REPORT

Nalunaq Gold Mine

Surface Water Infrastructure Design

Submitted to:

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1.0 INTRODUCTION

Nalunaq A/S ("the Company") has engaged WSP-Golder ("Golder") to provide technical support at its Nalunaq Gold Mine ("the Project") in southern Greenland. Following discovery of the Nalunaq mine in the early 1990s and development and operation by Crew Gold Corporation ("Crew Gold"), development was continued by Angus & Ross plc and Angel Mining (Gold) A/S, between 2004 and 2013. Subsequently additional exploration work has been undertaken in the Nalunaq area.

Golder has been contracted by the Company to provide support for the water and tailings management of its Project. As part of the Hydrology and Hydrogeology Assessment, Golder has prepared a Surface Water Management Strategy and drainage plan ("Drainage Plan"). This technical report outlines the conceptual designs for surface water diversion and surface water management for the site.

2.0 SITE DESCRIPTION

The Project is located in southern Greenland, approximately 35 kilometres (km) northeast of the town of Nanortalik, in the Municipality of Kujalleq. The mine lies on the northern slopes of Kirkespirdalen (Kirkespir Valley) around nine km from the eastern side of the Sarqå Fjord. The Project location is shown on Figure 1.

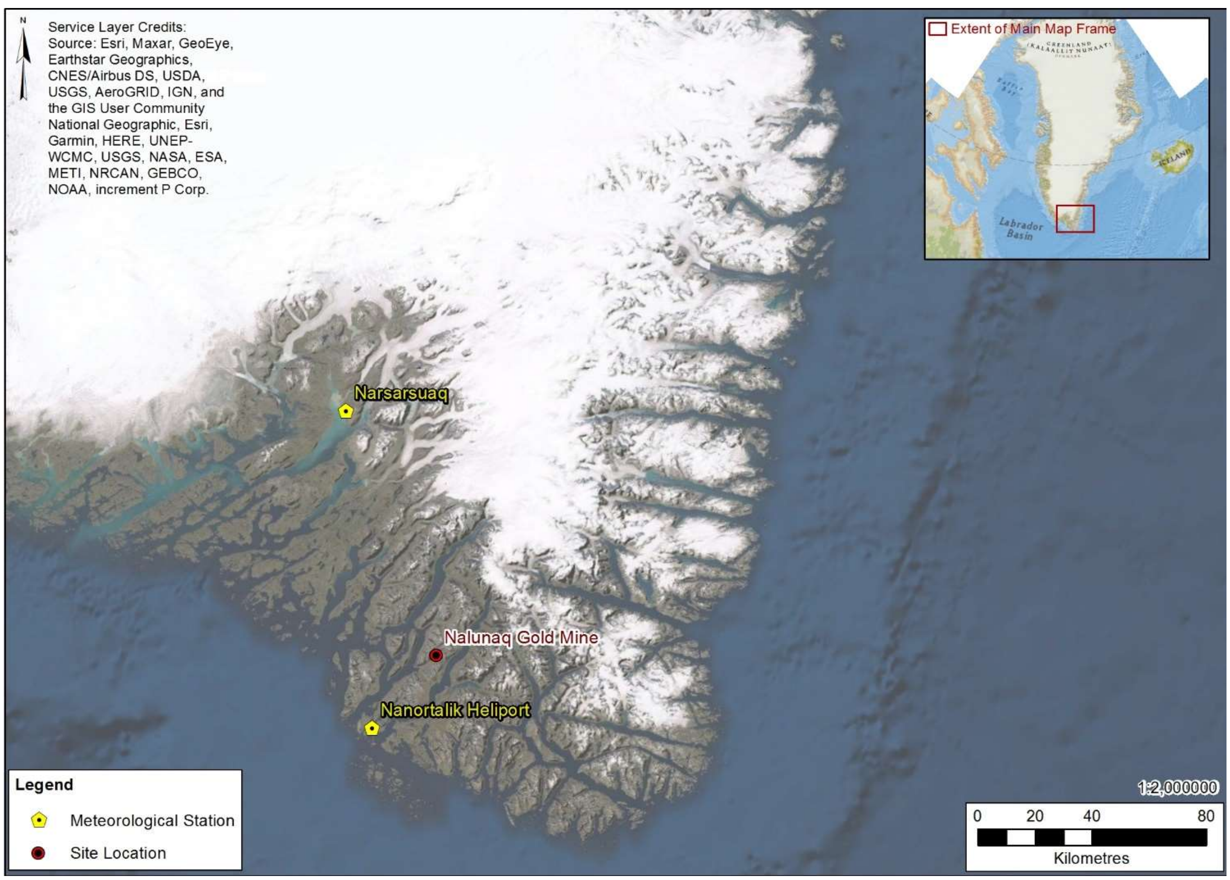


Figure 1: Project Location Plan

2.1 Proposed Site Layout

The mine facilities will consist of underground workings along the northern slopes of Kirkespirdalen, as well as several facilities along the valley bottom, namely a Dry Stack Tailings Storage Facility (DTSF), Process Plant and Ore Pad (Figure 2). The Kirkespir River flows as a braided network of streams across the valley floor, with the centreline of the main river channel currently aligned approximately 20 to 50 metres (m) away from the proposed facility layouts.

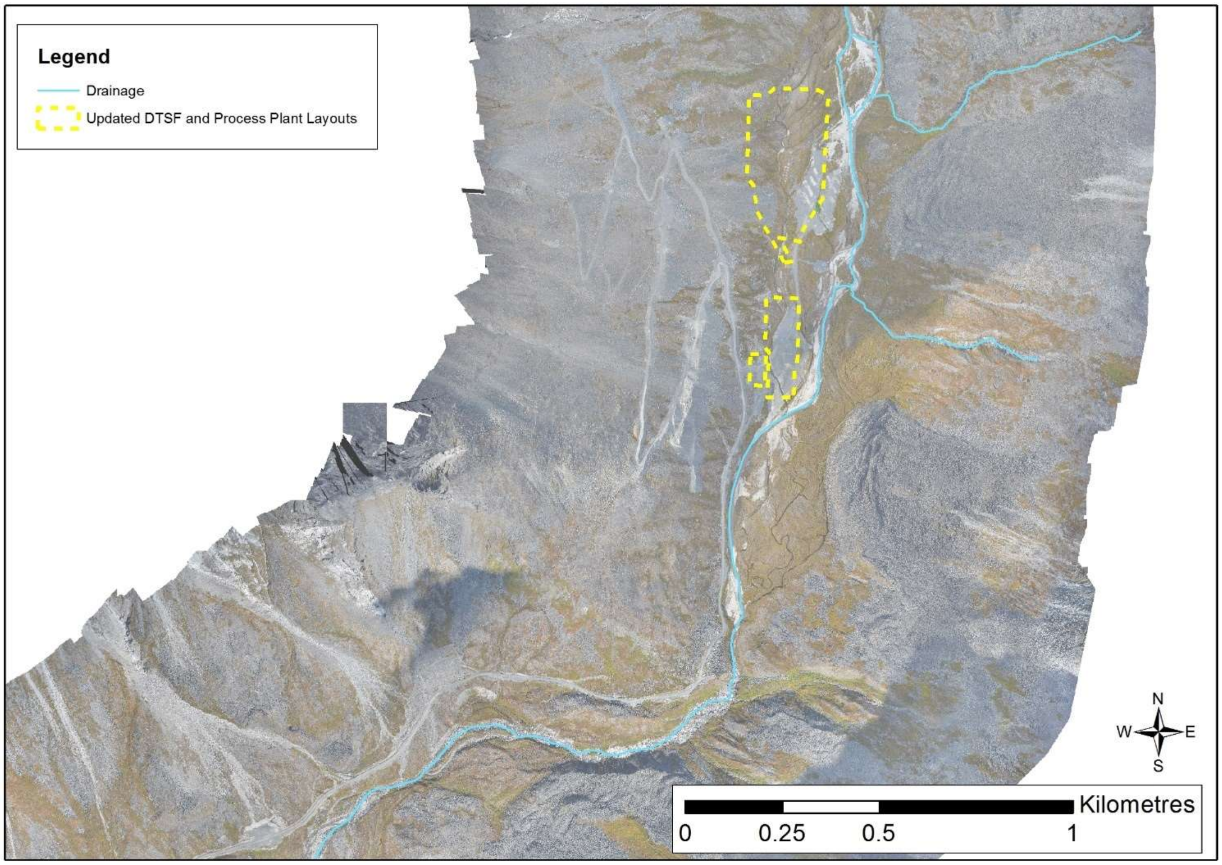


Figure 2: DTSF and Process Pad Layouts

3.0 CLIMATE ASSESSMENT

3.1 Climatic Setting

The Site location has a tundra climate with strong oceanic and polar influences (SRK Consulting, 2002). Precipitation (including both rainfall and snowfall) is moderate with available precipitation records for the region indicating an annual average cumulative depth of approximately 602 mm, as described below. It is noted however that precipitation within this unmetered area of the country is uncertain and anecdotal information from the Greenland Institute of Natural Resources (GINR) indicates that the cumulative annual precipitation depth may be as high as 900 mm. This report therefore considers the potential increase to design flow (and therefore flood height and flow velocity) arising as a result of an increase in precipitation from 602 mm to 900 mm per year, i.e. a 50% increase in design rainfall.

Snow cover is relatively limited within southern Greenland, with an average annual snowfall depth of 194 mm. Temperatures show relatively little variation between seasons. July is the hottest month with a mean temperature of 10.7 degrees Celsius (°C) and February is the coldest month with a mean temperature of -7.9°C.

3.2 Regional Climate Stations

There is no onsite meteorological station at the Site, with only short climate datasets available during which local data capture (e.g., rainfall) has been carried out as part of a specific site-based study. These are too short to be sufficient for hydrological analysis. As such, daily precipitation, and temperature data from two stations (Nanortalik Heliport and Narsarsuaq) were sourced from NOAA (2020) and Tutiempo (2020) respectively. The location of these stations relative to the Site are shown in Figure 1 and station details are listed in Table 1.

Table 1: Climate Station Details

Station Name	Latitude Longitude	Distance from Mine (km)	Elevation (m AD)	Record	Data Type	Portion of Record Complete
Nanortalik Heliport	60.13°N -45.23°E	35 km (SE)	5 mAD	01/01/1980 02/11/1985 (i.e. < 5 years)	Daily Precipitation	92.5%
				01/01/2014 10/07/2020 (i.e. < 6 years)	Hourly Average Air Temperature	89.7%
Narsarsuaq	61.13°N -45.41°E	91 km (NNE)	34 mAD	01/01/1973 31/12/2003 (i.e. 30 years)	Daily Precipitation	98.8%
					Daily Average Temperature	99.5%

As less than 5 years of daily precipitation data was available for the Nanortalik Station, this record was dismissed in favour of Narsarsuaq, which also has a longer and more complete dataset (1973 to 2003). For consistency, the same record was used for temperature.

A statistical analysis was carried out in order to interrogate seasonal climate trends between the adopted Narsarsuaq gauge, the Nanortalik gauge, and the available short-term records captured on site. The primary purpose of this analysis was to identify any potentially significant precipitation and/or temperature variability between the adopted climate record, and the site. In summary it was determined that the Narsarsuaq gauge is representative of site conditions, and no regional adjustments were warranted.

3.3 Precipitation

3.3.1 Regional Recorded Precipitation Statistics

Total precipitation depths (i.e. including both rainfall and snowmelt) were available for the Narsarsuaq Station as outlined in Table 1 above, and these were used as the basis for the analysis of potential flood risk to the proposed mine infrastructure, as described in “Nalunaq Gold Mine - Flood Risk Assessment Report” (Golder, 2021a).

In order to estimate rainfall and snowfall values, potential snowfall depths were derived using the degree-day method (Maidment, 1993). A base daily average air temperature of 0 °C was assumed between April and October (period of major melt), while a base daily average air temperature of 2.5 °C was assumed between November and March. Any daily recorded precipitation which occurred on days with recorded daily air temperatures that exceeded the base temperature was assumed to report to the Site as rainfall. The assignment of these base temperatures reflects lower air temperatures required to trigger snowmelt between April and

October, as opposed to other times of the year. This is due in part to energy available from the sun, as well as other factors, such as warmer rainfall and higher ground temperatures. A melt factor of 0.9 mm per °C per day was also applied, which accounts for the accelerating effect of rainfall on the melting of the snowpack (and hence rate of snowmelt).

Annual total precipitation averaged 601.8 mm, of which it was determined approximately 68% was estimated to occur as rainfall and the remainder as snowfall. The wettest month was September with an average monthly total precipitation depth of 73.8 mm, and the driest month was March with an average monthly total precipitation depth of 35.6 mm. Measurable snowfall occurred from October to April, with rainfall occurring predominantly in the summer months.

Precipitation, rainfall, and snowfall depths for Narsarsuaq are provided in Table 2 and Figure 3.

Table 2: Average Monthly Precipitation at Narsarsuaq Station (1973 – 2003)

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Precipitation (mm)	44.0	37.7	35.6	45.6	35.8	57.4	58.2	64.6	73.8	57.6	47.6	43.9	601.8
Rainfall (mm)	3.2	7.5	2.4	33.5	35.0	57.4	58.2	64.6	73.1	50.4	16.2	6.4	407.8
Snowfall (mm) (1)	40.7	30.3	33.3	12.2	0.8	0.0	0.0	0.0	0.6	7.2	31.4	37.5	194.0

NOTES: (1) As water equivalent.

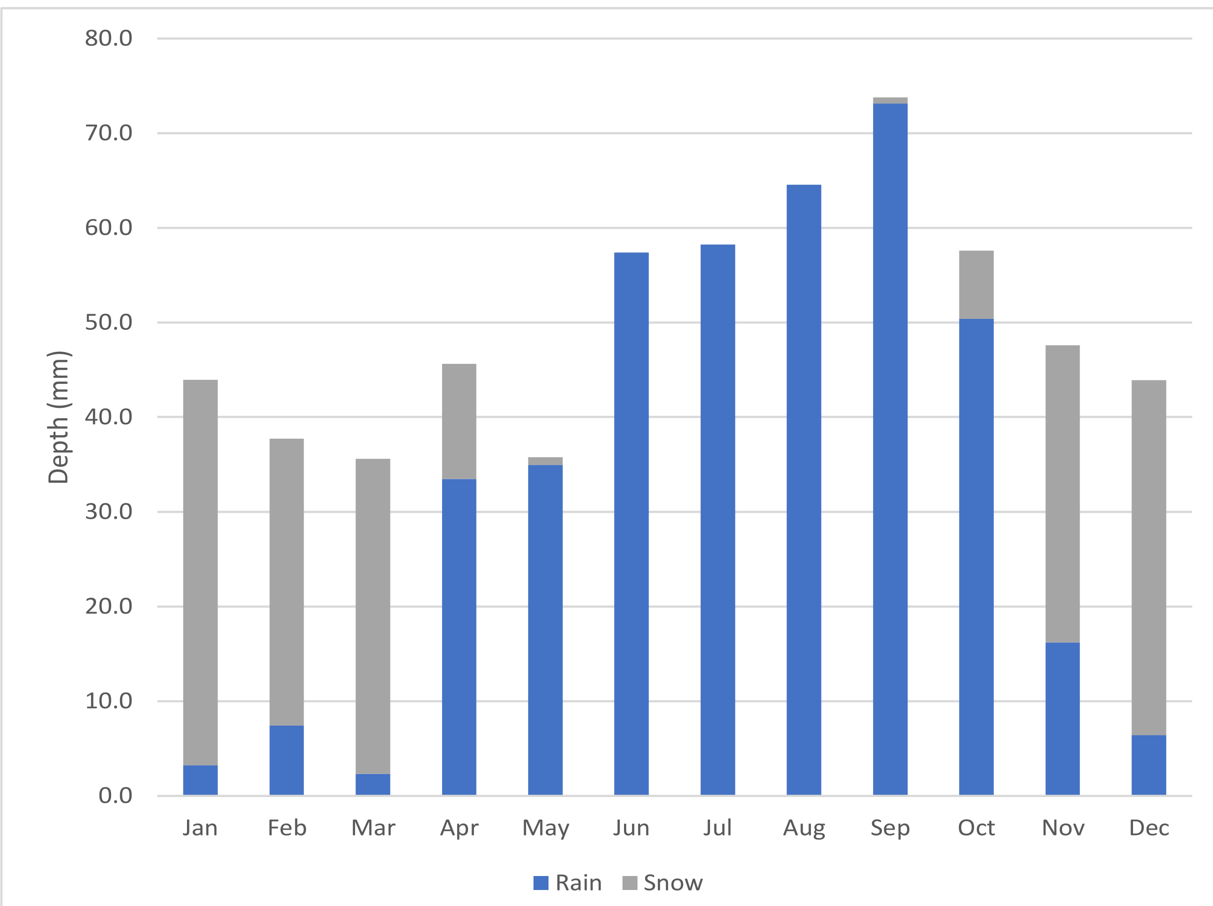


Figure 3: Average Monthly Rainfall and Snowfall at Narsarsuaq Station (1973 – 2003)

3.3.2 Increased Precipitation (Anecdotal Information)

As outlined in Section 3.1 above, anecdotal information has been provided by GINR that indicates that the annual precipitation on site may be as high as 900 mm, or a 50% increase in the recorded precipitation per year at Narsarsuaq. An analysis has therefore been carried out to establish the potential impacts upon the surface water infrastructure design regime of a 50% increase in design rainfall. This is explained in Section 4.2 below.

3.4 Temperature

Average temperature data recorded at the Narsarsuaq Station between 1973 and 2003 are presented in Table 3, including the mean (average) minimum, mean maximum and mean daily temperatures for the 30-year period of record. The mean annual temperature during this period was 0.9 °C. Temperatures were highest from April to October, and lowest from November to March (mean temperatures did not exceed 0 °C). July was the hottest month with a mean maximum temperature of 20.3 °C . February was the coldest month, with a mean minimum temperature of -24.0 °C. The highest temperature recorded in the 30-year record was 25 °C (02/04/1998) and the lowest was -39.8 °C (23/01/1984).

Table 3: Average Temperature at Narsarsuaq Station (1973 – 2003)

Parameter	Temperature (°C)												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Mean (Average) Maximum Daily Temperature	8.4	7.0	8.6	12.3	16.2	19.2	20.3	19.2	16.6	12.8	11.4	9.1	13.4
Mean (Average) Daily Temperature	-7.5	-7.8	-6.0	0.3	5.4	8.9	10.7	9.4	5.8	0.8	-3.5	-6.0	0.9
Mean (Average) Minimum Daily Temperature	-23.3	-24.0	-21.1	-13.1	-4.3	1.3	3.4	2.3	-3.1	-9.8	-17.9	-20.7	-10.9

3.5 Evaporation

Potential evapotranspiration (PET) at the Narsarsuaq Station between 1973 and 2003 was calculated from the temperature dataset using the Thornthwaite method (Thornthwaite, 1948). Average monthly and annual PET depths are presented in Table 4. Average annual evapotranspiration over the 30-year period of record was 465.2 mm. Evapotranspiration rates were highest from June to August (over 95 mm of evaporation occurred in each month). Potential evapotranspiration rates were lowest from November to March, with little to no evaporation in these months.

Table 4: Average Potential Evapotranspiration at Narsarsuaq Station (1973 – 2003)

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Potential Evapotranspiration (PET) (mm)	0.1	0.5	0.0	14.9	64.4	100.6	118.0	96.3	56.3	12.2	1.5	0.4	465.2

4.0 HYDROLOGICAL ASSESSMENT

4.1 Hydrological Setting

The Nalunaq Mine is located in the fjords of southern Greenland. The area is mountainous and is characterised by steep topography with slopes reaching from sea level to elevations of approximately 1500 meters above sea level (masl). The mine sits on the northern slopes of Kirkespirdalen U-shaped glacial valley. The valley surface is predominantly covered in grass and scree; however, vegetation becomes more limited at higher elevations.

The Kirkespir River flows approximately 15 km along the length of the valley, originating at a small glacial lake at the head of the valley and discharging into the Sarqå fjord at its base. The stream has no major tributaries and has an estimated catchment area of 95 km^2 (Kvaerner E&C, 2002). Flow measurements from the river are limited, though measurements taken between 25/05/1998 and 31/08/1998 give an indication of typical base flow in the river, with an average[1] flow rate of 3.95 m^3/s being recorded immediately downstream of the Site over the 3-month monitoring period (SRK Consulting, 2002).

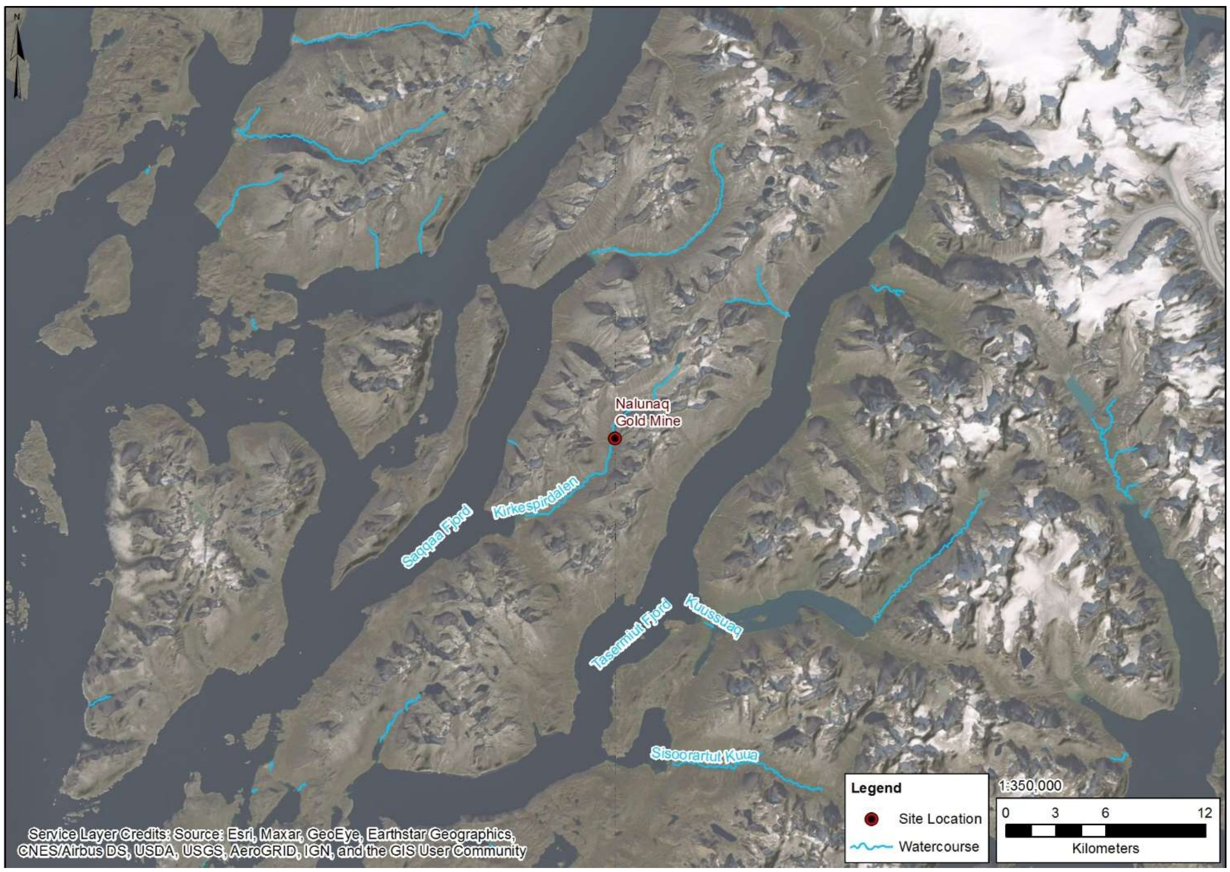


Figure 4: Location of Mine and surrounding fjords

[1] The maximum recorded stream flow rate was in late May 1998 (i.e. 4.4 m^3/s) and the minimum recorded stream flow rate was in late August 1998 (3.6 m^3/s). There was no rainfall recorded during the 3 month monitoring window, with the last recorded rainfall observed on 25th April 1998

4.2 Precipitation Analysis

4.2.1 Snowmelt

The annual spring melt plays a key part in the local hydrology and the 30 year total precipitation and temperature records for the Narsarsuaq Station (1973 - 2003) were used to derive snowmelt data. As mentioned in Section 3.3, snowmelt data was derived using the degree-day method (Maidment, 1993) with a melt factor, which accounts for the accelerating effect of rainfall on the snowmelt rate. This approach allows for accumulation of a synthetic snowpack according to the daily snowfall and subsequent depletion of the snowpack, based on a potential snowmelt. A snow density of 0.1 was assumed in the calculations to convert snow depth into its water equivalent.

The calculated average monthly and annual snowmelt water equivalents from 1973 to 2003 are presented in Table 5, along with average rainfall plus snowmelt depths. Snow melt is predicted in all months barring August, however it peaks in spring (i.e. April) with a maximum average of 83.1 mm.

Table 5: Average Snowmelt and Rainfall plus Snowmelt at Narsarsuaq Station (1973 – 2003)

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Snowmelt (mm)	6.2	7.9	6.0	83.1	50.3	26.9	3.0	0.0	0.1	3.2	6.5	8.3	201.4
Rainfall plus Snowmelt (mm)	9.4	15.4	8.3	116.6	85.2	84.3	61.2	64.6	73.3	53.6	22.7	14.7	609.3

The total rainfall and snowmelt of 609.3 mm, indicated in Table 5 (above) is a *calculated* value based on the degree-day method described in Section 3.3, and therefore the annual total is slightly higher than the annual average recorded precipitation of 601.8 mm (Table 2).

As outlined in Section 3 above, GINR recommends that a more conservative range of annual precipitation be used, i.e. instead of the 602 mm/year, 900 mm/year should be used.

In the absence of an available rainfall records to support the anecdotal information provided, the analysis has (through necessity) taken a relatively conservative approach, i.e. assuming a 50% increase in rainfall across the full recorded rainfall distribution. The analyses below reflect the resulting increase in precipitation depth and design flow(s).

4.2.2 Rainfall plus Snowmelt Depths

Annual maximum daily rainfall plus snowmelt was compiled from the 30-year record for the Narsarsuaq Station. Frequency analysis of the annual maximum timeseries was undertaken to estimate rainfall and snowmelt depths for a range of return periods. The Log Normal probability distribution was deemed to provide the most representative fit to the results. The results are presented in Table 6.

Table 6: Annual Maximum Daily Rainfall plus Snowmelt Depths at Narsarsuaq Station

Return Period (Years)	Annual Exceedance Probability (%) (1)	Rainfall plus Snowmelt Depth (mm)	50% increase in Rainfall plus Snowmelt depth, adopted for design purposes (mm)
2	50	42.4	63.6
5	20	59.0	88.5
10	10	70.1	105.2

Return Period (Years)	Annual Exceedance Probability (%) (1)	Rainfall plus Snowmelt Depth (mm)	50% increase in Rainfall plus Snowmelt depth, adopted for design purposes (mm)
25	4	84.3	126.4
50	2	95.9	142.4
100	1	106.6	158.4
200	0.5	116.5	174.7
500	0.2	131.1	196.7
1,000	0.1	142.5	213.7

NOTES: (1) The Annual Exceedance Probability (AEP) refers to the probability of a flood event occurring in any given year.

4.3 Probable Maximum Precipitation

Probable Maximum Precipitation (PMP) is defined as "*theoretically the greatest precipitation for a given duration that is physically possible over a given watershed area, or size of storm area at a particular location at a certain time of year, under modern meteorological conditions"* (WMO, 2009).

To account for the possible influence of rain-on-snow events at the Site, two scenarios for assessing the PMP were considered, i.e.:

- Scenario 1 - The PMP depth considers daily rainfall data over the entire year; and
- Scenario 2 - The PMP depth considers rainfall limited to the snowmelt season (April to July), to which the 1-in-100 year (1% AEP) snowmelt depth was added to obtain a combined PMP.

The PMP depths were calculated using the statistical procedure described by the WMO (2009), and the results are summarised in Table 7.

Table 7: Probable Maximum Precipitation Results (mm)

Assessment Basis	PMP Depth (mm)	50% increase in PMP Depth, adopted for design purposes (mm)
Assessment based on daily rainfall over the entire year (Scenario 1)	439.3	658.9
Assessment based on spring rainfall plus 1-in-100 year snowmelt (Scenario 2)	428.6	642.9

As shown in Table 7, rainfall events which occur outside of the snowmelt season are the driving factor behind the large storm events. These rainfall events typically occur in July and August. On this basis, the PMP depth represented by the annual rainfall record (Scenario 1) of 658.9 mm (i.e. assuming a 50% increase as per GINR anecdotal information) was selected for use in the flood risk assessment.

4.4 Land Cover Assessment

In order to calculate the rate and volume of runoff from a catchment area (Table 9), it is necessary to interrogate the typical land cover that characterises the contributing watershed. This is then used to define the "SCS Curve Number" that is used by internationally recognised hydrological models. The "SCS Curve Number" is an empirical parameter that was developed by the USDA Natural Resources Conservation Service (formally the Soil Conservation Service (SCS)). A higher curve number represents a lower rate of infiltration into the underlying ground material, as is dependent on several factors, including land cover, soil type and land slope.

To assist in the assessment of land type cover, aerial photography in the form of orthophotos was used in combination with satellite imagery. The orthophotos extend 9 km upstream of the Sarqå Fjord, covering an area of approximately 19 km^2.

While these photos cover the Site and its immediate surrounds, this detailed imagery does not extend higher than 600 masl. As such, publicly available aerial imagery was used to supplement the orthophoto imagery.

To assist in the determination of catchment boundaries, 0.6 m resolution Light Detection and Ranging (LiDAR) data (covering the same extent as the orthophotos) was used in combination with publicly available topographic data.

Based on a visual assessment of the landcover, Kirkespirdalen is predominantly composed of scree and grass. The curve numbers used to represent each land type are presented below in Table 8, and were selected based on guidance provided in Chow *et al.* (1988).

Table 8: Land Type Properties

Land Type	Curve Number	Basis (1)
Grass	74	Grass, good condition, >75% cover
Scree	89	Gravel, soil group C

NOTES: (1) Based on land type description provided in Chow et al. *(1988).*

5.0 SURFACE WATER INFRASTRUCTURE DESIGN

5.1 Strategy

Golder has developed a preliminary surface water management strategy for the proposed Site aimed at handling "contact" water and "non-contact" water. The strategy to minimise "contact" water consists of the following main principles:

- Intercepting flows from areas upgradient of the DTSF and Processing Plant, and conveying them away from the facilities to minimise flood risk;
- Reducing offsite sediment transport to within acceptable limits by settling (and thereby removing) sediment from surface water runoff from the DTSF;
- Placing any structures for water retention away from frequently flooded areas as much as possible; and
- Determining flood armouring (i.e. erosion protection) requirements for the DTSF.

Surface runoff generated from catchment areas upgradient of the proposed DTSF and Process Plant facilities will be intercepted by a series of diversion channels and drains, and then conveyed to the Kirkespir River. This water is considered "non-contact" water as it will not have interacted with any operational areas of the mine site.

“Contact” water from the top surface of the DTSF stack will be discharged to a proposed settling basin (“Sediment Pond”). Water in the Sediment Pond will then be allowed to discharge through a weir to a receiving channel, which in turn discharges to the Kirkespir River.

Any runoff from the DTSF slopes or platform is considered “non-contact” water (discussed further in Section 5.3.1), and will bypass the Sediment Pond, discharging via a constructed channel to the Kirksepir River.

The design considerations for the surface water infrastructure are discussed in the following sections.

5.2 Surface Water Diversion System

5.2.1 Diversion Strategy

The proposed drainage plan for the Site involves the diversion of “non-contact” water around the DTSF as shown in Drawing 1, **Appendix A**. Surface water from the hillside to the west of the DTSF and Process Plant will be intercepted by series of diversion channels and drains. This water will then be allowed to discharge to the Kirkespir River.

As mentioned previously, “contact” water generated from the DTSF will report to the Sediment Pond.

Design considerations for the DTSF diversion systems are presented in Section 5.2.2, and design considerations for the Process Pant diversions systems are presented in Section 5.2.3.

5.2.2 DTSF Diversion

5.2.2.1 Diversion Channels

The DTSF will be constructed at the base of the Kirkespirdalen, at the foot of the western valley slopes. As mentioned in Section 2.1, the Kirkespir River flows as a braided network of streams across the valley floor, with the centreline of the main river channel currently aligned approximately 20 to 50 m away from the proposed facility. The braided stream network extends into the proposed footprint of the DTSF, as such the DTSF intercepts a minor tributary of the Kirkespir River.

To mitigate the risk of pluvial flooding (or surface water flooding due to overland runoff) to the DTSF, the following diversion channels and drains are proposed for intercepting and directing water away from the facility:

- The DTSF Diversion channel will direct flows from the minor braid of the Kirkespir River (i.e. that currently flows through the proposed DTSF footprint) away from the DTSF and into the main channel of the Kirkespir River.
- Existing branches of the tributary (North Extension and South Extension) will be redirected to a main channel along the north perimeter of the DTSF (Main Diversion), as depicted in Drawing 1 and Drawing 3 (**Appendix A**). The South Extension will intercept runoff reporting to the DTSF from the western valley slopes and convey it north of the DTSF. The channels will be excavated at a minimum distance of 10 m from the toe of the DTSF cut slope.
- The Upper and Lower Haul Road Diversion channels will intercept runoff from the hillside areas west of the DTSF and convey it south-east towards the Kirkespir River (Drawing 1 and Drawing 2, **Appendix A**).
- The DTSF Drain will intercept runoff from the hillside below the Haul Road Diversion Channel and convey it south-east to the Kirkespir River (Drawing 1 and Drawing 2, **Appendix A**). This ditch will run the length of the DTSF access road (along the eastern edge of the road).

The Haul Road Diversion channels have been designed to convey the peak flow during the 1-in-25 year (4% AEP) combined rainfall, and snowmelt event, increased by 50% with a minimum freeboard of 150 mm. The freeboard ensures that the diversion channel will contain a 1-in-200 year (0.5% AEP) event before spilling onto the haul road.

The DTSF Drain has been designed to convey the peak flow during the 1-in-200 year (0.5% AEP) combined rainfall, and snowmelt event increased by 50% with a minimum freeboard of 60 mm. The freeboard ensures that the drainage channel will contain a 1-in-500 year (0.2% AEP) event before spilling. Residual flow in an extreme flooding event can be contained within a safety berm (optional) to be constructed adjacent to the channel.

The DTSF Main Diversion channel has been designed to convey:

- The 1-in-2 year (50% AEP) combined rainfall and snowmelt event increased by 50% reporting to the North Extension to cater for the “bank-full” flow, thereby mimicking the natural hydraulic characteristics of the braided river channel; and
- The 1-in-25 year (4% AEP) combined rainfall and snowmelt event increased by 50% reporting to the South Extension, with 200 mm freeboard.

The area around the channel will be graded to gently slope towards the low-flow channel help convey flows from larger events away from the DTSF. The edge of the graded area will be aligned at least 10 m away from the toe of the DTSF.

The catchment areas contributing to each of the above-mentioned channels are presented in Figure 5.

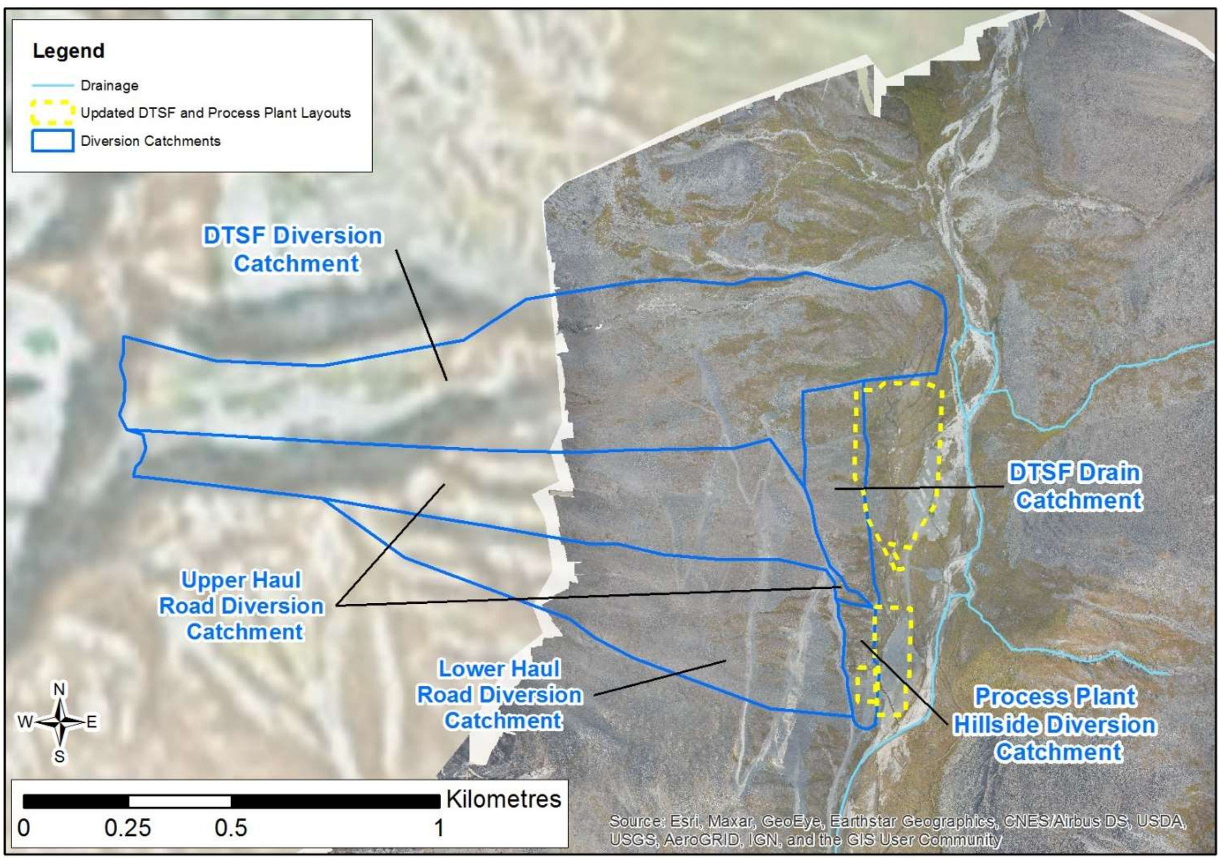


Figure 5: Catchments draining to diversion channels[2]

[2] Note that the apparent overlap with the DTSF layout is purely a graphical idiosyncrasy as the catchments were delineated based on a more refined topographic base map.

Key catchment properties considered in the channel design are provided in Table 9. Rainfall-runoff processes were simulated using the United States Army Corps of Engineers Hydrologic Engineering Centre's Hydrologic Modelling System (HEC-HMS) (USACE, 2020). For storms ranging from the 1-in-2 year (50% AEP) to the 1-in-1000 year (0.1% AEP) event precipitation data was distributed using the Frequency Storm method in HEC-HMS. The use of this method required sub-daily precipitation depth estimates, presented in **Appendix B**.

For each catchment, the lag times were assessed using the NRCS watershed Lag Method (USDAa, 2010). The NRCS method uses Curve Numbers (which represent the watershed's soil and cover conditions) to define infiltration loss, as described in Section 4.4 above. In the absence of soil hydraulic conductivity data, a Hydrologic Soil Group of C was assumed for the catchments. The curve number for each catchment was based on a weighted average of the curve number for exposed/bare rock areas (curve number 89), grass (curve number 74), and rockfill/scree deposits (curve number 72).

The catchment properties assigned to each catchment are presented in Table 9. The channels were sized using the Manning Equation for open channel flow (Manning, 1891), an approach considered appropriate for this study. The channels have been designed with uniform trapezoidal sections.

Table 9: Diversion Channels - Catchment Properties

Channel	Design Event	Catchment Area (ha)	Lag Time (mins)	Curve Number	Design Flow (m^3/s)	Design Flow (m^3/s) assuming a 50% increase in precipitation
Upper Haul Road Diversion Channel	1-in-25 year event	30.1	16.2	72	0.6	0.9
Lower Haul Road Diversion Channel	1-in-25 year event	25.0	14.2	72	0.5	0.8
DTSF Diversion Channel - **North** Extension	1-in-2 year event (North Extension)	20.9	15.5	89	0.4	0.6
DTSF Diversion Channel - **South** Extension	1-in-25 year event (South Extension)	35.1	16.1	72	0.7	1.1
DTSF Drain	1-in-200 year event	7.0	7.3	89	1.5	2.2

The geometric properties of the Haul Road Diversion, DTSF Diversion and DTSF Drain channels are presented in Table 10.

Table 10: Diversion Channels - Geometric Properties

Channel Section	Length (m)	Channel Bottom Width (m)	Depth (m)	Depth (m) assuming a 50% increase in precipitation	Side Slopes	Bottom Slope (%) (1)	Channel Lining
Upper Haul Road Diversion Channel	510	0.5	0.35	0.4	1H : 1V	15.7 % - 28.3 %	Protective layer to reduce scour (e.g. HDPE or geotextile overlain with gravel)
Lower Haul Road Diversion Channel	410	0.5	0.35	0.4	1H : 1V	1.5 % - 12.0 %	Protective layer to reduce scour (e.g. HDPE or geotextile overlain with gravel)
DTSF Diversion Channel - **North** Extension	50	3.0	0.35	0.5	4H : 1V	2.0%	Unlined (excavated in bare earth)
DTSF Diversion Channel - **South** Extension	600	1.0	0.5	0.55	1.5H : 1V	1.5 - 19.7%	Lined with rip-rap
DTSF Diversion Channel – **Main** Diversion	235	3.0	0.5	0.6	4H : 1V	0.3 %	Unlined (excavated in bare earth)
DTSF Drain	440	0.5	0.5	0.6	1H : 1V	1.6 % – 9.1 %	Protective layer to reduce scour (e.g. HDPE or geotextile overlain with gravel)

NOTE: (1) Where applicable, the range of channel bottom slopes has been presented

Due to the high velocities which will be experienced in the Haul Road Diversion and DTSF Drain channels, these channels will need to be protected from scour (i.e. through the installation of a protective layer) and will need to be maintained frequently (particularly after heavy storm events) to repair any damage to the channels.

The DTSF Diversion channels were designed (as much as practically possible) to replicate the natural hydraulic characteristics of the braided river channels, i.e. maintaining the same slope and cross sections of the upstream braids. The North Extension and Main Diversion sections will be excavated in bare earth (unlined), however, the steep section of the South Extension section (as it descends the hillside) will be protected against scour through the installation of a rip-rap lining. Typical sections for these channels are presented in Drawing 4, **Appendix A**.

The hydraulic properties of the channels are presented in Table 11.

Table 11: Diversion Channels - Hydraulic Properties Under Design Conditions

Channel	Design Event	Recorded Precipitation (Narsarsuaq)			Anecdotal 50% Increase in Precipitation		
		Peak Flow (m^3/s)	Maximum Depth (m)	Maximum Velocity (m/s)	Peak Flow (m^3/s)	Maximum Depth (m)	Maximum Velocity (m/s)
Upper Haul Road Diversion Channel	1-in-25 year event	0.6	0.2	6.1	0.9	0.2	6.9
Lower Haul Road Diversion Channel	1-in-25 year event	0.5	0.2	4.2	0.8	0.2	5.0
DTSF Diversion Channel - **North** Extension	1-in-2 year event	0.4	0.2	0.6	0.6	0.2	0.6
DTSF Diversion Channel - **South** Extension	1-in-25 year event	0.7	0.3	2.1	1.1	0.4	2.4
DTSF Diversion Channel - **Main** Diversion	Capacity determined as cumulative peak design flows from North & South Extensions	1.1	0.3	0.8	1.7	0.4	0.9
DTSF Drain	1-in-200 year event	1.5	0.4	5.1	2.2	0.5	5.6

5.2.3 Process Plant Diversion Channel

The Process Plant Hillside Diversion will intercept runoff from the hillside areas to the west of the Process Plant and convey it south-east towards the Kirkespir River. This channel will run along the western edge of the Process Plant (Drawing 1 and 5, **Appendix A**).

The channel has been designed to convey the peak flow during the 1-in-200 year design event (i.e. combined rainfall and snowmelt event) with a minimum freeboard of 0.2 m, and convey the 1-in-1000 year event without overtopping.

Key catchment properties considered in the channel design are provided in Table 12.

Table 12: Process Plant Diversion Channels - Catchment Properties

Channel	Design Event	Design Flow (m^3/s)	Design Flow (m^3/s) assuming a 50% in precipitation	Catchment Area (ha)	Lag Time (mins)	Curve Number
Process Plant Hillside Diversion	1-in-200 year	0.44	0.7	2.0	3.25	89

The geometric properties of the Process Plant Hillside Diversion are presented in Table 13; the typical section is presented in Drawing 4 (**Appendix A**). The hydraulic properties of the channel are presented Table 14. The channel will be provided with a protective layer (e.g. geofabric or HDPE overlain by gravel) and have a trapezoidal cross-section.

Table 13: Process Plant Hillside Diversion - Geometric Properties

Channel Section	Length (m)	Channel Bottom Width (m)	Depth (m)	Depth (m) assuming a 50% increase in precipitation	Side Slopes (xH:1V)	Bottom Slope (%)	Channel Lining
Process Plant Hillside Diversion	230	0.5	0.4	0.45	1.0	2.2	Protective layer to reduce scour (e.g. HDPE or geotextile overlain with gravel)

Table 14: Process Plant Hillside Diversion - Hydraulic Properties

Channel Section	Return Period	Peak Flow (m^3/s)	Peak Flow (m^3/s) assuming a 50% increase in precipitation	Peak Flow Depth (m)	Peak Flow Velocity (m/s)
Process Plant Hillside Diversion	1-in-200 year	0.44	0.7	0.3	2.4

5.3 On-site Ponds

As part of the site water management approach, a settling basin (“Sediment Pond”) was considered to remove particles mobilised from runoff from the top of the DTSF (Drawings 1 and 5, **Appendix A**). Settling of runoff generated from the slopes of the DTSF will not be required, as it is not anticipated that fine particles would be mobilised from the slopes. However, due to the constant movement of haul trucks depositing material onto the DTSF, some mobilisation of particles is anticipated on the top surface of the facility.

A holding pond (“Holding Pond”) was also considered to temporarily hold water pumped from the 235 Level Portal, which would then be then used for drilling in the underground mine.

The design considerations for these ponds are discussed in the following sections.

5.3.1 Sediment Pond

The Sediment Pond was designed to retain particle sizes with a median diameter as low as 0.076 mm (fine sand, 76 microns[3]) for the 99.9th percentile daily rainfall conditions. The pond was also designed to contain the 1-in-2 year runoff (3-hour storm duration) from the DTSF with no freeboard. Berms of 0.5 m will be constructed around the ponds to:

(i) Provide 0.5 mm freeboard for the 1-in-2 year (50% AEP) storm event; and

(ii) Protect the structure from fluvial flooding from the Kirkespir River (up to the 1-in-5 year (20% AEP) event).

The Sediment Pond was designed based on the relationship described in Metcalf and Eddy (2003) for the settling of discrete particles. At this design stage, Golder has assumed that the inflow (uniform across the width of the pond) is equal to the outflow.

Key design criteria for the Sediment Pond are presented in Table 15.

Table 15: Key Design Criteria - Sediment Pond

Design Criteria	Description
Inflow Source	Runoff from DTSF
Design Inflow	36 m^3/hr (0.010 m^3/s)
Design Inflow assuming a 50% increase in precipitation	54 m^3/hr (0.15 m^3/s)
Outflow Conditions	Gravity Controlled (V-notch weir)

The key dimensions of the Sediment Pond are presented in Table 16.

[3] Determined as the equivalent of the design P_{80} from the mill grinding circuit.

Table 16: Key Dimensions - Sediment Pond

Dimension	Description
Depth (m)	1
Base Length (m)	30
Base Width (m)	8
Base Length (m) assuming a 50% increase in precipitation	38
Base Width (m) assuming a 50% increase in precipitation	10
Side Slopes (H:1V)	2
Total Capacity (m^3)	320
Total Capacity (m^3) assuming a 50% increase in precipitation	480

The pond will be located to the south of the DTSF facility, and due to the high groundwater levels experienced at the site, it will be limited in the maximum depth of 1 m below ground level (mbgl). Water from the pond will flow through the weir to a discharge channel, from where it will flow south-east to the Kirkespir River.

5.3.2 Sediment Pond Channels

The DTSF Runoff Channel #1 will capture runoff from the top of the DTSF stack and convey it to the Sediment Pond. This drain has been designed to capture the 1-in-2 year (50% AEP) flow (combined rainfall and snowmelt) from the DTSF surface. The DTSF Runoff Channel # 2 will capture runoff (1-in-2 year flow (50% AEP) combined rainfall and snowmelt) from the slopes of the DTSF and convey it to the Kirkespir River.

Discharge from the Sediment Pond will report to the Sediment Pond Channel.

The flow from the above-mentioned channels passes through the DTSF Drain Culvert #2, before being discharged into the Kirkespir River via the Combined Discharge Channel (Drawing 1 and 5, **Appendix A**). The geometric properties of channels reporting to/ from the Sediment Pond are presented in Table 17, while the hydraulic properties are presented in Table 18.

Table 17: Sediment Pond Channels - Geometric Properties

Channel Section	Length (m)	Channel Bottom Width (m)	Depth (m)	Depth (m) assuming a 50% increase in precipitation	Side Slopes (xH:1V)	Bottom Slope (%)	Channel Lining
DTSF Runoff Channel #1	490	0.5	0.3	0.5	1H : 1V	1 %	Unlined (excavated in bare earth)
DTSF Runoff Channel #2	220	0.5	0.3	0.5	1H : 1V	1 %	Unlined (excavated in bare earth)
Sediment Pond Channel	50	0.5	0.3	0.5	1H : 1V	1 %	Unlined (excavated in bare earth)
Combined Discharge Channel	110	0.5	1.4	1.4	1H : 1V	1 %	Unlined (excavated in bare earth)

Table 18: Sediment Pond Channels - Hydraulic Properties

Channel Section	Design Return Period	Recorded Precipitation (Narsarsuaq)			Anecdotal 50% Increase in Precipitation		
		Peak Flow (m^3/s)	Peak Flow Depth (m)	Peak Flow Velocity (m/s)	Peak Flow (m^3/s) + 50% increase	Peak Flow Depth (m)	Peak Flow Velocity (m/s)
DTSF Runoff Channel #1 and #2	1-in-2 year	0.1	0.3	0.5	0.15	0.3	0.6
Sediment Pond Channel	1-in-2 year	0.1	0.3	0.5	0.15	0.3	0.6
Combined Discharge Channel	Capacity determined based upon cumulative peak design flows from the DTSF Drain, the Upper Haul Road Diversion and the DSTF runoff	2.4	1.0	1.5	3.6	1.2	1.7

5.3.3 Holding Pond

The Holding Pond was designed to temporarily retain the flows required for drilling in the underground mine (i.e. 15 m^3/hr) for 24 hours, with a 0.3 m freeboard. This requires a storage capacity of 380 m^3 (excluding freeboard).

Key design criteria for the Holding Pond are presented in Table 19 below.

Table 19: Key Design Criteria - Holding Pond

Design Criteria	Description
Inflow Source	Pumped water from the underground mine
Design Inflow	15 m^3/hr

The key dimensions of the Holding Pond are presented in Table 20 below.

Table 20: Key Dimensions - Holding Pond

Dimension	Description
Depth (m)	1
Base Length (m)	30
Base Width (m)	15
Side Slopes (H:1V)	2
Total Capacity (m^3)	540

If required, overflow from the Holding Pond can be discharged to the Kirkespir River (via gravity) through a V-notch weir.

5.4 Culvert Crossings

A series of culverts will be required to convey flows under site access roads to the Kirkespir River (Drawing 1, **Appendix A**). The culvert crossings have been designed to convey the design capacity of the approaching channels without causing overtopping of the channels.

Culvert design was carried out using the United States Federal Highway Administration's HY-8 Culvert Hydraulic Analysis (FHWA, 2020). A constant road crest elevation above the culvert inlet elevation was assumed over the crest length, perpendicular to the direction of flow. The road crest elevations and barrel lengths (i.e. based upon estimated embankment widths) were estimated from the 0.6 m resolution Light Detection and Ranging (LiDAR) data available for the Site. Culvert inlet and outlet elevations were assumed equal to the proposed channel inverts at the crossing location. Culvert analysis and design considered a 30º - 75º flare wingwall at the inlet of each culvert crossing.

Based on the results of the culvert analysis, corrugated steel circular pipes are proposed at each crossing. The required culvert dimensions are presented in Table 21.

Table 21: Culvert Crossings

Channel ID	Culvert Crossing ID	Culvert Span	Culvert Span assuming a 50% increase in precipitation	Assumed Culvert Length (m)
Haul Road Diversion	Upper Haul Road Culvert and Lower Haul Road	3 x 350 mm Diameter	3 x 350 mm Diameter	20
DTSF Drain	DTSF Drain Culvert #1 and DTSF Drain Culvert #2	3 x 900 mm Diameter	3 x 900 mm Diameter	15
Process Plant Hillside Diversion	Process Plant Culverts	3 x 600 mm Diameter	3 x 600 mm Diameter	12

5.5 DTSF Platform Protection

In support of the DTSF design, Golder carried a Flood Risk Assessment (FRA), as part of an Options Analysis (Golder, 2021a). The FRA was based on a review of available information and hydraulic modelling to assess flood extent, water levels and flow velocities.

As described in the FRA Report (Golder, 2021a) the assessment of the flood risk to the Site was carried out using a 2-D unsteady-state hydraulic model to simulate flows in the Kirkespir River. The model was developed using the U.S. Army Corps of Engineers' Hydraulic Engineering Centre River Analysis System (HEC-RAS) (USACE, 2016) Version 5.0.7.

The objectives of the hydraulic modelling were to:

- Assess the flood risk to the mine site surface facilities from the Kirkespir River; and
- Support the Options Analysis of the proposed mine site facility layouts.

The FRA analysis has considered an "Updated" footprint of the DTSF and Process Pad. This assumes a DTSF Layout accommodating a 5-year deposition schedule as set out in Figure 2 below. This layout is approximately 30 m narrower than the "Original" layout considered in January 2021, in order to reduce the width of the DTSF within the river valley, and to relocate the Ore Pad to the west of the Process Plant. The Original and Updated layouts are shown in Figure 6.

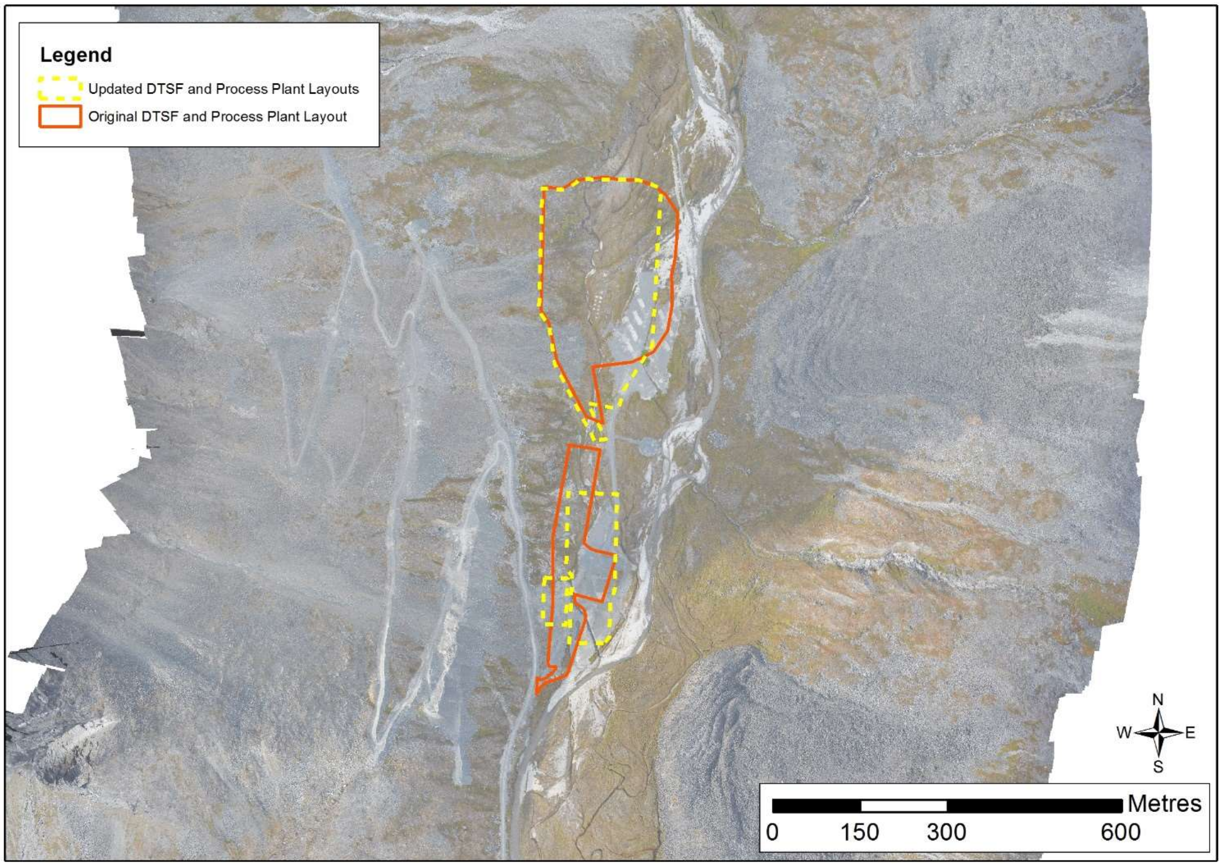


Figure 6: DTSF and Process Pad Layouts (Original and Updated)

The results of the FRA indicated that the DTSF (regardless of the layout) would be exposed to flooding from the Kirkespir River during high return-period events. As such, a consideration for the DTSF design is clearance of the facility above the Kirkespir River floodplain through the construction of a platform, as well as armouring requirements for the platform.

The flood depth and velocity results from the FRA were used to assess the required armouring for the DTSF platform, as presented in Table 22 for the Updated Layout. These results assume that any pre-existing areas with compacted material within the existing site (i.e. the previous camp platforms) will be removed and the underlying ground conditions returned to mimic the natural riverbed (i.e. loose gravel).

Table 22: Key Model Outputs – Updated Layout - DTSF

Design Event	Recorded Precipitation (Narsarsuaq)			Anecdotal 50% Increase in Precipitation		
	Design Flow Rate (m^3/s)	Predicted Depth (m)	Maximum Velocity (m/s)	Design Flow Rate (m^3/s)	Predicted Depth (m)	Maximum Velocity (m/s)
1-in-100 year	115	1.2	2.1	212	1.3	2.4
1-in-200 year	134	1.3	2.2	244	1.4	2.6
1-in-1000 year	182	1.5	2.4	320	1.6	3.0
PMF	621	2.7	3.8	953	2.8	4.6

The water surface elevations upstream and downstream of the DTSF for each design event are presented in Table 23 for the Updated DTSF Layout.

Table 23: Water Surface Elevations –Updated DTSF Layouts

Design Event	Water Surface Elevation upstream of DTSF (m AD)		Water Surface Elevation upstream of DTSF (m AD)	
	Recorded Precipitation (Narsarsuaq)	Anecdotal 50% Increase in Precipitation	Recorded Precipitation (Narsarsuaq)	Anecdotal 50% Increase in Precipitation
1-in-100 year	243.7	237.5	243.7	237.5
1-in-200 year	243.8	237.5	243.8	237.5
1-in-1000 year	243.9	237.9	244.0	237.7
PMF	244.9	238.6	245.1	238.7

Considerations for protection of the DTSF platform against erosion during periods of high flow included rip-rap and gabion arrangements, each of which are discussed in the following sections. Rip-rap was considered for armouring the raised DTSF Platform, and gabions were considered to prevent scour at the toe of the facility.

5.5.1 DTSF Embankment Face – Scour Protection

Rip-rap sizing requirements were assessed based on the relationship outlined by Isbash (1936) for the design of rip-rap at bridge abutments, as presented in Lagasse *et al* (2001). This relationship, as well as the input parameters used to assess the DTSF requirements, are presented in **Appendix C**.

The required d_{50} for the rip-rap is presented in Table 24 for the Updated layout. No factor of safety has been included in this assessment, as the empirical approach used to estimate the rip-rap requirements is already deemed to be conservative.

Table 24: Rip-Rap requirements at DTSF Platform

Return Period	Required Median Rock Size (d_{50}) (m)	
	Recorded Precipitation (Narsarsuaq)	**Anecdotal 50% Increase in Precipitation**
1-in-100 year	0.4	0.4
1-in-200 year	0.4	0.5
1-in-1000 year	0.5	0.5
PMF	0.9	1.0

The construction/ installation guidelines for the placement of the rip-rap are as follows:

- Rip-rap will be placed between the toe and the crest of the embankment, to a minimum height of 300 mm above the peak design flood level.
- Rock rip-rap thickness will be placed to d_{100} (maximum stone diameter). The rip-rap would be graded according to USDAb (1989), with a d_{100} equal to 2 times d_{50}. The required rip-rap thickness (as measured perpendicular to the embankment slope) for each return period are also presented in Table 25 for the Updated DTSF Layout.
- A geofabric (or geotextile) or granular filter material should be installed beneath the rip-rap.

Table 25: Rip-rap placement requirements – Updated DTSF Layout

Rip-rap Placement Requirement	Return Period			
	1-in-100 year	**1-in-200 year**	**1-in-1000 year**	**PMF**
Rip-rap Thickness, m	0.8	0.8	1	1.8
Rip-rap Thickness, (m) assuming a 50% increase in precipitation	0.8	1	1	2

It is recommended that semi-annual inspections be carried out during the operational phase of the mine to ensure that the rip-rap remains in good working order.

5.5.2 DTSF Embankment Toe - Scour Protection

Potential scour depths are the depth to which erosion may occur within the riverbed (i.e. at the toe of the DTSF platform) and thereby possibly undermine the structural integrity of the facility. Estimated scour depths at the toe of the DTSF were therefore assessed using the relationship outlined by Neill (1973). This relationship, as well as the inputs required to assess scour, are presented in **Appendix C**. The potential scour depths for and the Updated layout are presented in Table 26, calculated assuming a Factor of Safety of 1.5.

Table 26: Potential Scour Depths

Return Period	Potential Scour Depth (m)	
	Recorded Precipitation (Narsarsuaq)	Anecdotal 50% Increase in Precipitation
1-in-100 year	2.5	4.2
1-in-200 year	2.8	4.7
1-in-1000 year	3.5	5.7
PMF	9.6	13.8

Gabion sizing requirements were assessed based on the relationship outlined by Freeman and Fischenich (2000) for the design of gabions for streambank erosion control. This relationship, as well as the inputs used to assess the gabion sizing requirements, are presented in **Appendix C**.

A number of potential options were considered in order to protect the toe of the DTSF against scour, as outlined below.

Option 1 – Buried Gabion Baskets

An analysis of the potential to install gabion baskets along the toe of the DTSF platform has been carried out.

The input parameters used for the assessment of the gabion d_{50} sizes are presented in Table 27 for the Updated layout.

Table 27: Required Gabion Rock Sizes - Updated Layout

Return Period	Required Median Rock Size (d_{50}) (m)	
	Recorded Precipitation (Narsarsuaq)	Anecdotal 50% Increase in Precipitation
1-in-100 year	0.07	0.09
1-in-200 year	0.07	0.11
1-in-1000 year	0.15	0.15
PMF	0.32	0.32

The gabion placement and installation requirements are summarized in Table 28 for Updated DTSF layout. The analyses have assumed a Factor of Safety of 1.5, in accordance with good international practice for facilities of this nature.

Table 28: Gabion placement requirements – Updated DTSF Layout

Gabion Placement Requirement	Return Period			
	1-in-100 year	1-in-200 year	1-in-1000 year	PMF
Gabion Placement Depth (below DTSF toe), m (1)	2.5	3	3.5	10
Gabion Placement Depth (below DTSF toe), m (1) assuming a 50% increase in precipitation	3	4	4.5	11

NOTES: (1) All measurements presented in multiples of 0.5 m (thickness of each gabion basket).

The gabion baskets would be encased in wire mesh, which each basket having a thickness of 1m.

The construction/ installation guidelines recommended for the placement of gabion baskets include:

- Baskets to be manufactured using high-tensile wire mesh.
- Gabion baskets will be placed in an arrangement of 3.5 m by 3.5 m around the base of the entire facility.
- Gabion baskets will be placed to the depth of expected scour. Alternatively, other scour countermeasures can be considered, such as the construction of a rip-rap apron around the toe of the DTSF.
- A woven needle punched geotextile permeable filter membrane will be installed to the rear of the gabions. A gravel filter material should also be placed below the gabions to prevent erosion of the underlying soil material. It is recommended that semi-annual inspections be carried out during the operational phase of the mine to ensure that and the gabion installations remain in good working order.

This option has been dismissed on constructability grounds, i.e. the installation of the gabion baskets would require an open excavation to a depth of 3.5m, which would pose a significant logistical and safety challenge in light of the very high groundwater table and gravel riverbed (to depth).

Option 2 – Concrete or Sheetpile Curtain

Consideration has been taken of either the installation of a buried impermeable “curtain” surrounding the toe of the facility, including either the in-situ casting of concrete to the required depth to prevent undermining of the facility (refer Table 26), or the driving of sheetpiles to the same depth.

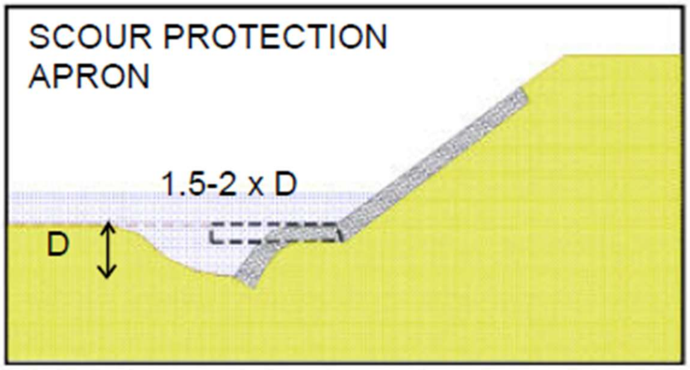
These options have been readily dismissed for logistical and constructability reasons, i.e.:

- The casting of a concrete barrier will require the installation of formwork to a 3.5m depth, which is not considered viable. The alternative would be to inject concrete until resistance is achieved, however the depth of the underlying gravels means that this is unlikely to be achieved; and
- The installation of steel sheetpiles will require the transportation of a pile driver and piles to site, which poses an immediate logistical challenge. Furthermore, the nature of the underlying ground would make precision (and therefore creating the required impermeable barrier) very difficult.

Option 3 – Scour Apron (Gabion Mattress)

In light of the constructability challenges posed by excavation into the riverbed, an “apron” arrangement has been considered. The design has assumed a gabion mattress, however alternative materials (e.g. concrete) may be suitable, if environmental concerns can be addressed in light of the impact to the natural riverbed.

The general scour apron (gabion mattress) arrangement will be in accordance with the schematic below, i.e. the apron must be constructed to a length of no less than 1.5 times the anticipated maximum scour depth (as presented in Table 26). Assuming a 1:1000 year (0.1% AEP) standard of protection is to be provided for the "updated" DTSF platform configuration therefore, it will be necessary to provide a gabion mattress that will extend no less than 4.75m[4] from the toe of the embankment into the riverbed.



The configuration of the gabion mattress will mimic the gabion basket design outlined above, i.e. with a d_{50} of 0.1 m and a minimum mattress depth of 1 m. It will be imperative that the gabion mattress is constructed to industrial standards, incl. high tensile strength wire casing, and regular monitoring and maintenance of the mattress will be required.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Surface water site infrastructure design was carried out for the Nalunaq Mine, in the Municipality of Kujalleq, Greenland. The design accounted for various layouts of the proposed Dry Tailings Stack Facility (DTSF). These layouts were assessed under various climate scenarios, with design assessments undertaken for the 1-in-100 year (1% AEP), 1-in-200 year (0.5% AEP), 1-in-1000 year (0.1% AEP) and Probable Maximum Precipitation (PMP) conditions.

The following key infrastructure were considered:

- Haul Road Diversion channels to divert runoff from the hillside to the east of the DTSF from affecting the facility during storm events. These channel was designed to carry the flows generated by 1-in-25 year event (4% AEP), with 150 mm freeboard, i.e. containing flows generated from the 1-in-1000 year event (0.1% AEP) prior to spilling.
- DTSF Diversion channel to divert a braided tributary of the Kirkespir River away from the DTSF and the Process Plant, while also diverting runoff generated on the eastern valley slopes away from the DTSF. This South Extension of the channel was designed to carry the flows generated by the 1-in-25 year event (4% AEP), with 150 mm freeboard, i.e. containing flows generated from the 1-in-200 year event (0.5% AEP) prior to spilling. The North Extension of the channel was designed to carry the flows generated by the 1-in-2 year event (50% AEP) with no freeboard, i.e. mimicking the natural bank-full capacity of the river braid.
- DTSF Drain to divert runoff from areas below the Haul Road Diversion and convey it south-east to the Kirkespir River. This channel was designed to carry the flows generated by 1-in-25 year event (4% AEP),

[4] Assuming a FoS of 1.5 in the analysis of the scour depth.

with 60 mm freeboard, and contain flows generated from the 1-in-1000 year event (0.1% AEP) through the use of an optional safety berm.

- Process Plant Hillside Diversion channel to divert runoff from the hillside east of the Process Plant from flooding the facility during storm events. This channel was designed to carry the flows generated by the 1-in-200 year event (0.5% AEP), with 200 mm freeboard, and contain flows generated from the 1-in-1000 year event (0.1% AEP), with 100 mm of freeboard;
- A Sediment Pond to allow for settlement of particles from runoff from the DTSF, under average daily rainfall conditions. This pond will also have the capacity to store the 1-in-2 year flood event. A 0.5 m berm will be constructed around the pond to prevent runoff from surrounding area from entering the pond; this berm will also provide 0.5 m of freeboard to the pond.
- A Holding Pond to allow for temporary storage (24 hours) of water to satisfy drilling requirements (at 15 m^3/hr) in the underground mine.

Erosion protection for the DTSF during flood events was also considered. The use of rip-rap protection was considered for armouring of the DTSF embankment face, whereas the use of gabion mattresses was considered for scour protection along the toe of the DTSF.

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Signature Page

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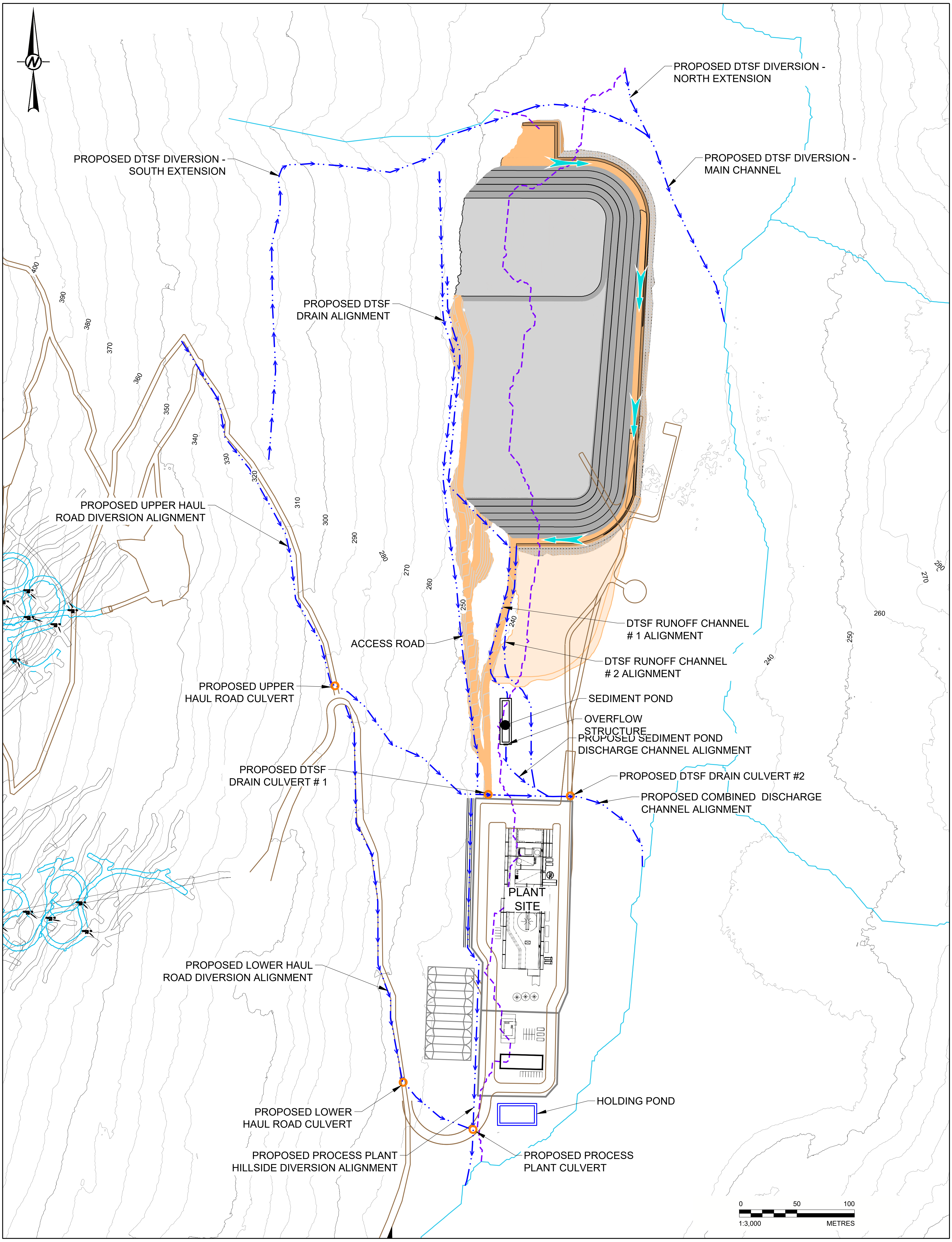
APPENDIX A

Design Drawings



LEGEND

- PROPOSED CHANNEL ALIGNMENT
- STREAM TO BE ABANDONED
- CULVERT LOCATION
- EXISTING RIVERS

CLIENT
NALUNAQ A/S

CONSULTANT
GOLDER

YYYY-MM-DD	2021-01-11
DESIGNED	ECS
PREPARED	JR
REVIEWED	JR
APPROVED	KD

PROJECT
NALUNAQ
SURFACE WATER INFRASTRUCTURE DESIGN

TITLE
SITE LAYOUT

PROJECT NO.	CONTROL	REV.	DRAWING
20136781	3400-DC-0001	A.0	1

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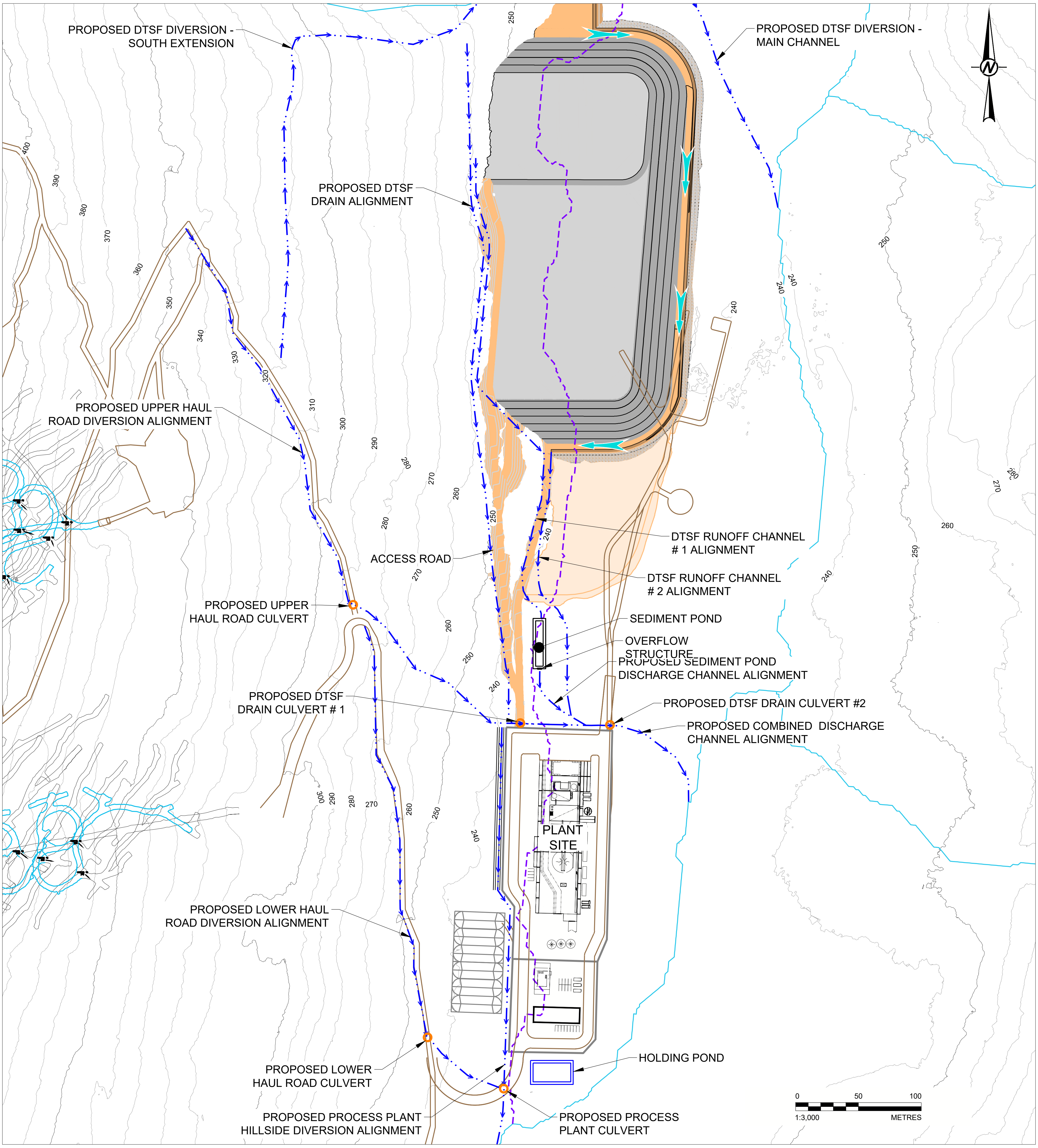
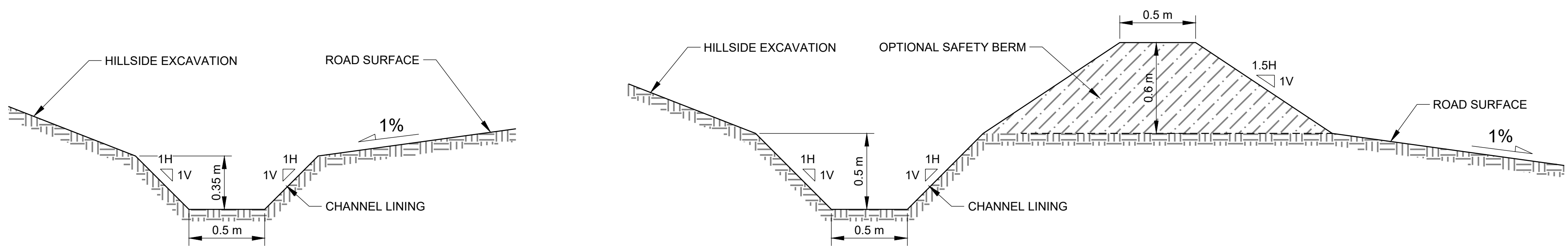




UPPER/LOWER HAUL ROAD DIVERSION CHANNEL - TYPICAL SECTION
SCALE 1:40

DTSF DRAIN - TYPICAL SECTION
SCALE 1:40

LEGEND

- PROPOSED CHANNEL ALIGNMENT
- STREAM TO BE ABANDONED
- CULVERT LOCATION
- EXISTING RIVERS
- DIRECTION OF DTSF RUNOFF FLOW

CLIENT
NALUNAQ A/S

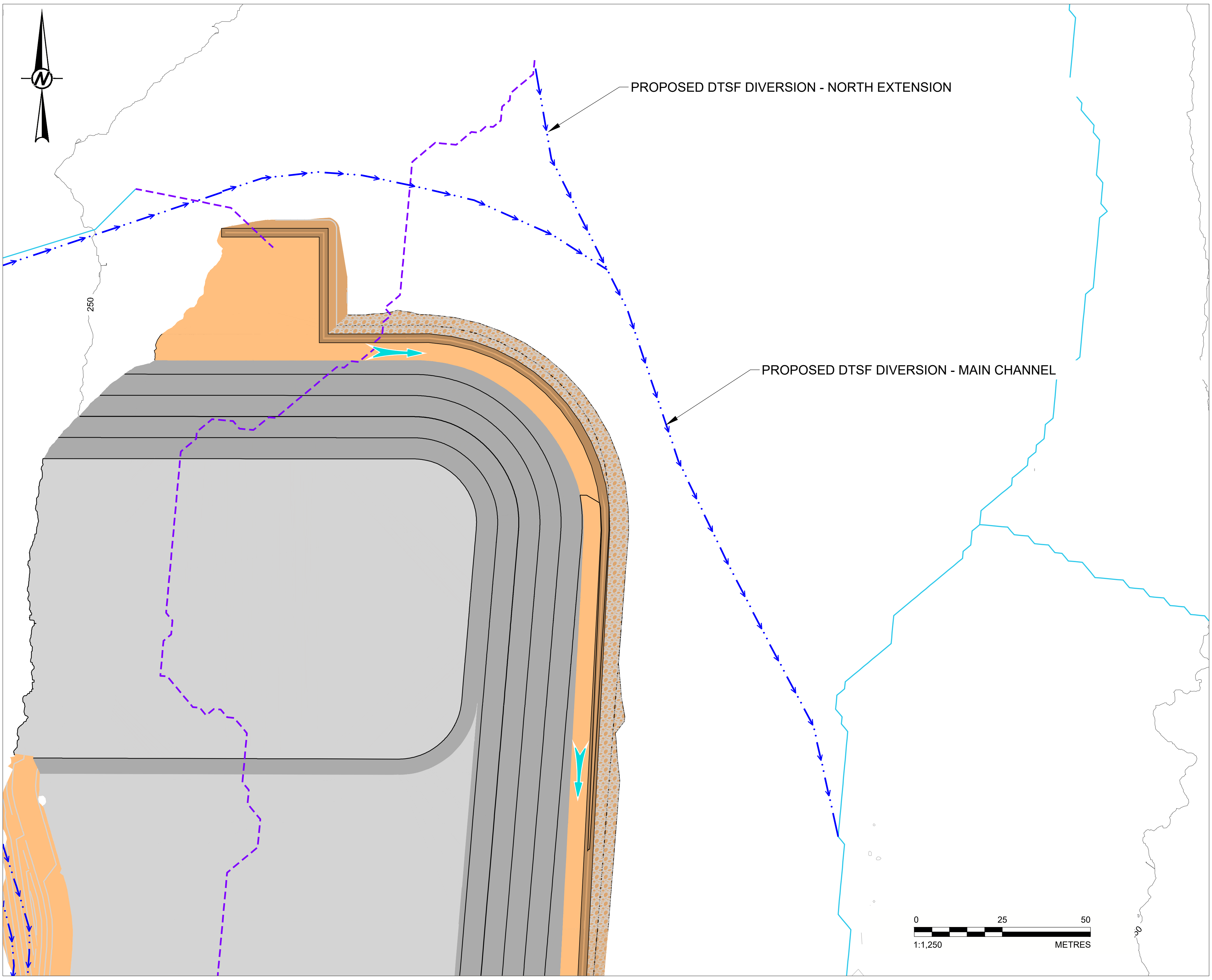
PROJECT
NALUNAQ
SURFACE WATER INFRASTRUCTURE DESIGN

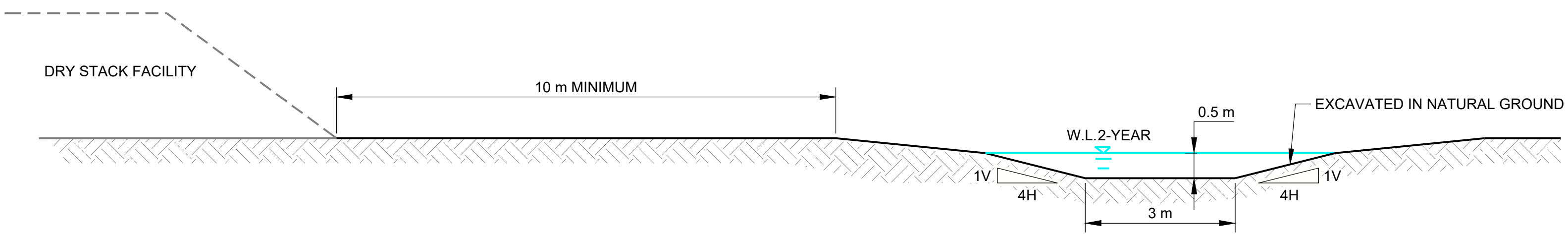
CONSULTANT
GOLDER

YYYY-MM-DD	2021-01-11
DESIGNED	ECS
PREPARED	JR
REVIEWED	JR
APPROVED	KD

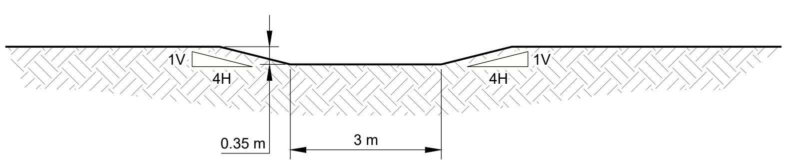
TITLE
HAUL ROAD DIVERSION AND DTSF DRAIN LAYOUT

PROJECT NO.	CONTROL	REV.	DRAWING
20136781	3400-DC-0001	A.0	2

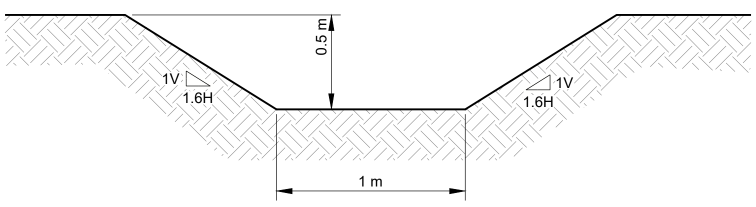




MAIN DTSF DIVERSION CHANNEL
SCALE 1:150



NORTH EXTENSION
SCALE 1:150



SOUTH EXTENSION
SCALE 1:40

LEGEND

- PROPOSED CHANNEL ALIGNMENT
- STREAM TO BE ABANDONED
- EXISTING RIVERS
- DIRECTION OF DTSF RUNOFF FLOW

CLIENT	NALUNAQ A/S
PROJECT	NALUNAQ SURFACE WATER INFRASTRUCTURE DESIGN
CONSULTANT	GOLDER
YYYY-MM-DD	2021-01-11
DESIGNED	ECS
PREPARED	JR
REVIEWED	JR
APPROVED	KD
TITLE	**DTSF NORTH DIVERSION LAYOUT**
PROJECT NO.	20136781
CONTROL	3400-DC-0001
REV.	A.0
DRAWING	3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3

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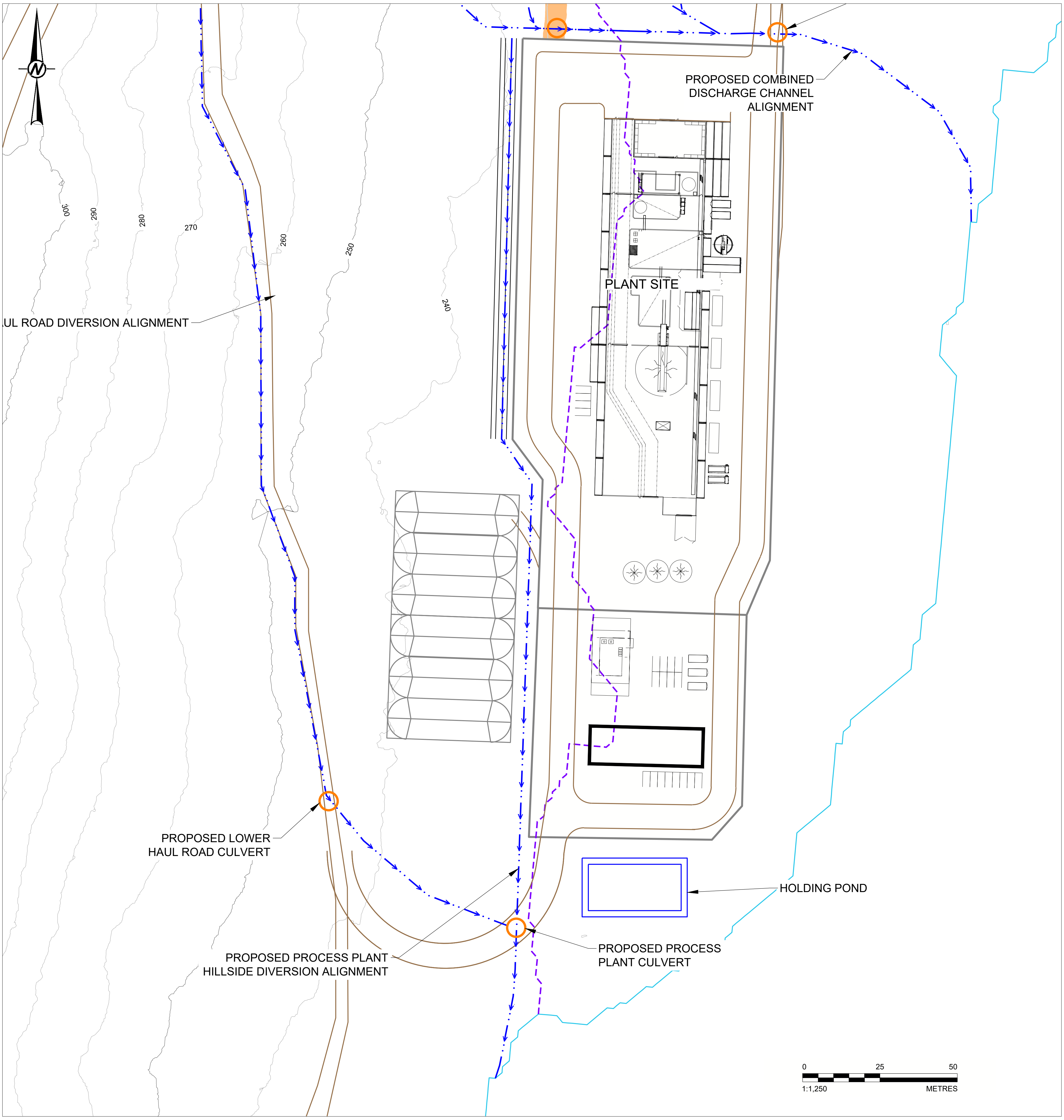

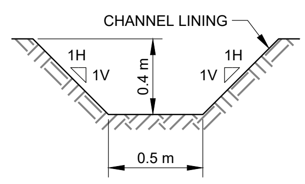

PROCESS PLANT DIVERSION - TYPICAL SECTION

SCALE 1:40

LEGEND

- PROPOSED CHANNEL ALIGNMENT
- STREAM TO BE ABANDONED
- CULVERT LOCATION
- EXISTING RIVERS

CLIENT	NALUNAQ A/S
PROJECT	NALUNAQ SURFACE WATER INFRASTRUCTURE DESIGN
CONSULTANT	GOLDER
YYYY-MM-DD	2021-01-11
DESIGNED	ECS
PREPARED	JR
REVIEWED	JR
APPROVED	KD
TITLE	**PROCESS PLANT DIVERSION LAYOUT**
PROJECT NO.	20136781
CONTROL	3400-DC-0001
REV.	A.0
DRAWING	4

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3

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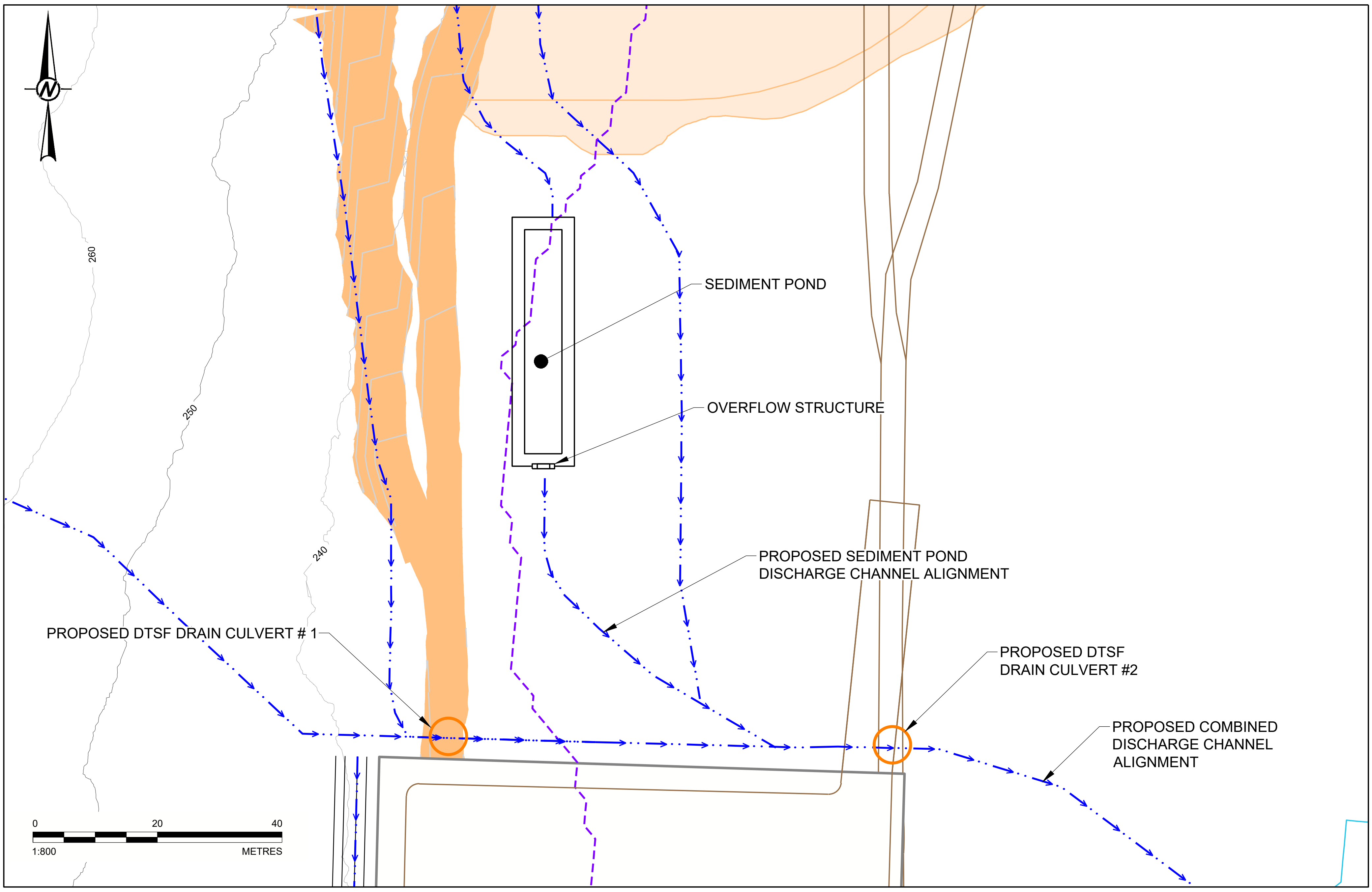
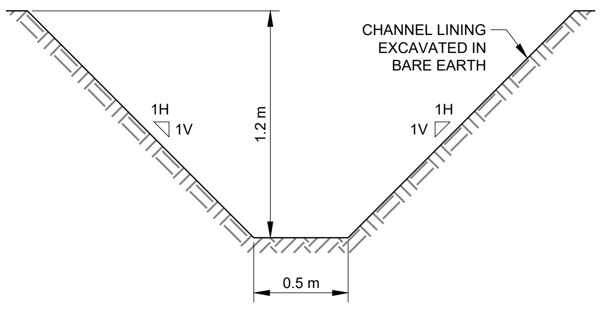
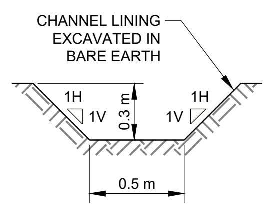
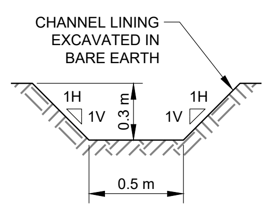




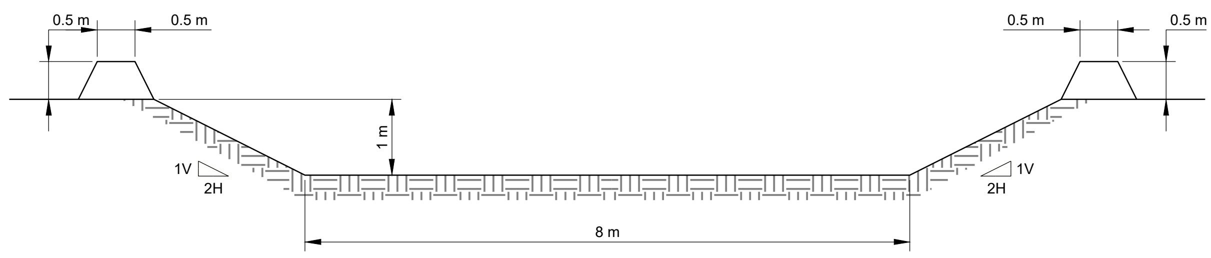
COMBINED DISCHARGE CHANNEL - TYPICAL SECTION
SCALE 1:40



DTSF RUNOFF CHANNEL # 1 AND 2
SCALE 1:40



SEDIMENT POND CHANNEL - TYPICAL SECTION
SCALE 1:40



SEDIMENT POND - TYPICAL SECTION
SCALE 1:100

LEGEND

- PROPOSED CHANNEL ALIGNMENT
- STREAM TO BE ABANDONED
- CULVERT LOCATION
- EXISTING RIVERS

CLIENT
NALUNAQ A/S

CONSULTANT
GOLDER

YYYY-MM-DD	2021-01-11
DESIGNED	ECS
PREPARED	JR
REVIEWED	JR
APPROVED	KD

PROJECT
NALUNAQ
SURFACE WATER INFRASTRUCTURE DESIGN

TITLE
SEDIMENT POND LAYOUT

PROJECT NO.	CONTROL	REV.	DRAWING
20136781	3400-DC-0001	A.0	5

APPENDIX B

Derived Sub-daily Precipitation Depths

Due to the lack of sub-daily rainfall data, the calculated Annual Maximum Daily Rainfall depths required downscaling to allow assessment of sub-daily rainfall events. Several methods were considered, including the Bell (1969), Wild (1982) and Herschfield (1961) methods, each described in Adamson and Chong (1992).

The Wild method was discounted as it was developed using data collected at one rain gauge in a tropical environment. The Bell method was developed considering extreme storm rainfall patterns from various parts of the world but requires a locally specific empirical coefficient. As no coefficient was available for Greenland or any other Arctic region, this method was also found to be unsuitable.

The Herschfield method was therefore found to be the most applicable. The method was developed from storm rainfall in the US but, as it reflects a wide range of climates from tropical to arid, it has found application in many other parts of the world. Due to the region's limited rainfall, the Site may be classified as 'semi-arid'. This assumption allowed Golder to select appropriate downscaling ratios to calculate sub-daily rainfall. The selected ratios are presented in Table A1.

Table A1: Downscaling Ratios for 1 to 24 hours, Arid/Semi-arid Zones (from Adamson and Chong, 1992).

Storm Duration (Hours)	Downscaling Factor
1	0.40
2	0.50
3	0.62
6	0.80
12	0.95
24	1.00

For sub-hourly rainfall values, the derived hourly rainfall was multiplied by the downscaling ratios recommended by Hershfield (1961) which are presented in Table A2.

Table A2: Downscaling Ratios for 5 to 60 minutes (from Adamson and Chong, 1992).

Storm Duration (Minutes)	Downscaling Factor
5	0.29
10	0.45
15	0.57
30	0.79
60	1.00

Table A3 and Table A4 present the Depth-Duration-Frequency (DDF) and Intensity-Duration-Frequency (IDF) data for the rainfall plus snowmelt data generated through the use of the above-mentioned methods, respectively.

Table A3: Rainfall plus Snowmelt Depth-Duration-Frequency Table (mm)

Return Period (Years)	Storm Duration (Hours)									
	0.08	0.17	0.25	0.50	1	2	3	6	12	24
2	4.9	7.6	9.7	13.4	17.0	21.2	26.3	33.9	40.3	42.4
5	6.8	10.6	13.4	18.6	23.6	29.5	36.6	47.2	56.0	59.0
10	8.1	12.6	16.0	22.2	28.0	35.1	43.5	56.1	66.6	70.1
25	9.8	15.2	19.2	26.6	33.7	42.1	52.2	67.4	80.1	84.3
50	11.0	17.1	21.6	30.0	38.0	47.5	58.8	75.9	90.2	94.9
100	12.3	19.0	24.1	33.4	42.2	52.8	65.5	84.5	100.3	105.6
200	13.5	21.0	26.6	36.8	46.6	58.2	72.2	93.2	110.6	116.5
500	15.2	23.6	29.9	41.4	52.4	65.6	81.3	104.9	124.6	131.1
1,000	16.5	25.6	32.5	45.0	57.0	71.2	88.3	114.0	135.3	142.5
10,000	21.1	32.8	41.6	57.6	72.9	91.1	113.0	145.8	173.2	182.3

Table A4: Rainfall plus Snowmelt Intensity-Duration-Frequency Table (mm/hr)

Return Period (Years)	Storm Duration (Hours)									
	0.08	0.17	0.25	0.50	1	2	3	6	12	24
2	59.0	45.8	38.7	26.8	17.0	10.6	8.8	5.7	3.4	1.8
5	82.1	63.7	53.8	37.3	23.6	14.7	12.2	7.9	4.7	2.5
10	97.6	75.7	63.9	44.3	28.0	17.5	14.5	9.3	5.5	2.9
25	117.3	91.0	76.9	53.3	33.7	21.1	17.4	11.2	6.7	3.5
50	132.1	102.5	86.5	60.0	38.0	23.7	19.6	12.7	7.5	4.0
100	147.0	114.1	96.3	66.7	42.2	26.4	21.8	14.1	8.4	4.4
200	162.1	125.8	106.2	73.6	46.6	29.1	24.1	15.5	9.2	4.9
500	182.5	141.6	119.6	82.9	52.4	32.8	27.1	17.5	10.4	5.5
1,000	198.3	153.9	129.9	90.0	57.0	35.6	29.4	19.0	11.3	5.9
10,000	253.8	196.9	166.3	115.2	72.9	45.6	37.7	24.3	14.4	7.6

APPENDIX C

Rip-rap and Gabion Design Parameters

1.0 DTSF RIP-RAP REQUIREMENTS

Rip-rap sizing requirements were assessed based on the relationship outlined by Isbash (1936) for the design of rip-rap at bridge abutments, as presented in Lagasse et al (2001). This relationship is presented in Equation 1 below:

$$\frac{d_{50}}{y} = \frac{K}{(S_s - 1)} \left[\frac{V^2}{gy}\right]^{0.14} \qquad \textit{Eqn. 1}$$

Where,

d_{50} = Median stone diameter, m

y = Depth of flow in the contracted section, m

K = 0.61 for spill-through abutments[1]

S_s = Specific gravity of rock riprap

V = Characteristic velocity in the contracted section, m/s

G = Acceleration due to gravity, m/s^2

The input parameters used for the assessment, as well as the required d_{50} for the rip-rap, are presented in Table C1 for the Original Layout, and Table C2 or the Updated Layout.

Table C1: Rip-Rap requirements at DTSF Platform – Original Layout

Design Parameter	Return Period			
	PMF	1-in-1000 year	1-in-200 year	1-in-100 year
Velocity at DTSF Platform, V [m/s]	4.5	2.5	2.4	2.3
Acceleration due to gravity, g [m/s^2]	9.81	9.81	9.81	9.81
Specific gravity of rock rip-rap, S_s	2.65	2.65	2.65	2.65
Velocity multiplier, K	1.02	1.02	1.02	1.02
Depth of flow at the contracted opening, Y [m]	3.1	1.9	1.7	1.6
Required d_{50}, m	**1.8**	**1.0**	**0.9**	**0.9**

[1] A spill-though abutment refers to an abutment that is set back from the main river body

Table C2: Rip-Rap requirements at DTSF Platform – Updated Layout

Design Parameter	Return Period			
	PMF	1-in-1000 year	1-in-200 year	1-in-100 year
Velocity at DTSF Platform, V [m/s]	3.8	2.4	2.2	2.1
Acceleration due to gravity, g [m/s^2]	9.81	9.81	9.81	9.81
Specific gravity of rock rip-rap, S_s	2.65	2.65	2.65	2.65
Velocity multiplier, K	0.61	0.61	0.61	0.61
Depth of flow at the contracted opening, Y [m]	2.7	1.5	1.3	1.2
Required d_{50}, m	**0.9**	**0.5**	**0.4**	**0.4**

Potential scour depths were assessed using the relationship outlined by Neill (1973). This relationship is presented in Equation 2 below:

$$y_n = y_{bf}\left(\frac{q_d}{q_{bf}}\right)^m \quad \textit{Eqn. 2}$$

Where:

y_n = scour depth below design flow level (m)

y_{bf} = average bank-full flow depth (m)

q_d = design flow discharge per unit width (m^2/s)

q_{bf} = bankfull flow discharge per unit width (m^2/s)

m = exponent varying from 0.67 for sand and 0.85 for coarse gravel

Based on visual inspection of the in-situ material around the DTSF, a median soil diameter of 10 mm (medium gravel) was assumed to be representative of the material.

The potential scour depths are presented in Table C3 for the Original layout, and Table C4 for the Updated layout. A factor of safety of 1.5 was applied to the scour depth calculations to account for uncertainty in the inputs, such as the particle size distribution of the native soil material.

Table C3: Potential Scour Depths – DTSF Original Layout

Design Parameter	Return Period			
	PMF	1-in-1000 year	1-in-200 year	1-in-100 year
Average bank-full flow depth, y_{bf} [m]	0.95	0.95	0.95	0.95
Design flow discharge per unit width, q_d [m^2/s]	7.93	2.57	1.91	1.66
Bankfull flow discharge per unit width, q_{bf} = [m^2/s]	0.28	0.28	0.28	0.28
Exponent for gravel	0.85	0.85	0.85	0.85
y_n = scour depth below design flow level (m)	8.24	3.16	2.45	2.18
FS	1.5	1.5	1.5	1.5
Design Scour Depth, m	**12.4**	**4.7**	**3.7**	**3.3**

Table C4: Potential Scour Depths – DTSF Updated Layout

Design Parameter	Return Period			
	PMF	1-in-1000 year	1-in-200 year	1-in-100 year
Average bank-full flow depth, y_{bf} [m]	0.95	0.95	0.95	0.95
Design flow discharge per unit width, q_d [m^2/s]	5.88	1.83	1.39	1.20
Bankfull flow discharge per unit width, q_{bf} = [m^2/s]	0.28	0.28	0.28	0.28
Exponent for gravel	0.85	0.85	0.85	0.85
y_n = scour depth below design flow level (m)	6.4	2.4	1.9	1.7
FS	1.5	1.5	1.5	1.5
Design Scour Depth, m	**9.6**	**3.5**	**2.8**	**2.5**

2.0 DTSF GABION REQUIREMENTS

Gabion sizing requirements were assessed based on the relationship outlined by Freeman and Fischenich (2000) for the design of gabions for streambank erosion control. This relationship is presented in Equation 3 below:

$$d_{50} = S_f C_s C_v d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V}{\sqrt{g d K_1}} \right]^{2.5} \qquad \text{Eqn. 3}$$

Where,

C_s = stability coefficient (0.1

C_v = velocity distribution coefficient

D_{50} = average rock diameter in gabions, m

d = local flow depth, m

g = acceleration due to gravity, m/s^2

k1 = side slope correction factor

R = centerline bend radius of main channel flow

Sf = safety factor

V = depth-averaged velocity

W = water surface width of main channel

γ_s = unit weight of stone

γ_w = unit weight of water

The input parameters used for the assessment of the gabion d_{50} sizes are presented in Table C5 for the Original layout, and Table C6 for the Updated layout.

Table C5: Required Gabion Sizes - Original Layout

Design Parameter	Return Period			
	PMF	1-in-1000 year	1-in-200 year	1-in-100 year
Local flow depth, d [m]	3.10	1.90	1.70	1.60
Sf	2.5	4	3	2.5
Cs	0.10	0.10	0.10	0.10
Cv	1.00	1.00	1.00	1.00
Unit weight of water (γ)	9810	9810	9810	9810
Specific Weight of Stone (γ_s)	25900	25900	25900	25900
V	4.50	2.50	2.40	2.30
W	100.0	100.0	100.0	100.0
Sg	2.65	2.65	2.65	2.65
g	9.81	9.81	9.81	9.81
Required d50, dm	**0.25**	**0.11**	**0.07**	**0.07**

Table C6: Required Gabion Sizes - Updated Layout

Design Parameter	Return Period			
	PMF	1-in-1000 year	1-in-200 year	1-in-100 year
Local flow depth, d [m]	2.70	1.50	1.30	1.20
Sf	2.50	3.00	2.50	2.50
Cs	0.10	0.10	0.10	0.10
Cv	1.00	1.00	1.00	1.00
Unit weight of water (γ)	9810	9810	9810	9810
Specific Weight of Stone (γ_s)	25900	25900	25900	25900
V	3.80	2.40	2.20	2.10
W	100.0	100.0	100.0	100.0
Sg	2.65	2.65	2.65	2.65
g	9.81	9.81	9.81	9.81
Required d_{50}, dm	**0.2**	**0.1**	**0.07**	**0.07**

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