# Back River Property Geotechnical Design Parameters

**Prepared for**

Sabina Gold & Silver Corp.

**Prepared by**

SRK Consulting (Canada) Inc.
1CS020.008
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# 1 Introduction

## 1.1 Background

The Back River Project (the Project) is a proposed gold mining project located in the West Kitikmeot region of Nunavut, approximately 520 km northeast of Yellowknife and 75 km south of Bathurst Inlet (Figure 1). The Back River Property (the Property) is owned by Sabina Gold & Silver Corp. who have recently completed a Feasibility Study (FS) for the Project which includes the Goose Property located approximately 125 km southeast of a proposed Marine Laydown Area (MLA) in Bathurst Inlet. The MLA will serve as the primary resupply point for the Property. Mining at the Goose Property will consist of both open pit and underground mining. Currently only limited exploration infrastructure exists at the Goose Property and no infrastructure is present at the MLA.

The George Property is an exploration site with multiple deposits where some overburden characterization has taken place, but where the feasibility of mining is not currently being assessed. Regardless, overburden information gathered at the George Property is considered to also be applicable at the Goose Property as the origin of the overburden is thought to be similar.

Proposed surface infrastructure development for the Project to support the current feasibility-level mine plan generally includes the following:

- Marine Laydown Area:
  - Laydown Area
  - Fuel Storage Area
  - Freight Storage Area
  - Camp and ancillary facilities
  - Workshop, warehousing and ancillary facilities
  - Water management facilities (desalination plant)
  - All-weather roads linking the above infrastructure elements
- Goose Property:
  - Four (4) open pits: Umwelt, Llama, Echo, and Goose Main
  - Four (4) undergrounds: Umwelt, Llama, Echo, and Goose Main
  - Processing plant and ancillary facilities
  - Mine equipment workshops, warehousing and ancillary facilities
  - Laydown facilities (for each underground mine and for plant site)
  - Fuel tank farm

- Camp and ancillary facilities
- Tailings Storage Facility (TSF)
- Waste Rock Storage Areas (WRSAs)
- Ore Stockpile
- Airstrip
- Water management facilities (treatment plant, pipelines, ponds, berms)
- Saline Water Pond
- All-weather roads linking the above infrastructure elements
- Culverts

In addition, seasonal winter ice roads are used to link areas on the Property.

## 1.2 Scope of Work

This report documents the geotechnical design parameters to be used for the proposed surface infrastructure designs of the Property. It is important to note that this document contains the fundamental geotechnical parameters which are independent of the intended use by other engineering disciplines. For example, this report does not contain estimates of settlement, as settlement is a function of the geometry and load of a structure which is unknown to SRK at this time; however, the geotechnical parameters provided will allow calculation of settlement once the structure geometry and loads are available at later stages of design.

## 1.3 Report Layout

Section 2 of this report provides an overview of generalized site conditions with details regarding the seismicity of the Property provided in Appendix A. The historic overburden characterization carried out on the Property is summarized in Section 3. Section 4 provides the geotechnical design principles to be followed for surface infrastructure development at the Property. The fundamental geotechnical design properties are presented in Section 5.Specific details pertaining to the thermal analysis is presented in Appendix B.

# 2 General Site Conditions

## 2.1 Permafrost

Surficial and deep geotechnical investigations of the Back River region confirm that the Property is within the region of continuous permafrost. The active layer depth ranges from approximately 1 to 4 m below ground surface, with the greatest active layer depths occurring in areas with thin soil veneers above bedrock. Due to the presence of salinity in some surficial groundwater, the active layer can take up to 60 days to freeze in some areas from the time when the mean average air temperature drops below $0^oC$. Permafrost temperatures, below the point of zero amplitude, range between $-6^oC$ and $-8^oC$. A geothermal gradient of 0.013 to $0.014^oC/m$ exists between -50 and -400 metres above sea level (masl). This gradient results in basal permafrost depths ranging from -190 to -260 masl, which corresponds to depths of 490 to 570 m below ground surface.

In general, overburden soils are frozen from mid-October to the beginning of June. Overburden reaches its maximum (warmest) seasonal temperature between the middle of August and the middle of September, at which point the average air temperature starts to decrease and freeze-up begins. Figure 2 displays typical trumpet curve in the Goose Property, showing an active layer thickness of approximately 1.5 m below ground surface and a freeze-up date just after the beginning of October. Minimum (coldest) temperatures are experienced in mid-March.

## 2.2 Bedrock Characteristics

The majority of the Property is underlain by clastic meta-sedimentary rock types consisting of turbidites (greywacke and mudstone) of the Slave Province. The Goose Property is underlain by the folded meta-sedimentary turbidite sequence belonging to the Beechey Lake Group which consists of banded iron formations hosted in greywacke and mudstone country rock. The stratigraphic sequence includes the central greywacke, lower iron formation, middle mudstone, upper iron formation, and interbedded sediments.

Regional folding trends to the northwest with associated steeply dipping faults. These formations were then intruded by felsic dykes of the Regan Intrusive Suite and younger gabbro dykes (Rescan 2014, Knight Piésold 2013). The gold mineralization at the Property is the result of this widespread quartz and carbonate veining and sulphidization related to the brittle faulting and subsequent folding (SRK 2012a).

The rock mass characterization for each major lithologic unit has been summarized by Knight Piésold (2013) and is provided in Table 1 below:

**Table 1: Summary of Rock Mass Characterization**

| Lithological Unit | Rock Quality Designation (NGI-Q) | Rock Mass Rating ($RMR_{89}$) | UCS (MPa) |
|---|---|---|---|
| Central Greywacke | Good | 60 to 75 | 120 (mean) |
| Lower Iron Formation | Good to Very Good | 65 to 85 | 260 (mean) |
| Middle Mudstone | Poor to Good | 35 to 70 | 60 (Mean) |
| Upper Iron Formation | Fair to Good | 55 to 80 | 190 (mean) |
| Interbedded Sediments | Fair to Good | 55 to 75 | 110 (mean) |
| Felsic Dykes | Good | 60 to 75 | 130 (mean) |
| Gabbro Dykes | Fair to Good | 55 to 75 | 120 (mean) |

## 2.3 Regional Geomorphology

During the last Quaternary period, the region was subjected to multiple glaciations. This has resulted in the striated landscape and overburden materials characteristic of a post-glacial environment with moraine sediments being the most dominant. Other sediments include glaciofluvial sediments, organic sediments, and exposed weathered bedrock. Marine sediments are also found, only in proximity to Bathurst Inlet. Overburden thickness varies from 1 m in higher elevation areas with shallow outcropping bedrock to greater than 37 m in topographic lows (Rescan 2014). Slope processes, frost action, and permafrost have further contributed to the current landscape of the Property.

The Property is situated in the central-eastern portion of the Slave structural province of Nunavut Territory (Tetra Tech 2013). Brittle fault structures associated with folding have been identified at the Goose Property striking north and northwest (SRK 2012b).

During the last Quaternary period the region was subjected to multiple glaciations. This has resulted in the striated landscape with post-glacial overburden deposits, such as glacial till and glaciofluvial soils, which exist at the Property today (Rescan 2014). Slope processes, frost action, and permafrost have further contributed to the current landscape of the Property.

## 2.4 Regional Seismicity

The Goose and MLA properties are located in low seismicity zones. Seismic parameters were calculated for both properties (Appendix A) using the National Building Code of Canada website (NRCC 2014) which provides ground accelerations and probability of occurrence. At each property, the peak ground acceleration (PGA) at a set spectral acceleration are the same; this information is summarized in Table 2. Essentially the seismic hazard is described by spectral acceleration (Sa) values at periods of 0.2, 0.5, 1.0 and 2.0 seconds. The PGA presented in Table 2 is to be used for surface infrastructure foundation design.

**Table 2: Back River Property Seismic Hazard Values**

| Spectral Acceleration | Ground Motion (g) |
|---|---|
| $Sa_{(0.2)}$ | 0.095 |
| $Sa_{(0.5)}$ | 0.057 |
| $Sa_{(1.0)}$ | 0.026 |
| $Sa_{(2.0)}$ | 0.008 |
| PGA | 0.036 |

# 3 Overburden Characterization

## 3.1 Characterization Programs

Complete summaries of historic overburden characterization programs conducted at the Property, including but not limited to the three distinct properties mentioned above, are listed in Table 3. This information is also presented in Figures 3 through 9. It should be noted that data collected for the George Property has been included for completeness even though there are currently no plans to develop it.

Table 3 provides a record of all field drilling and test pitting programs completed at the Property to date. Table 4 provides a record of other activities completed which also contribute to the overall understanding of the overburden characteristics at the Property.

**Table 3: Summary of Historic Drilling and Test Pitting Programs**

| Date | Area | Investigation Type | Installations | Laboratory/In-Situ Testing | Reference |
|---|---|---|---|---|---|
| December 2001 | South of Marine Laydown Area | 6 offshore drill holes | - | Indicator testing (PSD[6], water contents, Atterberg Limits) | Nishi-Khon/SNC Lavalin 2001 |
| August 2010 | Goose Lake Airstrip | 4 test pits | - | Indicator testing (PSD, water contents), Proctor Compaction | SRK (2010) |
| December 2011 | Goose Lake Airstrip | 11 drill holes in airstrip area | 1 thermistor | Indicator testing, Proctor Compaction, triaxial shear strength testing | SRK (2011a) |
| April to August 2013[1] | Goose Property, George Property, MLA | 34 drill holes at Goose Property (4 at airstrip, 4 at Goose Property[2], 4 at plant site[3], 21 in tailings area[4]); 67 hand-dug test pits (31 at Goose Property, 28 at George Property, 8 at MLA) | 9 thermistors at Goose Property (4 at airstrip, 1 at plant site, 4 at tailings area) | Indicator testing | Knight Piésold (2013) |
| March to April, June 2015[5] | Goose Property, MLA | 36 drill holes at Goose Property (3 at plant site, 9 water management holes, 3 at other planned infrastructure locations, 1 under Llama Lake, 20 at TSF); 11 drill holes at MLA (spread across the Freight and Fuel Storage, Camp, and Laydown Areas); 4 hand-dug test pits at in the MLA Fuel Storage Area | 10 thermistors at the TSF | Indicator testing (PSD, water contents, specific gravity, atterberg limits, in-situ density), pore water salinity, direct simple shear, consolidation, concrete aggregate, groundwater quality | SRK (2015a) SRK (2015b) |

Notes:
1. Program was completed in support of pre-feasibility study (PFS) to educate engineering design and was based on the mine plan at that time.
2. Goose pit rim has changed slightly in the FS mine plan from the PFS mine plan.
3. The plant site is in a different location in the FS mine plan than in the PFS mine plan.
4. The tailings impoundment area (TIA) was a ring-dyke facility in the PFS mine plan and has been superceded by the FS tailings storage facility (TSF), a valley-fill facility with a Main TSF Dam and small South TSF Dam far to the south of the original TIA in the FS mine plan.
5. Program was completed in support of feasibility study (FS) to educate engineering design and was based on the FS mine plan.
6. Particle Size Distribution (PSD).

**Table 4: Summary of Other Relevant Characterization Programs**

| Date | Area | Study Details | Reference |
|---|---|---|---|
| 2007 to May 2014 | Goose and George Sites | Cumulative Permafrost Baseline Data Report from 2007 to 2014. Study included observations on active layer freeze-thaw cycles and active layer depth | Rescan (2014) |

## 3.2 Geotechnical Conditions

Laboratory testing on geotechnical samples, collected during the 2010 test-pitting program (SRK 2010), the 2011 airstrip drilling program (SRK 2011a), the 2013 drilling and test-pitting program (Knight Piésold 2013), and the 2015 drilling and test-pitting program (SRK 2015a; SRK 2015b), confirms that overburden soils at the Goose, George, and onshore Marine Laydown properties generally consist of silty sands with some clay and gravel (SM, ML, and SW according to the United Soil Classification System (USCS)), which are likely the result of the reworked marine and glacial sediments. Overburden thickness varies across the Property.

Based on the geological history of the region, it is assumed that overburden soils are normally consolidated. The physical state (frozen or thawed) will likely define its apparent strength. In a thawed state, the soils will likely have a relatively low strength, whereas in a frozen state the ice will contribute to the cohesive strength so that the soil strength increases with decreasing temperatures.

### 3.2.1 Marine Laydown Area

The 2013 Knight Piésold test-pitting program and the 2015 SRK drilling program both revealed a surficial layer of silty sand onshore in the Marine Laydown Area (Figure 3). Clayey material was encountered in the Camp Area and the Laydown Area during the 2015 drilling program. The silty sand specimens from the two programs had an average natural moisture content of 13% and one gravel specimen had a natural moisture content of 4%. The clay specimens encountered during the 2015 program had an average moisture content of 26%. Atterberg limit testing was completed on the clay samples resulted in low plasticity indices averaging 12%.

During both the 2013 and 2015 programs, none of the test pits or drillholes reached bedrock. Therefore, the total overburden thickness is not known in this area. Understanding of the permafrost conditions in this area is limited as there have not been any thermistors installed, and test pits and drillholes generally stopped at or above frozen ground. Two test pits during the 2013 program met refusal due to permafrost at 0.4 m and 0.8 metres below ground surface (mbgs) and four drillholes during the 2015 program encountered frozen ground at approximately 0.75 mbgs, indicating the active layer is at least 0.8 mbgs in some parts of the MLA. It should be noted that permafrost was not encountered in any drillholes in the Freight or Fuel Storage Areas, but that the maximum drilling depth in these areas was 2 mbgs. The active layer depth may be deeper than the drilling refusal depth in these areas.

A Feasibility Study of the Bathurst Inlet Port and Road (BIPAR), which was intended to be constructed to support many of the mines in the Project area, yields limited information on offshore sediments in an adjacent area 13 km to the south shown in Figure 3 (Nishi-Khon/SNC Lavalin 2001). The BIPAR study indicated that the offshore soil consists primarily of normally consolidated marine clay deposits (CL to CH) with occasional silty sand lenses with a thickness of approximately 20 m at a distance of 150 m from the shoreline. The soft, weak clays were estimated to have an undrained shear strength ranging from 7 kPa at the sea floor to 40 kPa at a depth of 20 m below the sea floor. They were also found to be unfrozen with an average water content of 60% (Nishi-Khon/SNC Lavalin 2001). With similar regional geomorphology, it can be expected that the talik and sediments at the MLA will be similar; however, no site specific information is available to confirm this.

### 3.2.2 Goose Property

The 2010, 2011, 2013, and 2015 site investigations conducted by SRK and Knight Piésold (Table 3) revealed that pockets of sandy silt till underlie the surficial silty sands in some areas of the Goose Property. In general, ice content within the soils at the Goose Property is low (around 12%); however, visible ice and small zones of higher ice content were observed in some locations, especially directly above the bedrock contact. The natural moisture content of the sandy silts was found to be approximately 14%; the silty sand samples 12.5%; and the silty sandy gravel samples 10% (SRK 2010; SRK 2011a; Knight Piésold 2013; SRK 2015a).

Detailed overburden drilling revealed the overburden depth in the airstrip, Goose Main Pit, the PFS TIA and Plant Site areas, and the FS TSF, mine infrastructure, and Plant Site areas. In general, the overburden thickness ranges from 0 to 25 m across the Property, with thickness is specific areas as follows:

- Up to 25 m in Goose Main and Llama pits and in the Airstrip Area (Knight Piésold 2013; SRK 2011a);
- Between 0 and 5 m in the PFS TIA and Plant Site areas (Knight Piésold 2013);
- Between 0 and 8 m, depending on topography, at the FS Plant Site (SRK 2015a);
- Between 2 and 5 m in the FS Saline Water Pond area (SRK 2015a);
- Between 0 and 2.5 m in the FS Umwelt and Llama WRSA areas (SRK 2015a); and
- From 3 m at the west side of the FS TSF up to 11 m in the centre, to 1.5 m on the east side and 1 m at the south end (SRK 2015a).

Thermistors installed in the active layer revealed that the active layer depth ranges from 1.3 m to 4.2 mbgs, with the deepest active layer depths occurring in areas where bedrock forms part of the active layer (Rescan 2014). The shallow bedrock underlying soils at the Goose Property are generally good in quality with minor occurrences of poor, fair, and very good rock mass qualities across the site (Knight Piésold 2013; SRK 2015a). This is corroborated by the hydraulic testing results from the 2015 drilling program that resulted in relatively low hydraulic conductivity values for the shallow bedrock averaging approximately 1 x $10^{-8}$ m/s (SRK 2015a).

Laboratory testing of near-surface soil samples indicated that there are some scattered occurrences of high-salinity pore water across the Goose Property. Due to a relatively limited set of samples, the high salinity values cannot be attributed to a particular soil terrain unit (SRK 2011a; Knight Piésold 2013; SRK 2015a). The porewater salinity across the Goose Property averages approximately 26 ppt with a maximum of 85 ppt encountered in the PFS TIA area. High salinity values have the effect of depressing the freezing point, as well as contributing to high unfrozen water content. These salinity values have been attributed to the relatively long freezing time of the active layer in some areas at the Goose Property (Rescan 2014).

Proctor compaction testing was carried out on samples from both the 2010 and 2011 airstrip field investigations. Testing was carried out on surficial samples taken from test pits in the 2010 program and on blended samples taken from drill core in the 2011 program. As such, the results differ slightly. Average maximum dry densities for the 2010 and 2011 programs were found to be approximately 1,975 kg/m$^3$ and 2,130 kg/m$^3$, respectively, and average optimum water contents were found to be 8.4% and 8%, respectively (SRK 2010; SRK 2011a).

Frozen in-situ density was estimated during the 2015 drilling program and was found to increase with depth from an average of approximately 1.8 g/cm$^3$ at surface to approximately 2.3 g/cm$^3$ at 5 m depth. This was confirmed with laboratory measurements of wet density (SRK 2015a).

Triaxial tests completed on reconstituted samples from the 2011 airstrip investigation revealed that, at 98% of standard Proctor compaction, the silty sand from the airstrip at the Goose Property has a friction angle of approximately 33$^o$ (SRK 2011a). Direct shear testing completed on silty sand samples from across the Property at approximately in-situ densities resulted in an average friction angle of 36$^o$ (SRK 2015a).

The foundation of the Main TSF Dam is expected to remain frozen during the operation of the facility, and therefore foundation settlement is not expected. Consolidation testing was however performed on thawed samples from the FS TSF area to allow for an assessment of settlement should the foundation thaw. These results confirm an average compression index of 0.014 and an average recompression index of 0.007 (SRK 2015a).

### 3.2.3 Overburden Isopach Contours

Overburden isopach contours were generated by Associated Mining Consultants Ltd. (AMC 2013) for the Goose Property using existing exploration drill data. SRK confirmed the work by AMC (2013) by generating isopach contours for the Goose Property which included the exploration drill data as well as the SRK (2011a) geotechnical field investigation. Overviews of the FS infrastructure at the Goose Property is pictured in Figures 4 through 7, and the updated overburden isopach contours which include data from the 2013 and 2015 overburden investigations is shown in Figures 8 and 9. Overburden isopach contours in the Goose Property (within the Goose Main Pit, Llama Pit, Umwelt Pit, and Echo Pit) show overburden thickness is generally 10 m in most areas. The thickest overburden deposits, of up to 25 m, are along the airstrip alignment, along the west edge of Goose Main Pit, and along the north western end of the Llama Pit.

No overburden isopach contours are available for the MLA. While the 2015 investigation in the MLA area improved the understanding of the surficial overburden, drilling did not progress into bedrock so overburden thickness could not be confirmed.

## 3.3 Borrow Sources

Borrow material may be required as general construction fill, aggregate, or bedding materials throughout the Property. The initial site investigation for the Goose Lake airstrip includes some basic observational notes on potential rock quarries located to the west of the airstrip (SRK 2010). Knight Piésold (2012) completed a reconnaissance investigation of borrow areas in 2012 and identified potential esker and rock borrow sources. Rescan (2013) subsequently completed a sampling program to characterize the geochemistry of alternate potential borrow sources (i.e. overburden and quarries) for the Goose Property. SRK (2015e) updated the geochemical characterization of both the quarry borrow sources and overburden at the Goose and MLA properties.

Overburden material, whether from open pit pre-stripping, quarry development or other infrastructure development excavations, will either be stockpiled separately, or co-disposed with the waste rock. Where excavation of overburden soils is required for infrastructure development, the practicality of properly conditioning these materials (i.e. thawing and subsequent moisture control) must be carefully considered. The salinity content of the overburden material should be taken into consideration if it is to be used.

Bedrock quarries will be used to produce engineered fill required for construction. Different material types will be created through appropriate crushing operations. Quarry rock may only be used once it has been confirmed to be geochemically suitable (including blast residue). Two quarries, the Airstrip Quarry and the Umwelt Quarry, have been investigated and geochemically characterized. Further characterization is necessary at the Airstrip Quarry to delineate potential potentially acid generating (PAG) zones; however, the Gabbro dykes located there have been classified as non-potentially acid generating (NPAG), and the greywacke and mudstone units include a mixture of material that is PAG and NPAG. The Umwelt Quarry areas were selected to be entirely within the upper greywacke unit, which has been characterized as NPAG or low sulfide with a negligible potential for ARD (SRK 2015e).

In the Plant Site area, aggregate will be needed to construct pads and foundations for mine infrastructure. Aggregate testing was performed on a batch of ten bedrock samples collected from the Plant Site and Ore Stockpile areas during the 2015 drilling program. The testing was intended to determine the resistance of the bedrock in the area to mechanical degradation for the purpose of its use in aggregate for construction. The material was crushed so that no particles larger than 20 mm in diameter remained. The bulk density of the material was found to be 1,793 kg/m$^3$ and grain-size analysis revealed well-graded gravel with 3% of material passing the No. 200 sieve (SRK 2015a). At the MLA, initial geochemical testing of weathered samples collected from the surface indicates that the rock there mainly consists of quartzite conglomerate and quartz arenite. Results show that these materials have a negligible potential for ML/ARD and would likely be suitable for construction, however further sampling and testing should occur of fresh material from greater depths in advance of or concurrent with development (SRK 2015e).

# 4 Geotechnical Design Principles

## 4.1 Overburden Stripping

Overburden investigations to date do not suggest that the soils will be highly sensitive (i.e. dilatant), though some of the soils may not be trafficable over the short-term during periods of thaw. The low moisture contents indicate that liquefaction is unlikely during transport of typical materials from the pits to the overburden stockpiles.

It is assumed that the overburden can be mined using conventional truck and shovel techniques. Some temporary access roads comprised of competent material (quarried or Run-of-Mine rock) may need to be constructed during the summer months when permafrost degradation due to excavation is in full effect. Alternate methods could include low pressure equipment if trafficability is significantly poor. Further site-specific geotechnical characterization may be required at the detailed design stage.

Winter excavation of overburden soils will require drilling and blasting. These soils will absorb a significant amount of the blast load, and as a result closer drill hole spacing will be required and the blast load factor will have to be higher than for regular rock blasting.

## 4.2 Overburden Disposal

Overburden stockpiles will need to be constructed at each quarry and/or mine area, or these materials must be co-disposed with waste rock. Currently, the FS mine plan includes co-disposal of overburden within the WRSAs. Two overburden products will be produced in each area: (1) frozen overburden from permafrost areas, and (2) unfrozen overburden from summer stripping or talik zones.

As described previously, frozen overburden soil excavation will require drill and blast techniques, resulting in blocky frozen material, with entrained ice intact. Compaction to consolidate this material will not be practical in the frozen state, and as a result significant thaw and settlement is to be expected seasonally. During thaw, portions of these overburden stockpiles may become unstable due to the high water content of the thawed materials. Surficial sloughing or bearing capacity failures may be observed. If this material is co-disposed with waste rock, these stability concerns remain, but may be more difficult to monitor since areas of concern may be buried. To minimize this risk, overburden soils should not be disposed of close to the outer limits of the waste rock pile, staying at least 30 m away from the final outer edge.

Since limited occurrences of massive fines were observed during investigations completed at the Goose Property, overburden slopes should be reasonably stable over the long term. It is however recommended that the overall slope angle of any exposed slopes or overburden stockpiles not exceed 18 degrees (3H:1V) with appropriate buttressing where required.

There will be significant release of water from any overburden stockpiles, and this water will have a high Total Suspended Solids (TSS) load and may have elevated chloride concentrations (SRK 2015e). Appropriate water management measures will therefore be integral to the design of any overburden stockpiles, probably necessitating dedicated sedimentation/pollution control structures.

Foundation requirements for overburden soil stockpiles will be similar to waste rock storage areas, discussed in Section 4.4.

## 4.3 Tailings Storage Facility Dam Foundation

In-situ characterization of the Main TSF Dam and South TSF Dyke foundations was completed during the 2015 drilling program (SRK 2015a). As mentioned above, overburden is generally thin at the east and west abutments (1.5 and 3 m, respectively), and increases towards the middle of the dam to a maximum of approximately 11 m. Approximately half of the length of the proposed dam alignment appears to have bedrock at less than 4 m below ground (SRK 2015f).

In general, the foundation consists of a relatively thin layer of silty sand material, with a pocket of sandy silt material above bedrock from the centre of the dam towards the eastern abutment (Figure 10). The average excess ice content of the foundation material is related to topography in that on the eastern and western flanks at higher elevations, the average excess ice is low (between 0 and 5%). At the centre of the dam alignment in the bottom of the valley, the average excess ice content is between 10 and 30% and there are scattered occurrences of thin layers of massive ice (Figure 11). This distribution is to be expected due to the fact that surface drainage will converge in the valley bottom and result in higher average moisture content than on the eastern and western flanks.

Bedrock is generally competent, with only three of the 19 boreholes drilled along the alignment indicating a highly fractured bedrock contact zone having a rock quality designation (RQD) less than 60%. All three instances of fractured rock were found to underlay the dam near the west abutment (SRK 2015f). Hydraulic testing completed in the shallow bedrock along the alignment indicated a hydraulic conductivity between $3 \times 10^{-9}$ and $3 \times 10^{-8}$ m/s (SRK 2015a).

During the 2015 field investigation, 9 thermistor strings were installed along the alignment of Main TSF Dam and one thermistor string was installed under the South Dyke footprint. Temperature measurements were taken in April and August of 2015 in an attempt to define the approximate depth of the active layer in the area. Based on these readings, the active layer depth appears to be between 1 and 2 m below ground along the main dam alignment and in the area of the South Dyke (Figure 12) (SRK 2015a).

Thermal modelling was completed using the TSF dam design configuration, updated overburden properties ,and ground temperature data obtained from the 2015 field investigation, and conservatively assuming 10 years of operations with water behind the dam at full supply level (SRK 2015c). The modelling confirmed that the foundation and key trench of the dam will remain frozen, even in the unlikely event that the TSF is in operation for ten years.

After the completion of the 2015 field investigation and all associated laboratory testing and data analysis (SRK 2015a), the foundation was classified into three primary zones depending on the physical properties of each zone and the proposed foundation treatment during the construction of the dam, as outlined in Table 5.

**Table 5: TSF Dam Foundation Zones and Material Properties**

| Zone Descriptor | Properties | Keytrench Treatment |
|---|---|---|
| Shallow Bedrock | Bedrock is near surface, covered with less than 4 m of overburden. Some bedrock on western flank is highly fractured. | Excavate all overburden and highly fractured bedrock. No further treatment required. |
| Deep Ice-Poor Overburden | Deep overburden (greater than 4 m). Massive ice not present and interstitial ice content is less than 10%. | Excavate overburden to design extents of key trench. |
| Deep Ice-Rich Overburden | Deep overburden (greater than 4 m). Massive ice present and interstitial ice content is greater than 10%. | Excavate overburden and massive ice zones contiguous with the keytrench excavation. |

## 4.4 Waste Rock Storage Area (WRSA) Foundations

WRSAs constructed on permafrost soils (i.e. directly onto the tundra) should be designed to promote freeze back, thereby minimizing long-term environmental effects from possible acid rock drainage and/or metal leaching.

Permafrost soils are expected to provide suitable foundation conditions for WRSAs provided the foundation remains frozen. To facilitate this process, it is recommended that the first lift of all new WRSAs be constructed during the winter season. In the event the first lift of waste rock will have to be constructed during the summer months, the WRSA will be subject to differential settlement, due to consolidation settlement of the active layer, where the active layer is within overburden material. The amount of settlement will vary, but will likely be between 10 and 30% of the active layer thickness. This settlement will only occur during the first summer provided appropriate freeze back is obtained during the subsequent winter (SRK 2015d). Geotechnical investigations have however confirmed that beneath the Goose Property WRSAs, the overburden is typically less than 2 m (SRK 2015a) and therefore differential settlement is not an issue of concern.

In all cases, whether WRSA construction is started in summer or winter, once freeze back has been achieved and the active layer is demonstrated to remain within the base of the WRSA, there will likely be few further restrictions on the maximum waste rock lift thickness (subject to confirmation analysis). Overall maximum height (i.e. total vertical thickness) of the WRSAs should be limited to 100 m, unless appropriate analysis to confirm that greater heights will still be safe, is carried out. For a WRSA with a frozen foundation and maximum height of 80 m, the overall WRSA slopes can be based solely on the waste rock properties and no special buttressing considerations are required.

Provided the WRSA foundations remain frozen, the only deformation they will experience is creep deformation. Creep deformation is a long term process and specialized cold lab stress-strain tests are required for detailed design to define suitable properties, after which the actual deformation has to be numerically modelled. No laboratory creep tests have been performed to date, nor deemed necessary for future design, because the overburden thickness beneath the WRSAs are negligible (SRK 2015a).

In areas where the WRSA foundations are on exposed bedrock, no significant issues are expected. Therefore, placement on exposed bedrock is preferred and can proceed during any season provided adequate clearing of snow and ice has been completed.

## 4.5 Permafrost Foundations

Frozen overburden materials are expected to have a relatively high bearing capacity, while thawed overburden soils are expected to have only a medium strength when drained. Since most material is expected to have relatively low clay content, and thawing is typically a slow process, silty soils observed at the three properties (including Goose, the George Exploration Camp, and the MLA) will likely be under drained conditions during seasonal thaw. However, some deformation can be expected during permafrost thaw and it is recommended that surface infrastructure be founded on frozen soils as often as practical. Care must be taken when designing infrastructure and pads to ensure that heat generated from buildings does not promote thaw of permafrost materials.

Structures that are particularly sensitive to differential settlement, such as the mill complex and fuel tank farms, should as far as practical be founded on competent bedrock. Foundations may be constructed on exposed bedrock or, alternatively, overburden may be stripped to expose the underlying bedrock. An alternative foundation technique that could be employed for critical structures would be the use of rock-socketed load bearing piles. Appropriate trade-off studies of foundation alternatives would need to be undertaken to evaluate the appropriate foundation method. Having said this, the drilling completed in the Plant Site area during the 2015 field investigation revealed that overburden cover there is generally thin (0 to 2 m) except at the very southern edge of the plant site where 8 m of silty sand was encountered (SRK 2015a).

Where stripping of overburden has been completed as part of foundation design, the overall overburden soil design angles presented in Section 4.2 should be confirmed through appropriate analysis. If necessary, cladding of overburden slopes may have to be considered.

Structures and linear surface infrastructure elements (such as roads, pipelines, and airstrips) that are not sensitive to differential settlement can be founded on the overburden soils, provided an appropriate thermal protection layer is constructed. Appendix B contains complete details regarding the minimum pad thickness under different thermal conditions, including pore water salinity. Where thinner pads are constructed, or during summer construction, some thaw consolidation is expected. Once the active layer is re-established, which would likely be achieved within one or two seasons, no further settlement is expected provided structures do not generate heat.

Geosynthetics are not required to increase the foundation strength since the engineered fill thickness required for active layer insulation is likely to exceed the thickness required for allowable bearing capacity of most infrastructures.

Ad-freeze piles can be used for smaller structures such as radio towers, small bridge crossings, and culvert footings.

## 4.6 Talik Foundations

A description of the presence and extent of taliks on the Property is presented in SRK (2015g). It may be necessary to construct facilities on or near lakes, which may lead to foundation interactions with talik zones. Soils within talik zones may have lower bearing capacities and construction on these overburden soils poses some challenges including settlement and possible foundation bearing failure. Settlement may be compensated for by overbuilding. However, foundation bearing failure is a more challenging problem and may require pre-consolidation (in areas with significant fines) and/or the design of foundation strengthening elements. This can be achieved through constructing load distribution pads. The design thickness of these pads should be calculated based on the required loads and geometry. If necessary, geosynthetics (geogrids and geotextiles) can be used to optimize the fill requirements of these pads. Alternatively, load bearing piles can be driven through the talik overburden soils into bedrock under these conditions.

## 4.7 Surface Water Management Facilities

Surface water management facilities such as diversion berms, and both contact and non-contact water ponds will be required as part of the Project. Excavation of channels and/or ditches into overburden soils should be avoided where possible, and above ground solutions should be sourced. Currently, there are no plans in the FS mine plan for conveyance ditches or channels (SRK 2015h). If, in future changes to the mine plane, it proves necessary to excavate channels or ditches for water management facilities, these facilities will have to be over-excavated and lined with a thermal blanket to protect permafrost. Appropriate thermal and hydraulic assessment of these structures will be required.

Ponded water on permafrost soils should be avoided at all costs, except in specifically designed and constructed water containment dams and ponds. Uncontrolled ponding on permafrost soils will result in vegetation die-back followed by permafrost degradation. The exposed overburden soils will erode and be released into the receiving water bodies resulting in possibly high TSS concentrations. Even engineered containment structures, in areas where permafrost soils will be flooded, will be subject to shoreline erosion, and appropriate mitigation measures may be required. Source mitigation would likely consist of blanketing the area subjected to flooding with a layer of rock fill, which includes a filter layer to prevent fines from being released.

## 4.8 Infrastructure Foundation Preparation Recommendations

Considering all of the conditions listed in the preceding sections, the specific foundation preparation recommendations for the Project are summarized in Table 6.

**Table 6: Infrastructure Foundation Preparation Recommendations**

| Area | Recommendations |
|---|---|
| Goose Property | • Bedrock foundation required for critical structures such as fuel tank farms, heated buildings and process equipment foundations.<br>• 2.5 m compacted Run-of-Quarry (ROQ) rock fill pad (on top of undisturbed grade) required for unheated essential structures such as the airstrip.<br>• 1.0 m compacted ROQ rock fill pad (on top of undisturbed grade) required for unheated non-essential structures such as secondary roads.<br>• Mine Haul Roads should be 1.5 to 2 m thick to minimize deformation.<br>• For bedrock foundations:<br>  • Strip (doze) the upper 0.5 m of overburden and discard in WRSA as per Section 4.2. If winter construction is planned, drilling and blasting will be required.<br>  • Drill and blast upper 3.5 m of fractured rock. Only 50% can be assumed to be useable as ROQ construction fill. The remainder is likely to be unusable as it will contain predominantly oversize material. Discard material goes to the WRSAs.<br>• Rock shatter not required for foundations on bedrock. The exposed surface needs to be cleaned and roughly leveled.<br>• Rock shatter required where roads cross over rock highs that impact road grade.<br>• Rock fill pads will ideally be done in lifts no greater than of 1.5 m, with the maximum rock size limited to 0.9 m.<br>• A 150 mm thick layer of 2 inch minus surfacing material is recommended as a topping layer for ROQ pads. No transition layer required provided the ROQ is well graded. There may be some holes that develop as a result of consolidation but minimal repair should be required. An allowance of 20% extra 2” minus material should be made. |
| Marine Laydown Area | • Bedrock foundation required for critical structures such as fuel tank farms, heated buildings and process equipment foundations.<br>• 2.5 m compacted ROQ rock fill pad (on top of undisturbed grade) required for unheated essential structures.<br>• Laydown Area should be 2 m thick to minimize deformation.<br>• 1.0 m compacted ROQ rock fill pad (on top of undisturbed grade) required for unheated non-essential structures such as secondary roads.<br>• For bedrock foundations:<br>  • Strip (doze) the upper 0.5 m of overburden and use in non-critical pads. If winter construction is planned, drilling and blasting will be required.<br>  • Drill and blast upper 1.5 m of bedrock. 100% can be assumed to be useable as ROQ construction fill. |

# 5 Geotechnical Design Parameters

## 5.1 Typical Overburden Properties

Typical fundamental overburden properties are given in Tables 7 through 9. The properties are intended to be used for general geotechnical design (except overburden pit slopes) where site specific characterization is not available. When these properties are used, appropriate engineering judgement must be applied to account for uncertainty.

### 5.1.1 Onshore Overburden

The composition of the overburden materials throughout the Property consists of a surface layer of fibrous peat (Pt) which can vary from 0 m to 0.4 m thick but averages 0.1 m. This is underlain by sand (SM, ML, SW) with varying amounts of gravel and cobbles overlying bedrock. In general, the overburden sands are found to have both visible and non-visible ice and the occasional ice lense greater than 10 cm; however, the soil should not be considered ice-rich. Typical onshore overburden properties are presented in Table 7 and Table 8.

**Table 7: Typical Onshore Overburden Indicator Properties**

| Element | Value/Comments |
|---|---|
| Natural Moisture Content (%) | Variable with average near 12.5; increased moisture content with increased fines |
| Gravimetric Moisture Content (%) | 10 to 15 (increases with increasing fines. Scattered occurrences of high moisture content in massive ice zones) |
| Optimum Moisture Content (%) | 8 (average) |
| Degree of Saturation (%) | 68 (average) |
| Porosity, n | 0.35 (average) |
| Volumetric Water Content | 0.28 (average) |
| Volumetric Fraction of Unfrozen Water at -5°C (%) | 8.7 (average) |
| Salinity (ppt) | 26 (average) |
| Plasticity | Non-plastic on average. Some clayey sand/grave samples with plasticity index up to 8% |
| Primary Soil Type (USCS) | Sand (SM, ML, SW) |
| Specific Gravity | 2.69 (average) |
| Bulk Density (kg/m$^3$) | 2,000 (average) |
| Maximum Dry Density (kg/m$^3$) | 2,050 (average) |
| Moist Unit Weight (kN/m$^3$) | 19.5 (average) |

**Table 8: Typical Onshore Overburden Engineering Properties**

| Parameter | | Value |
|---|---|---|
| Unfrozen | Friction Angle, ϕ ($^o$) | 35 |
| | Apparent Cohesion, c' (kPa) | 25 |
| Frozen | Friction Angle, ϕ ($^o$) | 32 |
| | Apparent Cohesion, c' (kPa) | 50 |
| Saturated Hydraulic Conductivity – Unfrozen (m/s) | | 1 x $10^{-5}$ (average) |
| California Bearing Ratio<br>$D_{60}$ = 0.1 mm, MDD = 20.9kN/$m^3$, OMC = 8% | | 39 |

Notes:
1. Saturated Hydraulic Conductivity estimated by engineering judgement and literature references (Freeze and Cherry 1979).

### 5.1.2 Offshore Overburden

Typical overburden properties for marine sediments are presented in Table 9 and Table 10. No data was available in the MLA; therefore, these values reflect estimated material properties provided from the BIPAR study (Nishi-Khon/SNC Lavalin 2001) as well as engineering judgement.

**Table 9: Typical Offshore Overburden Indicator Properties**

| Element | Value/Comments |
|---|---|
| Natural Moisture Content | 60% |
| Saturation | 100% |
| Void ratio, e | 1.6 |
| Volumetric Water Content | 0.28 |
| Plasticity | Medium - High |
| Liquid Limit | 39 – 59 |
| Plasticity Index | 19 – 36 |
| Primary Soil Type | Clay (CL to CH) |
| Specific Gravity | 2.68 |
| Bulk Density (kg/$m^3$) | 1.63 |
| Moist Unit Weight (kN/$m^3$) | 16.0 |

**Table 10: Typical Offshore Overburden Engineering Properties**

| Parameter | Value |
|---|---|
| Undrained Shear Strength, $S_u$ | 7 kPa at surface, 40 kPa at 20 m depth |
| Friction Angle, ϕ ($^o$) – Unfrozen | 25.0 |

## 5.2 Borrow Properties

Table 11 outlines the recommended material properties for engineered fill (ROQ) material for the Property. No ROQ borrow source investigations have been completed to date and therefore, no test results are available to confirm potential borrow source geotechnical properties. The properties presented here are based on material properties reported in literature and SRK's internal database.

**Table 11: Typical in Place Run of Quarry Properties**

| Parameter | | Value |
|---|---|---|
| Moist Unit Weight (kN/m$^3$) | | 20 |
| Degree of Saturation (%) | | 30 |
| Porosity, n | | 0.3 |
| Volumetric Water Content | | 0.09 |
| Frozen | Apparent Cohesion, c' (kPa) | 0 |
| | Friction Angle, $\phi$ ($^o$) | 38 to 40 |
| Unfrozen | Apparent Cohesion, c' (kPa) | 5 |
| | Friction Angle, $\phi$ ($^o$) | 38 to 40 |

## 5.3 Bulking and Shrinkage Factors

Table 12 lists the recommended bulking and shrinkage factors to be used for the various geotechnical materials throughout the Property. The bulking and shrinkage factors are based on material properties reported in the literature and SRK's internal database. The bank density values are based on information provided by Sabina.

**Table 12: Bulking and Shrinkage Factors for the Project's Geotechnical Materials**

| Material | Bank Density ($T/m^3$) | Bank Cubic Meters (BCM) | Bulking Factor | Swell Factor | Load Factor | Loose Density ($T/m^3$) | Loose Cubic Meters (LCM) | Shrink-age Factor | Hybrid Swell Factor | Com-pacted Density ($T/m^3$) | Com-pacted Cubic Meters (CCM) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Lower Greywacke (LWGK) - Mineralized | 2.79 | 1.00 | 1.64 | 64% | 0.61 | 1.70 | 1.64 | 1.34 | 34% | 2.08 | 1.34 |
| Lower Greywacke (LWGK) - Not Mineralized | 2.79 | 1.00 | 1.64 | 64% | 0.61 | 1.70 | 1.64 | 1.34 | 34% | 2.08 | 1.34 |
| Upper Greywacke (UWGK) - Not Mineralized | 2.79 | 1.00 | 1.64 | 64% | 0.61 | 1.70 | 1.64 | 1.34 | 34% | 2.08 | 1.34 |
| Upper Greywacke (UWGK) - Not Mineralized | 2.79 | 1.00 | 1.64 | 64% | 0.61 | 1.70 | 1.64 | 1.34 | 34% | 2.08 | 1.34 |
| Lower Iron Formation (LIF) & Oxide Iron Formation (OIF) | 3.00 | 1.00 | 1.64 | 64% | 0.61 | 1.83 | 1.64 | 1.34 | 34% | 2.24 | 1.34 |
| Upper Iron Formation (UIF) | 2.95 | 1.00 | 1.64 | 64% | 0.61 | 1.80 | 1.64 | 1.34 | 34% | 2.20 | 1.34 |
| Quartz Vein (QV) | 2.83 | 1.00 | 1.64 | 64% | 0.61 | 1.73 | 1.64 | 1.34 | 34% | 2.11 | 1.34 |
| Quartz Feldspar (QFP) (Felsic Dyke) | 2.83 | 1.00 | 1.64 | 64% | 0.61 | 1.73 | 1.64 | 1.34 | 34% | 2.11 | 1.34 |
| Gabbro (GAB) | 2.98 | 1.00 | 1.64 | 64% | 0.61 | 1.82 | 1.64 | 1.34 | 34% | 2.22 | 1.34 |
| Overburden (OVB) - Frozen | 1.80 | 1.00 | 1.50 | 50% | 0.67 | 1.20 | 1.50 | 1.25 | 25% | 1.44 | 1.25 |
| Overburden (OVB) - Unfrozen | 1.80 | 1.00 | 1.30 | 30% | 0.77 | 1.38 | 1.30 | 0.95 | -5% | 1.89 | 0.95 |
| Airstrip Quarry ROQ | 2.80 | 1.00 | 1.64 | 64% | 0.61 | 1.71 | 1.64 | 1.34 | 34% | 2.09 | 1.34 |
| Airstrip Quarry ROQ – crushed gravel | 2.10 | 1.00 | 1.05 | 5% | 0.95 | 2.00 | 1.05 | 0.97 | -3% | 2.16 | 0.97 |
| Blended Waste Rock | 2.89 | 1.00 | 1.64 | 64% | 0.61 | 1.76 | 1.64 | 1.34 | 34% | 2.16 | 1.34 |
| Blended Ore Stockpiles | 3.00 | 1.00 | 1.64 | 64% | 0.61 | 1.83 | 1.64 | 1.34 | 34% | 2.24 | 1.34 |

## 5.4 Foundation Bearing Capacity

In unfrozen soils, the allowable bearing pressure for a shallow foundation is usually based on the safety against general soil failure and on the tolerable foundation settlement. Similar criteria are also applicable to shallow foundations in frozen soils, with the main difference that the strength of frozen soils is temperature dependant and the main source of foundation settlement is typically deviatoric creep rather than consolidation (Andersland and Ladanyi 2004).

The silty sand (SM) permafrost found at the Property may be subject to creep deformation which will impact excavation slopes and infrastructure foundations in the long term. Creep testing has not been carried out since none of the structures proposed would be sensitive to such movement.

Ultimate Bearing Capacity (UBC) takes into consideration fundamental soil characteristics (see Table 7 and Table 8), footing geometry, loads, and drainage. Other than fundamental soil characteristics, these other parameters are not yet known and thus SRK cannot provide definitive values for the UBC of foundations. Table 13 however lists typical ranges of UBC (Aysen 2005; Holtz *et al.*1981) for the geotechnical conditions that may be encountered at the Property. It will however be up to the designer to select appropriate values within these suggested ranges based on site-specific conditions.

Assuming limit state design, it is up to the designer to adopt appropriate Factor of Safety (FOS) to determine Allowable Bearing Capacity (ABC), where ABC = UBC/FOS. Guidelines for appropriate FOS should be obtained from design guidelines such as the Canadian Foundation Engineering Manual (CGS 2006).

For the Property, wherever possible, spread footings are recommended for structure foundations. These footings should not be constructed directly on tundra, but on appropriate thermal pads constructed of competent engineered fill. Notwithstanding adherence to these recommendations, the appropriate analysis taking into consideration stiff layers over weaker layers to prevent punching failures is recommended.

Table 13, Table 14, and Table 15 list further important geotechnical design parameters that are required to complete foundation, settlement, and retaining wall design. The choice of appropriate design values within the specified range is left up to the designer; however, SRK recommends targeting the lower end of the range given the uncertainty and amount of site specific data.

**Table 13: Summary of Foundation Characteristics**

| Element | Unit | Value/Comment |
|---|---|---|
| Uniaxial Compressive Strength[1] | Bedrock<br>MPa | 113.5 (average) |
| Hydraulic Conductivity (K) | Bedrock<br>m/s | 1.92E-09<br>(average) |
| Ultimate Bearing Capacity Pressure[2] | Competent Bedrock (Hard)<br>(Sound Igneous or Metamorphic Rock)<br>kPa | 7,500 |
| | Competent Bedrock (Medium Hard)<br>(Sound Sedimentary Rock to Foliated Metamorphic Rock)<br>kPa | 1,000 to 3,000 |
| | Bedrock (Soft to Medium Hardness)<br>(Weathered or Broken Rock, RQD typically <25)<br>kPa | 950 to 1,000 |
| | Engineered Fills > 1 m<br>(Crushed Rock 1 to 4 m thickness)<br>kPa | 200 to 600 |
| Allowable Bearing Capacity Pressure[3] (maximum) | Competent Bedrock<br>kPa | 500 to 2,000 |
| | Engineered Fills > 1 m<br>(Crushed Rock 1 to 4 m thickness)<br>kPa | 100 to 300 |

Notes:
1. SRK (2012b).
2. Based on first principles, engineering judgment and refs: Canadian Foundation Manual (GCS 2006); Department of the Army U.S Army Corps of Engineer (1992); Oloo *et al.* (1997). Values presented should only be used as presumed preliminary design bearing pressures. Field inspection of work site should be completed before final design to adjust design bearing pressures if/ as required. Ultimate bearing capacity should be calculated based on dimensions (i.e. Ultimate Bearing Capacity (UBC) = ultimate bearing capacity pressure * foundation width * foundation lateral length, for simple rectangular shape).
3. FOS of 2 has been utilized in determining the maximum allowable bearing capacity pressures presented.

**Table 14: Summary of Poisson's Ratio**

| Unit | Value/Comment |
|---|---|
| Overburden (Frozen) | 0.25 – 0.4 (note: Poisson's ratio for ice ~0.33)[1] |
| Overburden (Unfrozen) | 0.2 – 0.4 (SM) |
| Engineered Fill (Manufactured Fines) | 0.2 to 0.4 (Dependant on degree of compaction) |
| Engineered Fills (Crushed Rock) | 0.15 to 0.35 (Dependant on degree of compaction) |

Notes:
1. Based on engineering judgement and book values (Aysen 2005; Stanford University 2010; Gere *et al*. 1997; Iowa Statewide Urban Design and Specifications 2009).

**Table 15: Summary of Select Elastic Moduli (Young's Modulus or E)[2]**

| Unit | Value/Comment |
|---|---|
| Overburden (Frozen) | 70 to 150 MPa |
| Overburden (Unfrozen) | 10 to 50 MPa |
| Engineered Fills (Manufactured Fines) | 35 to 100 MPa (Dependant on degree of compaction) |
| Engineered Fills > 1 m (Crushed Rock) | 70 to 175 MPa (Dependant on degree of compaction) |
| Bedrock[1] | 13.2 GPa (average) |

Notes:
1. SRK (2012b).
2. Based on engineering judgement and book values. (Aysen 2005; Stanford University 2010; Gere *et al*. 1997; Iowa Statewide Urban Design and Specifications 2009).

## 5.5 Typical Thermal Properties

Permafrost characterization has been completed at the Goose Property by Rescan (2014) and SRK (2011b). The Property is located within a zone of continuous permafrost with the base of permafrost estimated about 400 mbgs. A seasonally thawed active layer, from mid-June to late September reaches a depth of 1.3 to 4.2 m and the average permafrost temperature within the active layer is -7$^{o}$C. Ground temperature measurements indicate unfrozen taliks are present beneath local lakes (e.g. Llama Lake). Rescan (2014) reports a geothermal gradient of 0.013 to 0.014 °C/m based on data from across the Property.

Appendix B presents thermal modelling that was carried out for the Property to determine the appropriate thermal cladding layer required to prevent permafrost thaw, both for heated and unheated structures. The thermal modelling was performed for a design life of 20 years.

Table 16 summarizes the material thermal properties used in the analysis. Although the thermal analysis, presented in Appendix B, attempts to account for heated buildings, there is really no viable maximum pad thickness to prevent permafrost thaw. Viable solutions to prevent permafrost thaw include: (1) raising the structure off the ground surface using piles; (2) placing thermosyphons beneath the structure; (3) placing ducts in the pads beneath the building to provide ventilation. If using Ad-freeze piles to increase foundation strength, site specific thermal analysis will need to be carried out to identify the effect on the active layer.

**Table 16: Typical Thermal Material Properties**

| Material | Degree of Saturation (%) | Porosity | Thermal conductivity (kJ $m^{-1}$ $day^{-1}$ $°C^{-1}$) | | Volumetric Heat Capacity (kJ $m^{-3}$ $°C^{-1}$) | |
|---|---|---|---|---|---|---|
| | | | Unfrozen | Frozen | Unfrozen | Frozen |
| Run-of-Quarry Material | 30 | 0.30 | 104 | 117 | 1,697 | 1,509 |
| Polystyrene Insulation | 0 | - | 3 | 3 | 38 | 38 |
| Peat | 100 | 0.65 | 48 | 138 | 2,600 | 2,200 |
| Sand (@ 23 ppt salinity) | 67 | 0.36 | 143 | 182 | 2,242 | 1,726 |

## 5.6 Lateral Earth Pressures

Retaining walls and bridge foundations can be protected against horizontal and vertical frost action forces with the use of non-frost susceptible backfill or insulation. The use of granular fill will limit frost heave forces and ensure good drainage and a lower groundwater level (Andersland and Ladanyi 2004). Note that frozen ground conditions (temperatures in the active zone below 0˚C) are typically required or suggested for bridge and retaining wall foundations (at the time of construction), as temperatures above 0˚C in the active layer can increase the potential for slope failure and lead to soil dewatering issues.

It is recommended that retaining walls be restricted to engineered fills alone. If constructing retaining walls in overburden, site specific characterization and design should be completed.

**Table 17: Summary of Select Lateral Earth Pressure Parameters**

| Element | Unit | Value/Comment |
|---|---|---|
| Rankine Passive (Kp) Soil Pressure Coefficient[1] | Engineered Fills > 1 m | 4.2 |
| Rankine Active (Ka) Soil Pressure Coefficient[2] | Engineered Fills > 1 m | 0.24 |
| Coefficient of at Rest Earth/Soil Pressures (Ko)[3] | Engineered Fills > 1 m | 0.38 |
| Angle of Repose[4] | Engineer Fill (Run-of-Quarry) | 1:1 to 1:1.3 H:V |
| Allowable Slopes | Engineered Fills (Run-of-Quarry) 1 m to 2 m | 1.5H:1V |
| | Engineered Fills (Run-of-Quarry) >2 m | 2H:1V |
| Coefficient of Friction | Between Concrete Wall and Run-of-Quarry Material | 25 to 27° (based on Run-of-Quarry friction angle and Coulomb equation) |

Notes:
1. Kp = (1+sinϕ')/ (1-sinϕ'); from Rakine's theory of active and passive soil pressures. Assumes/ requires a long smooth wall and assumes a linear distribution of lateral pressure.
2. Ka = (1-sinϕ')/ (1+sinϕ'); from Rakine's theory of active and passive soil pressures. Assumes/ requires a long smooth wall and assumes a linear distribution of lateral pressure.
3. Ko= 1-sinϕ'; for truly normally consolidated soils that exhibit zero cohesion during drained shear, based on Jaky empirical equation (simplification to be reassessed on a site by site case).
4. Based on engineering judgement for typical low height waste rock slopes.

Note that in Table 17 above, Rankin passive and active earth/soil pressures are outlined to give a starting/reference point for expected values. Rankin's theory assumes a long and smooth vertical wall where the lateral pressures increases linearly with depth. Should alternate conditions be imposed on the structure, other more appropriate analysis must be completed (Aysen 2005).

This report, "Back River Property Geotechnical Design Parameters", was prepared by SRK Consulting (Canada) Inc.



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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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