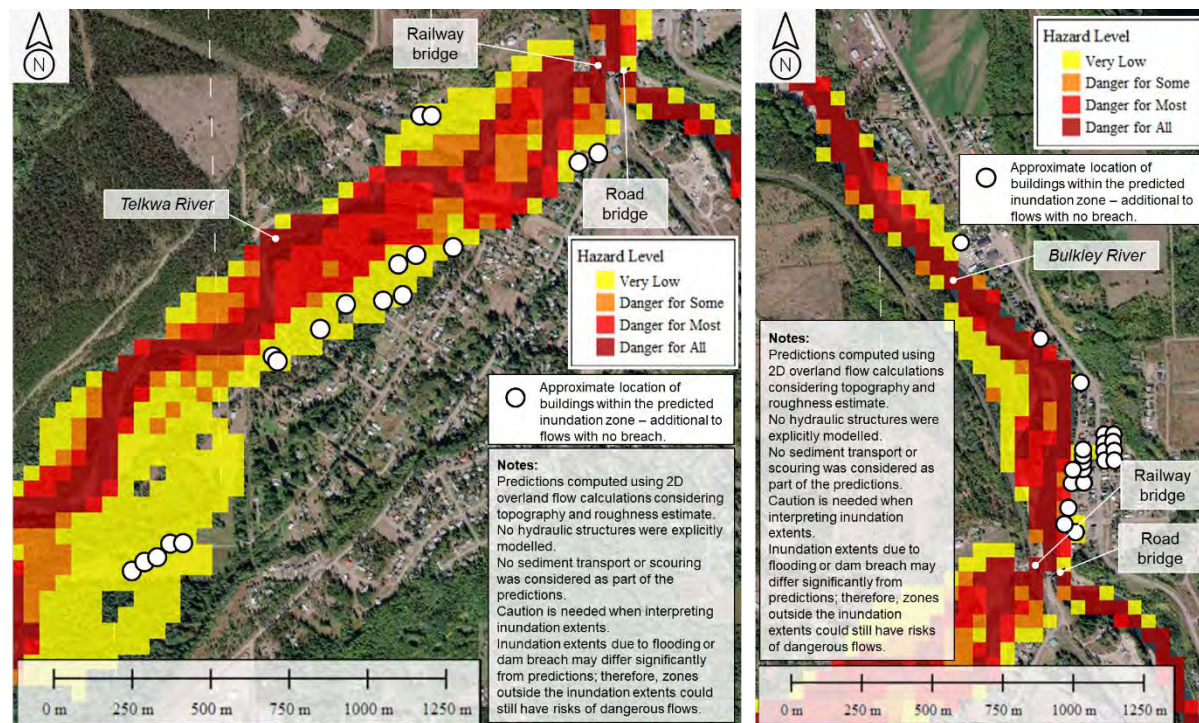
4.1.2 Sunny-Day Breach Scenario

Appendix B (Figures B.1 to B.4) provides inundation maps for maximum flow depths, maximum flow velocities, flood hazard, and arrival time for the sunny day cascade breach.

The predicted inundation area is not as extensive as that of the no breach rainy-day scenario, with most flows being confined to the existing riverbanks. There is some low hazard flooding (> 1 m depth, approximately 0.3 m/s velocity) of the predominantly uninhabited area at the southern tip the Village of Telkwa, as well as some contained inundation in the northern section (Figure 4-2). It is estimated that approximately 120 people would be affected during the sunny-day breach with the assumption of 3 persons per building, though the predicted flood hazard is considered is 'Very Low' for the vast majority of those at risk.

Bridge damage may occur, but the likelihood is much lower than those of the PMF/Q200 scenario as the predicted water depth at the bridges is greater than 3 m, with a velocity of greater than 2 m/s.

Figure 4-2: Sunny-day cascade breach flood hazard in Telkwa south (left) and north (right)



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\1080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

Notes: Background Imagery courtesy of Maxar 2012 via ArcGIS World Imagery.
Building count is an approximation used to estimate the population at risk.

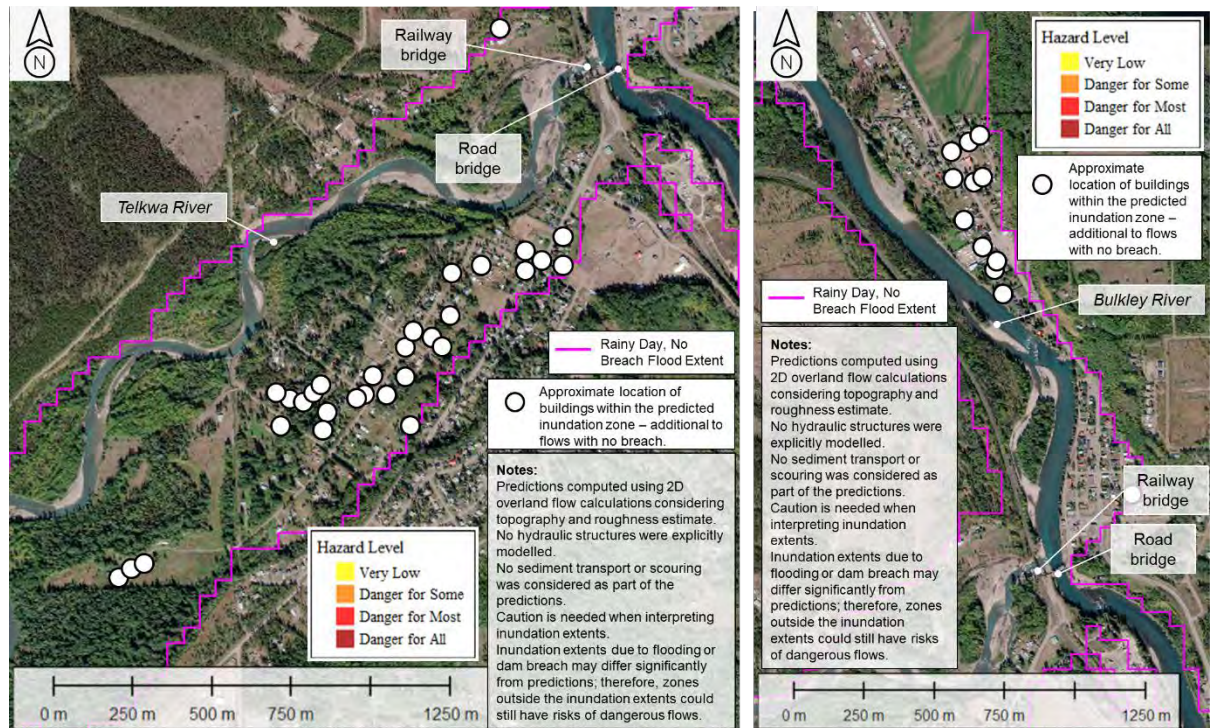
The greatest inundation effects of the sunny day breach occur in the natural area immediately north of the breach location, but there the risk to human life is lower since there is little to no permanent or temporary occupancy of that area.

4.1.3 Rainy Day Breach Scenario

Appendix C (Figures C.1 to C.3) provides inundation maps for maximum flow depths, maximum flow velocities, and flood hazard for the rainy-day cascade breach.

The inundation extents are very similar to the rainy-day with no breach scenario, and therefore the effects of the events are similar. Figure 4-3 shows locations of incremental hazard level resulting from the prediction of rainy-day breach as compared to the rainy-day, no breach scenario. The predictions suggest that approximately 150 people will be within an increased hazard level as compared to the rainy-day, no breach scenario.

Figure 4-3: Rainy-day cascade breach flood zones of increased hazard in Telkwa south (left) and north (right)



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\1080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

Notes: Background Imagery courtesy of Maxar 2012 via ArcGIS World Imagery.
Building count is an approximation used to estimate the population at risk.

The flooding is most extensive in the area directly downgradient of the breach location and is greater than of the sunny day breach. However, the population at risk directly downgradient of the breach is minimal compared to the town of Telkwa.

4.1.4 Discussion of Results

The incremental effects of the sunny-day breach are evaluated with respect to average flow conditions while the incremental effects of the rainy-day breach are compared to the rainy-day with no breach scenario. The resulting incremental population at risk is similar for both the sunny and rainy-day scenarios. The buildings that may be affected with a sunny day breach are predominantly located within the floodplains or near the banks of the river and are expected to experience flooding with a relatively low hazard level.

Though the incremental effects of the rainy-day scenarios introduce limited additional extent to the predicted inundation zone as compared to the rainy-day with no breach event, the hazard level in select areas increases to 'Danger for Some' and 'Danger for Most' from lower levels of hazard. The predicted incremental effects of the rainy-day breach (according to hazard rating) are more pronounced in locations

further away from the river banks, where there are slight increases in flow depth (up to 1 m) and velocity (up to 0.1 m/s).

The arrival times for either case (Table 4-2) are driven in part by the stream flows that convey the water released from the breach towards the Village of Telkwa. The predicted flood wave from the rainy-day breach arrives, on average, three hours earlier than that of the sunny day breach due to the PMF and 200-year flows from contributing streams and rivers.

Table 4-2: Time of arrival of the flood wave to the Village of Telkwa in Critical Failure Case #1

Time of Arrival	Sunday Day Breach	Rainy Day Breach
	6 to 7 hours	3 to 4 hours

Sources: \\srk.ad\dfs\alvan\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Telkwa_DamBreach_Inputs_1CT017-003_rev04_nf.xlsm

Notes: Time of arrival corresponds to time since start of the model, and not necessarily time of formation for the breach. Peak flows from the dam breach are reached approximately 2 hours after model start time.

4.2 Critical Failure Case #2

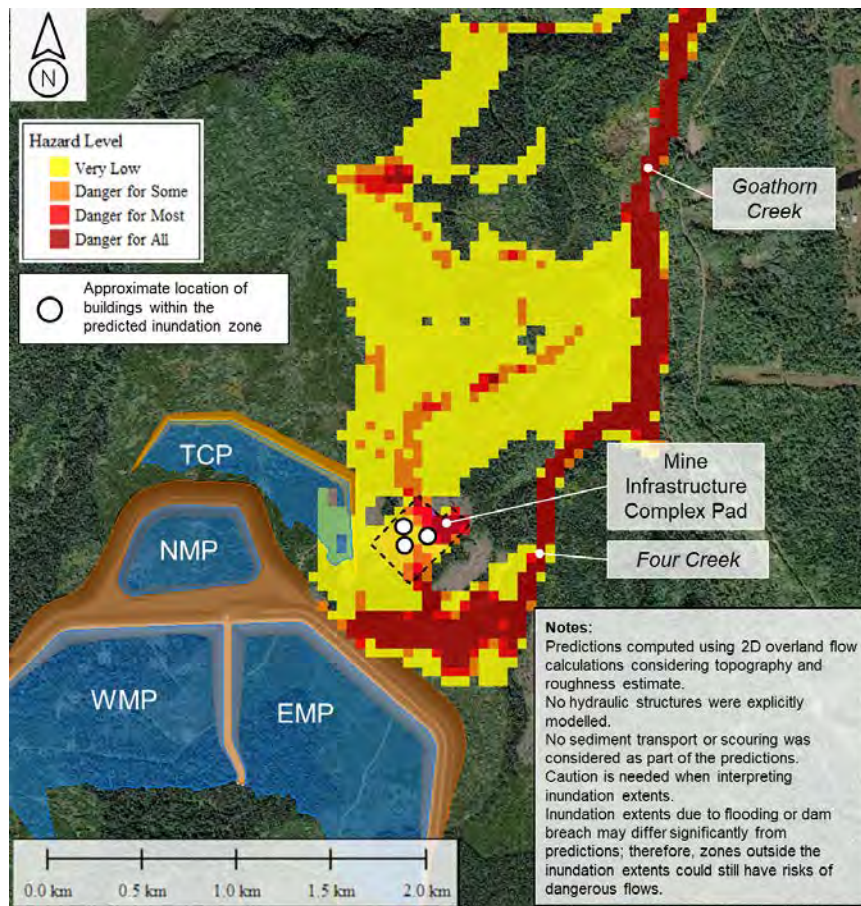
4.2.1 Sunny-Day Breach Scenario

Appendix D (Figures D.1 to D.4) provides inundation maps for maximum flow depths, maximum flow velocities, flood hazard, and time of arrival for the sunny day breach of the East Dam.

The sunny day breach of the East Dam affects firstly the TCL personnel working at the MIC, of which there are approximately 28 present for 24 hours a day and up to 80 personnel present at shift changes that occur twice a day for about 1 hour each. The predicted inundation hazard is 'Very Low' for the southwestern half of the MIC's Pad, where two of the buildings will be located (Figure 4-4), and may be 'Dangerous to Some' towards the northeastern part of the MIC's Pad, where there is another building. The CPP sedimentation pond is also located at the northeast portion of the MIC's Pad. The inundation depth (Figure D.1 in Appendix D) is fairly uniform at 0.4 m throughout the Pad. The increased hazard in some areas of the MIC's Pad are mainly due to slightly higher flows (Figure D.2 in Appendix D) driven by upgradient topography.

Aside from the MIC, the free water contained in the EMP is comparable to that released from the three ponds in the cascade failure, and the majority of the water bypasses the Pad and instead flows directly towards Four Creek and Goathorn Creek. As such, an East Dam sunny day breach affects the Village of Telkwa to a lesser but similar degree as the cascade sunny day breach. Furthermore, the clearing around the confluence of Goathorn Creek and the Telkwa River is also inundated.

Figure 4-4: Sunny-day East Dam breach flood hazard at the MIC Pad



Sources: \\srk.ad\dfs\alvan\Projects\01_SITES\Telkwa Coal\1CT027-003_2020 EA Water Management\1080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

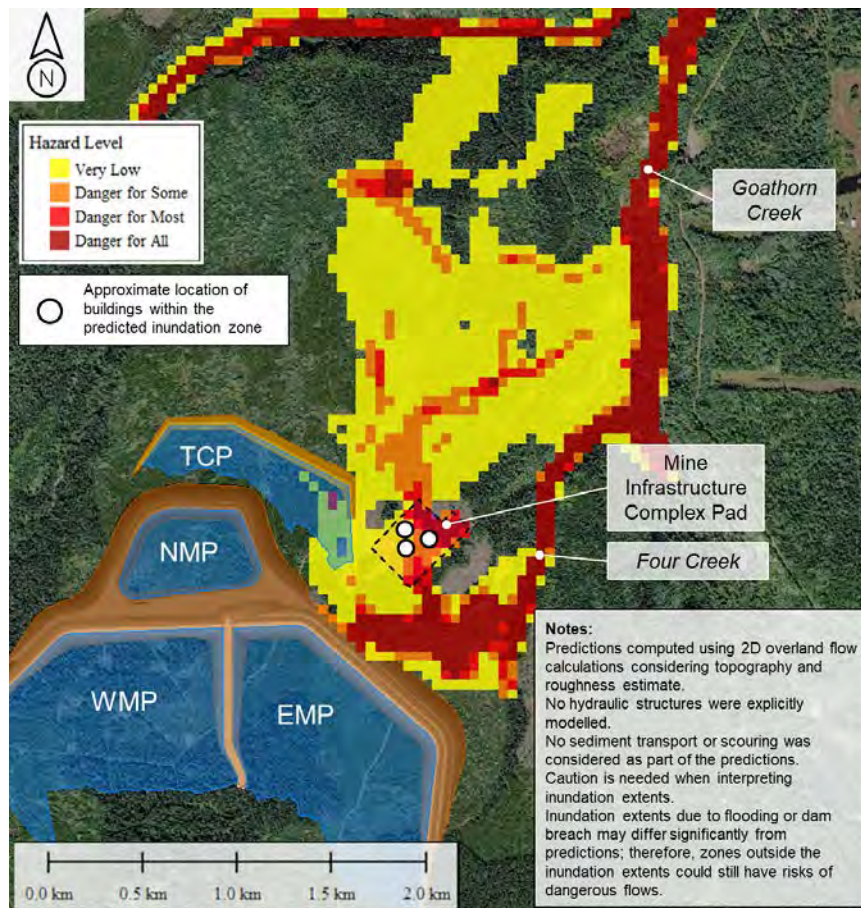
Notes: Background Imagery courtesy of Maxar 2012 via ArcGIS World Imagery.

4.2.2 Rainy-Day Breach Scenario

Appendix E (Figures E.1 to E.3) provides inundation maps for maximum flow depths, maximum flow velocities, and flood hazard for the rainy-day breach of the East Dam.

The rainy-day breach of the East Dam covers the same area of the CPP, but with an elevated hazard in the northeastern areas, including some locations with 'Danger for Some' or 'Danger for Most'. Likewise, the effect at the Goathorn and Telkwa confluence and the effects in the Village of Telkwa are slightly less significant, but similar to the rainy-day cascade failure.

Figure 4-5: Rainy day East Dam breach flood hazard at the MIC Pad



Sources: \\srk.ad\dfs\in\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\1080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

Notes: Background Imagery courtesy of Maxar 2012 via ArcGIS World Imagery.

4.2.3 Discussion of Results

The sunny and rainy-day scenarios both inundate the CPP, with the difference being the hazard level, with the rainy-day scenario generating greater risk to personnel. The time of arrival (Table 4-3) of the flood wave to the MIC is approximately 1.5 hours for both scenarios, relative to model start time. It takes the flood wave about an hour longer to reach the Village of Telkwa in the sunny day East Dam breach than with the sunny day cascade breach, though the arrival time is similar during a rainy-day for both breach events. Based on the flooding extents, it is possible that greater than 200 additional people may be put at risk during a breach of the East Dam, compared with if there was no breach.

Table 4-3: Time of arrival of the flood wave in Critical Failure Case #2

Time of Arrival	Sunny Day Breach	Rainy Day Breach
To the MIC	<1.5 hours	<1.5 hours
To the Village of Telkwa	7 to 8 hours	3 to 4 hours

Sources: \\srk.ad\dfs\Inalvan\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\800_Dam_Break\Inputs\Telkwa_DamBreach_Inputs_1CT017-003_rev04_nf.xlsm

Notes: Time of arrival corresponds to time since start of the model, and not necessarily time of formation for the breach. Peak flows from the dam breach are reached approximately 2 hours after model start time.

4.3 Sensitivity Analysis

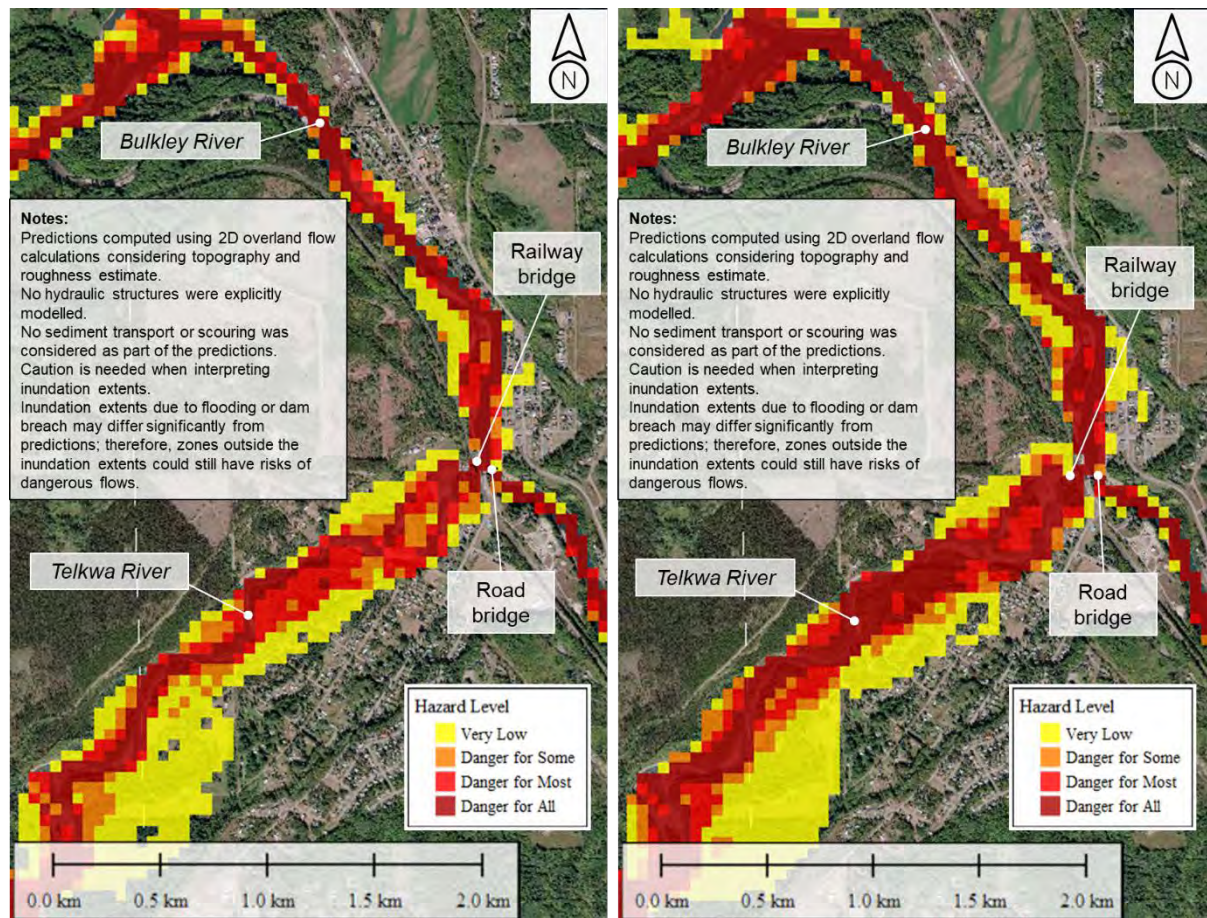
A sensitivity analysis was performed by simulating the sunny day cascade breach with 50% higher peak flows in the breach outflow hydrographs. The Telkwa River and post-confluence section of the Bulkley River show higher flows within the streams. The inundated area along the floodplains can extend up to approximately 100 m farther, nearly doubling the population at risk for the sunny day case.

The flow depths for the inundated areas of the Village of Telkwa are between 0.5 to 0.75 m greater, and the velocities are approximately 1 m/s faster. For the sunny day case, the depth and velocity increases are still unlikely to cause damage at the bridges. Some scouring and damage to the bridge is likely during the rainy-day event, regardless of whether a dam breach occurs since the majority of flow comes from precipitation rather than the reservoirs.

The flow hazards are compared in Figure 4-6 and show changes proportional to the according to the increasing depth and velocity. Hazard levels enter into the 'Danger for Most' and 'Danger for All' for those within the previous inundation area, and newer areas become assigned with a 'Very Low' or 'Danger for Some' rating.

Consideration should be given to uncertainties in model predictions if referring to them during emergency response planning. Variation in breach peak flows and volumes may cause underpredictions of inundation extent, which should be accounted for.

Figure 4-6: Telkwa hazards using original peak flows (left) and higher peak flows (right)



Sources: \\srk.ad\dfs\al\van\Projects\01_SITES\Telkwa Coal\1CT027.003_2020 EA Water Management\1080_Deliverables\Dam Breach Assessment Memo\040_Figures\Telkwa_DamBreak_Figs_1CT027-003_rev00_nf.pptx

Notes: Background Imagery courtesy of Maxar 2012 via ArcGIS World Imagery.

5 Consequence Identification and Dam Classification

The dam consequence classifications are assessed according to CDA Guidelines (2013) and supported by the Global International Standards on Tailings Management (GISTM) (ICMM, 2020).

Table 5-1 is a reproduction of the CDA-recommended consequence classification criteria and Table 5-2 is a reproduction of the GISTM criteria. The classification systems are based on the incremental consequences of a dam failure, as opposed to total consequences. The incremental consequences of failure are defined as the total damage from an event with dam failure, less the damage that would have resulted from the same event (e.g. a large earthquake or a large flood event) had the dam not failed. During the dam classification process, each of the four hazard rating components in Table 5-1 are considered individually, and the overall dam hazard rating is defined by the component with the highest (i.e. most severe) rating. The GISTM descriptions are used as a supplemental reference as the incremental losses are defined in more detail, though the ponds are not tailings management facilities.

Table 5-1: CDA (2013) Dam Classifications

Dam Class	Population at Risk ¹	Incremental Losses		
		Loss of Life ²	Environmental and Cultural Values	Infrastructure and Economics
Low	None	0	Minimal short-term loss	Low economic losses; area contains limited infrastructure or services
Significant	Temporary only	Unspecified	No long-term loss	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	No significant loss or deterioration of fish or wildlife habitat	High economic losses affecting infrastructure, public transportation, and commercial facilities
Very High	Permanent	100 or fewer	Loss of marginal habitat only	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Restoration or compensation in kind highly possible	Extreme losses affecting critical infrastructure or service (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)

Notes:

¹ Definitions for population at risk:

- None: There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.
- Temporary: People are only temporarily in the dam-breach inundation zone (e.g., seasonal cottage use, passing through on transportation routes, participating in recreational activities).
- Permanent: The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); Three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is carried out).

² Implications for loss of life:

- Unspecified: The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season.

Table 5-2: GISTM (2020) Dam Classifications

Dam Class	Population at Risk	Incremental Losses			
		Loss of Life ¹	Environment	Health, Social and Cultural	Infrastructure and Economics
Low	None	None expected	Minimal short-term loss or deterioration of habitat or rare and endangered species.	Minimal effects and disruption of business and livelihoods. No measurable effect on human health. No disruption of heritage, recreation, community or cultural assets.	Low economic losses: area contains limited infrastructure or services. <US\$1M.
Significant	1-10	Unspecified	No significant loss or deterioration of habitat. Potential contamination of livestock/fauna water supply with no health effects. Process water low potential toxicity. Tailings not potentially acid generating and have low neutral leaching potential. Restoration possible within 1 to 5 years.	Significant disruption of business, service or social dislocation. Low likelihood of loss of regional heritage, recreation, community, or cultural assets. Low likelihood of health effects.	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes. <US\$10M.
High	10-100	Possible (1 - 10)	Significant loss or deterioration of critical habitat or rare and endangered species. Potential contamination of livestock/ fauna water supply with no health effects. Process water moderately toxic. Low potential for acid rock drainage or metal leaching effects of released tailings. Potential area of impact 10 km ² – 20 km ² . Restoration possible but difficult and could take > 5 years.	500-1,000 people affected by disruption of business, services or social dislocation. Disruption of regional heritage, recreation, community or cultural assets. Potential for short term human health effects.	High economic losses affecting infrastructure, public transportation, and commercial facilities, or employment. Moderate relocation/compensation to communities. <US\$100M.
Very High	100-1,000	Likely (10 - 100)	Major loss or deterioration of critical habitat or rare and endangered species. Process water highly toxic. High potential for acid rock drainage or metal leaching effects from released tailings. Potential area of impact > 20 km ² . Restoration or compensation possible but very difficult and requires a long time (5 years to 20 years).	1,000 people affected by disruption of business, services or social dislocation for more than one year. Significant loss of national heritage, community or cultural assets. Potential for significant long-term human health effects.	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities, for dangerous substances), or employment. High relocation/ compensation to communities. < US\$1B.
Extreme	> 1,000	Many (> 100)	Catastrophic loss of critical habitat or rare and endangered species. Process water highly toxic. Very high potential for acid rock drainage or metal leaching effects from released tailings. Potential area of impact > 20 km ² . Restoration or compensation in kind impossible or requires a very long time (> 20 years).	5,000 people affected by disruption of business, services or social dislocation for years. Significant National heritage or community facilities or cultural assets destroyed. Potential for severe and/or long- term human health effects.	Extreme economic losses affecting critical infrastructure or services, (e.g., hospital, major industrial complex, major storage facilities for dangerous substances) or employment. Very high relocation/compensation to communities and very high social readjustment costs. >US\$1B.

Notes: ¹ Implications for loss of life:

Unspecified: The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season

5.1 Population at Risk/Loss of Life

5.1.1 West, North, and Tenas Dams

The sunny and rainy-day failures both contribute to flood-related dangers to residents in the Village of Telkwa. There are many residencies located within the floodplains of a sunny day breach, or for which a rainy-day breach would increase risk. In the event of a cascade breach, approximately 150 people may be affected, possibly more to account for model uncertainty.

A 'Very High' consequence rating is recommended for all three dams.

5.1.2 East Dam

The breach of the East Dam may put all personnel at the CPP at risk, as well as those along at the junction of Goathorn Creek and the Telkwa River, and in the Village of Telkwa. Up to 200 people are predicted to be affected by the EMP failure.

A 'Very High' consequence rating is recommended for the East Dam.

5.2 Environmental and Cultural

5.2.1 West, North, and Tenas Dams

The greatest impact to the terrestrial environment is located in an area under 10 square kilometres (km²) directly downslope of the Tenas Dam. The area is forested; habitat loss will be moderate and possible to restore after either the sunny or rainy-day event. Aquatic life in the streams will not experience significant effects from sediment deposition related to the breach in the long term, but the sudden high flows may temporarily damage fish habitat in the sunny-day breach. The effect of the rainy-day breach on aquatic habitat is low given the PMF/Q200 conditions already present.

PAG material and coal rejects are stored in the management ponds, and therefore there is potential for species from acid rock drainage (ARD) to be present in the free water released in a breach. As a mitigating factor, the NMP will have a NAG cover in the critical breach scenario, and the TCP does not contain waste material other than sediments accumulated during the operational period.

There are no known culturally significant artifacts in the area, though some sites significant to regional heritage sites are located within the Village of Telkwa.

A 'Very High' consequence rating is recommended for the West Dam as it is the primary source of water released in the breach and may also have poorer water quality. A 'High' consequence rating is recommended for the North and Tenas Dams.

5.2.2 East Dam

The primary impact to the terrestrial environment from the breach of the East Dam is also directly downslope of the breach. However, the majority of water released travels directly to Goathorn and Four Creeks, making the inundation of the forested area relatively low impact.

The impact to aquatic environment in the creeks may be moderate to significant due to the high addition of water, but the habitat can likely be restored. Similar to the WMP, there may be some impact from ARD.

A 'High' to 'Very High' consequence rating is recommended for the East Dam.

5.3 Infrastructure and Economics

5.3.1 West, North, and Tenas Dams

The road bridge and the railway bridge are both located at the junction of the Telkwa and Bulkley Rivers. The railway bridge is most at risk, as it is directly in the path of the flood wave. The road bridge is located over Bulkley River prior to the confluence but is likely to be affected due to backflow up the Bulkley River due to displacement caused by the flood wave. The rainy-day breach is unlikely to cause significantly more damage to the bridges than the PMF/Q200 flows due to similar predicted depths and velocities in the area, and the sunny day breach may cause minor or moderate damage.

Freeways, major access roads, and local roads may be temporarily unusable due to inundation or debris accumulation. Costs may be incurred from reparations for damage due to scouring, and debris removal.

Disruption to businesses and tourism such as those related to fishing would have an effect on local economy. Property damage requiring compensation to buildings located within the inundation area, but very high economic losses are not considered likely.

A 'High' consequence rating is recommended for all three dams.

5.3.2 East Dam

The impact of the East Dam breach on infrastructure and economics is similar to that of the cascade breach.

A 'High' consequence rating is recommended for the East Dam.

5.4 Recommended Dam Classification

Based on the consequence evaluations provided in the previous subsections, a consequence of 'Very High' for all four dams is recommended.

6 Conclusions and Recommendations

A dam breach assessment was completed to simulate flooding for two critical failure cases. The first critical case considers a cascade failure of the West Management Pond, North Management Pond, and Tenas Control Pond. The second critical case considers the failure of the East Management Pond. Sensitivity analyses were completed to check the variation due to changes in breach and modelling parameters.

Three scenarios were modelled to assist in evaluation of the potential effect of a dam breach on the downstream receptors for rainy-day and sunny-day conditions:

- The downstream creeks and rivers experiencing a large flood event with no dam breach (assuming 1 in 200-year flood for rivers and PMF for creeks)
- A Sunny-Day dam breach scenario (fair-weather scenario); and,
- A Rainy-Day dam breach scenario (flood-induced scenario).

The assessment suggests that the population at risk is up to approximately 200 for the sunny-day failure scenarios considered, with the majority of the buildings affected within the 'Very Low' to 'Danger for Some' hazard zones.

The assessment suggests there is significant risk to the majority of the population of the Village of Telkwa during a potential rainy-day event, but the danger is predominantly caused by the high flows due to the flood event rather than the dam breach. The predicted incremental population at risk is similar to the sunny-day breach scenario, but the resulting hazard is predominantly 'Danger for some' and 'Danger for Most'.

Terrestrial and aquatic habitats may be damaged during a breach event; however, clean up and restoration is possible.

Moderate damage to downstream infrastructure such as bridges, roadways, and some buildings may occur during a potential sunny-day breach, while the incremental effects of the rainy-day breach are generally less significant compared to the flood scenario with no breach.

Based on the dam breach assessment results, a 'Very High' consequence classification is recommended for the four embankments.

There are several sources of uncertainty that may impact model predictions. The inundation extents, depths, velocities, and arrival times from a breach event may vary significantly from that which is presented in this report. The overland flow calculations account for topography and estimates of surface roughness, but do not consider sediment transport or scouring. Hydraulic structures and infrastructure such as roadways and ditches were not explicitly modelled (though some larger features appear in the topography), therefore flow redirection unforeseen by the model presented in this report may occur.

Caution is needed when interpreting inundation predictions for the development of emergency response and action plans. A thorough review of topography, geology, hydraulic structures, and emergency travel routes should be undertaken when developing emergency response plans.

At the time of the study, there was limited detailed information on the grain size of the stored rock, which is currently assumed to be coarse in nature. The potential impacts of the liquefaction of coal rejects and waste material should be considered in later stages of design.

Note - the dam crest elevations and spillway inverts on the West and East Management dams as well as the Tenas Control Pond have been reduced after completion of the dam breach assessment. Both the EMP and WMP dam crests have been lowered to 850 masl, with spillways at 848 masl, and the Tenas Control Pond dam crest has been lowered to 800 masl with spillway invert of 798 masl. Due to the shapes of the ponds and associated stage-storage relationships, lower elevation crests and inverts correspond to smaller incremental volumes of water for a given water cover thickness, which would be expected to reduce the volume of water released during a dam breach. For example, the final storage volume in the TCP at the dam crest was reduced from 2.3 Mm³ to 1.6 Mm³. As such, the results of this study were accepted as conservative for the purpose of dam classification and emergency response planning at this stage of the Project. An updated dam breach assessment is recommended for final design of all dams.

Closure

This report, Tenas Project Dam Breach Assessment Report, was prepared by

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All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

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Appendix A Pond Elevation-Storage Tables

Table A-1: Summary Elevations

Pond	Dam Crest (MASL)	Spillway Invert (MASL)
Tenas Dam	802	801
North Dam	831	830
East Dam	852	851
West Dam	852	851

Table A-2: Pond Elevation-Storage

Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
752	-	-	0	-
752.5	-	-	0	-
753	-	-	0	-
753.5	-	-	120	-
754	-	-	820	-
754.5	-	-	2190	-
755	-	-	4210	-
755.5	-	-	6850	-
756	-	-	10210	-
756.5	-	-	14510	-
757	-	-	20040	-
757.5	-	-	26840	-
758	-	-	35430	-
758.5	-	-	46210	-
759	-	-	58790	-
759.5	-	-	72960	-
760	-	-	88660	-
760.5	-	-	105820	-
761	-	-	124250	-
761.5	-	-	144010	-

Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
762	-	-	165080	-
762.5	-	-	187560	-
763	-	-	211480	-
763.5	-	-	236810	-
764	-	-	263540	-
764.5	-	-	291650	-
765	-	-	321140	-
765.5	-	-	351980	-
766	-	-	384160	-
766.5	-	-	417610	-
767	-	-	452220	-
767.5	-	-	487960	-
768	-	-	524810	-
768.5	-	-	562730	-
769	-	-	601750	-
769.5	-	-	641870	-
770	-	-	683050	-
770.5	-	-	725300	-
771	-	-	768590	-
771.5	-	-	812880	-
772	-	-	858170	-
772.5	-	-	904460	-
773	-	-	951750	-
773.5	-	-	1000050	-
774	-	-	1049390	-
774.5	-	-	1099780	-
775	-	-	1151240	-
775.5	-	-	1203790	-

Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
776	-	-	1257480	-
776.5	-	-	1314720	-
777	-	-	1373180	-
777.5	-	-	1432730	-
778	-	-	1493350	-
778.5	-	-	1555000	-
779	-	-	1617700	-
779.5	-	-	1681430	-
780	-	-	1746210	-
780.5	-	-	1812020	-
781	-	-	1878850	-
781.5	-	-	1946720	-
782	-	-	2015620	-
782.5	-	-	2085550	-
783	-	-	2156520	-
783.5	-	-	2228590	-
784	0	-	2301860	-
784.5	-	-	2376500	-
785	0	-	2452530	-
785.5	-	-	2530030	-
786	230	-	2609060	-
786.5	-	-	2689720	-
787	1390	-	2772020	-
787.5	-	-	2855930	-
788	4610	-	2941440	-
788.5	-	-	3028550	-
789	15300	-	3117300	-
789.5	-	-	3207700	-

Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
790	39250	-	3299860	-
790.5	-	-	3394020	-
791	79570	-	3490540	-
791.5	-	-	3589430	-
792	141540	-	3690700	-
792.5	-	-	3794530	-
793	226940	-	3901050	-
793.5	-	-	4009970	-
794	337760	-	4120670	-
794.5	-	-	4232960	-
795	475070	-	4346740	-
795.5	-	-	4461890	-
796	642340	-	4578070	-
796.5	-	-	4695150	-
797	839060	-	4813100	-
797.5	-	-	4931940	-
798	1064650	-	5051910	-
798.5	-	-	5173060	-
799	1323980	-	5295380	-
799.5	-	-	5418890	-
800	1615300	-	5543790	-
800.5	-	-	5678350	-
801	1936630	-	5814320	-
801.5	-	-	5951470	-
802	2292120	-	6089750	-
802.5	-	-	6229100	-
803	-	-	6369490	-
803.5	-	-	6510930	-

Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
804	-	-	6653510	-
804.5	-	-	6797290	-
805	-	-	6942380	-
805.5	-	-	7088900	-
806	-	-	7236800	-
806.5	-	-	7386060	-
807	-	-	7536660	-
807.5	-	-	7688540	-
808	-	-	7841640	-
808.5	-	-	7995900	-
809	-	0	8151210	-
809.5	-	0	8307540	-
810	-	100	8464840	-
810.5	-	3350	8623050	-
811	-	9050	8782160	-
811.5	-	17320	8942140	-
812	-	28380	9103130	-
812.5	-	60090	9272160	-
813	-	97360	9442170	-
813.5	-	139980	9612990	-
814	-	187870	9784550	-
814.5	-	246300	9956810	-
815	-	308920	10129790	-
815.5	-	375800	10303530	-
816	-	446910	10478180	-
816.5	-	525550	10654290	-
817	-	607390	10831700	-
817.5	-	692780	11010460	-

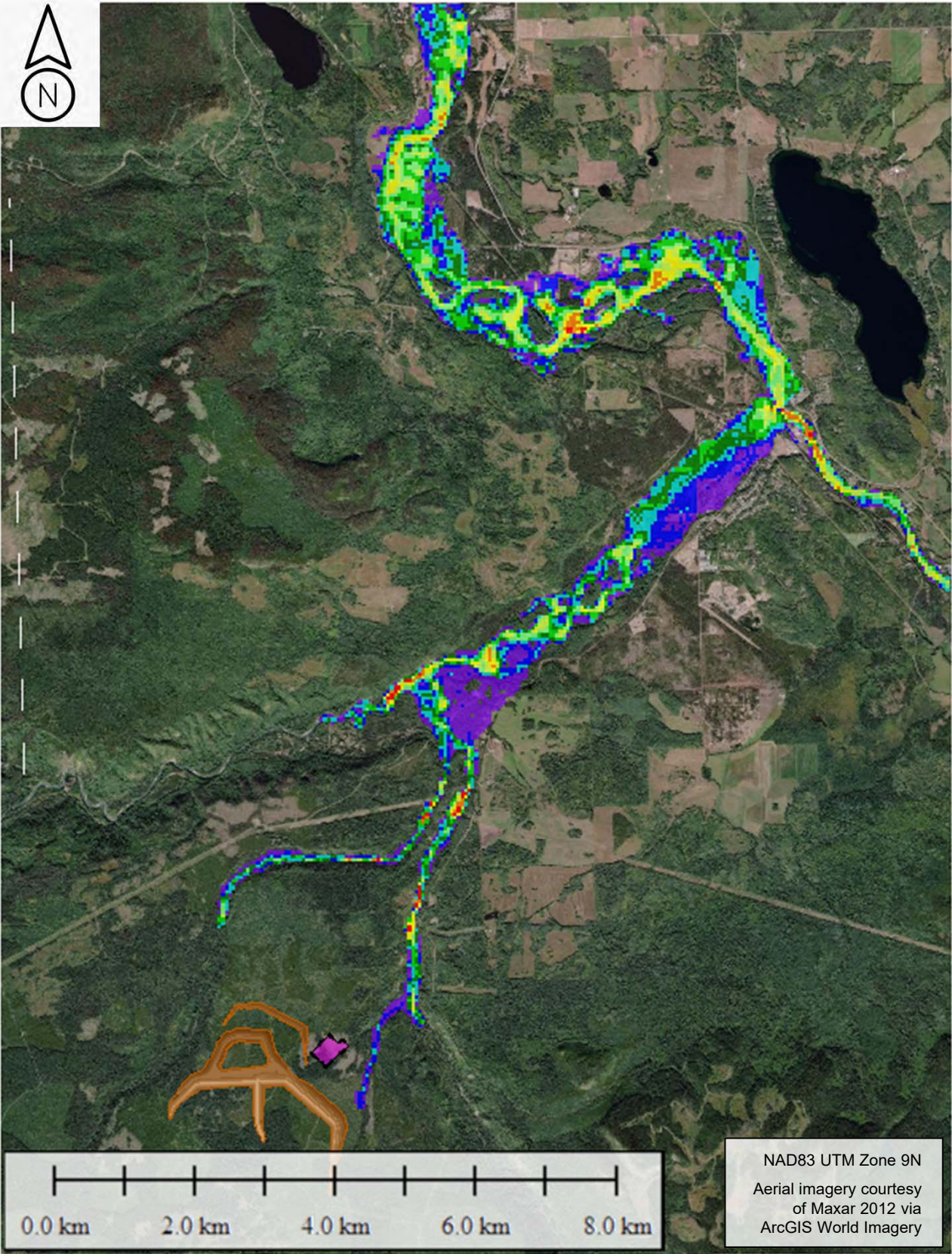
Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
818	-	782000	11190670	-
818.5	-	881020	11373620	-
819	-	983990	11558470	-
819.5	-	1090650	11745410	-
820	-	1200820	11934520	0
820.5	-	1316720	12127470	30
821	-	1435070	12323580	290
821.5	-	1555470	12523420	1000
822	-	1677560	12727910	2480
822.5	-	1800970	12940130	10530
823	-	1925590	13156820	22600
823.5	-	2051430	13378200	41410
824	-	2178490	13604830	68270
824.5	-	2306790	13859700	107850
825	-	2436310	14120330	154440
825.5	-	2567070	14386370	208130
826	-	2699070	14657870	269100
826.5	-	2832320	14937970	340900
827	-	2966830	15222750	419380
827.5	-	3102590	15512330	504360
828	-	3239610	15806980	595770
828.5	-	3377900	16110870	699130
829	-	3517480	16420000	809270
829.5	-	3658430	16734000	926100
830	-	3800820	17052600	1049630
830.5	-	3958230	17377830	1186010
831	-	4119020	17707750	1329520
831.5	-	-	18042250	1480120

Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
832	-	-	18381250	1637840
832.5	-	-	18725690	1814790
833	-	-	19074760	2000420
833.5	-	-	19428620	2195030
834	-	-	19787250	2399090
834.5	-	-	20152300	2624750
835	-	-	20522760	2860010
835.5	-	-	20898780	3105010
836	-	-	21280620	3360180
836.5	-	-	21680910	3638570
837	-	-	22088220	3924560
837.5	-	-	22501930	4218040
838	-	-	22921640	4519080
838.5	-	-	23350070	4842510
839	-	-	23782090	5173270
839.5	-	-	24217500	5511860
840	-	-	24656140	5858990
840.5	-	-	25098080	6235900
841	-	-	25543000	6620240
841.5	-	-	25990800	7012170
842	-	-	26441330	7412210
842.5	-	-	26895020	7838400
843	-	-	27351210	8273270
843.5	-	-	27809910	8717050
844	-	-	28271280	9170230
844.5	-	-	28735900	9642140
845	-	-	29203530	10120030
845.5	-	-	29674290	10603850

Elevation (MASL)	TCP Capacity (m³)	NMP Capacity (m³)	WMP Capacity (m³)	EMP Capacity (m³)
846	-	-	30148330	11093720
846.5	-	-	30626710	11592430
847	-	-	31108760	12097650
847.5	-	-	31594500	12609350
848	-	-	32084230	13127630
848.5	-	-	32584730	13653550
849	-	-	33089250	14185650
849.5	-	-	33597680	14723940
850	-	-	34110120	15268570
850.5	-	-	34628310	15820960
851	-	-	35151030	16379260
851.5	-	-	35678570	16943470
852	-	-	36211940	17514560

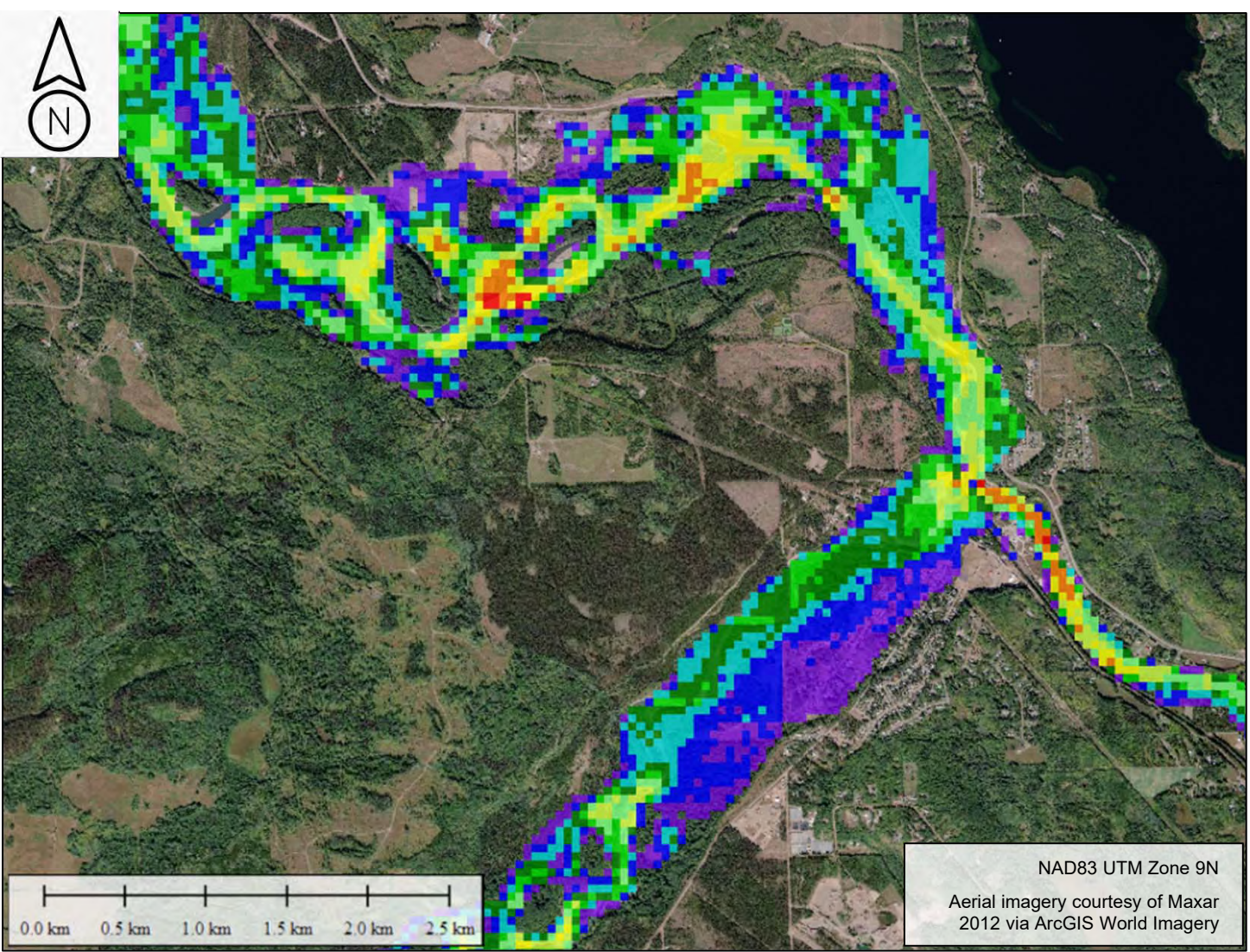
Appendix B No Breach, Rainy Day Scenario

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a) Overall Site Plan

Notes:
Predictions computed using 2D overland flow calculations considering topography and roughness estimate.
No hydraulic structures were explicitly modelled.
No sediment transport or scouring was considered as part of the predictions.
Caution is needed when interpreting inundation extents.
Inundation extents due to flooding or dam breach may differ significantly from predictions; therefore, zones outside the inundation extents could still have risks of dangerous flows.



b) Village of Telkwa Area



Job No: 1CT027.003
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Tenas Project – EA Water Management

Dam Break Assessment

**No Breach, Rainy Day (PMF/Q200)
Maximum Flow Depth**

Date: Jan. 2021	Prepared by: NF	Figure: B.1
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