Perimeter Dam Stability Evaluations

Dam 1, Dam 2, and Dam 5

Prepared for
Northshore Mining Company

Prepared by
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Certification

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of Minnesota.

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1.1 Introduction

This report presents the seepage and slope stability analyses performed by Barr Engineering Co. (Barr) for the raise of Dam 1, Dam 2, and Dam 5 at the Northshore Mining Company (NSM) Milepost 7 (MP7) tailings storage facility (TSF) in Silver Bay, Minnesota. Barr has completed analyses for the raise of Dam 1 to elevation 1,265 feet, Dam 2 to elevation 1,270 feet, and Dam 5 to elevation 1,270 feet, to assess the seepage and slope stability characteristics.

1.1.1 Dam 1 Background

Dam 1 is located at the southern end of the MP7 TSF as depicted in Figure 1. Dam 1 is currently approximately 12,000 feet long, and the crest of the dam is approximately 700 to 800 feet wide. Dam 1 consists of three general zones, the main segment, Dam 1 east (Dam 1E) and Dam 1 western progression (Dam 1W). Dam 1 and Dam 1E were two separate dams prior to 2004, with Dam 1E constructed on the east side of a high point, the previous eastern abutment for the main segment of Dam 1. Dam 1E was located east of the highpoint and west of another natural high ground area. As the crest is wide in this area, the dams share the same alignment, although it changes direction at the location of the underlying natural high ground point between the two dams. These two segments are typically collectively considered Dam 1. As the basin extends westward with increasing pond levels, an extension of Dam 1 to the west is required (D1W). Dam 1W will extend around the east and north side of the existing landfill and then continue to the north to tie into the topography at the maximum permitted dam crest elevation of 1,315 feet. Initial construction began for this extension in 2023.

The general stratigraphy beneath the highest areas of the dam consists of approximately 10 to 20 feet thick lacustrine clay deposits overlying glacial till of varying thicknesses and then bedrock. The lacustrine clay deposits are present in former river channel zones near the approximate center of the main dam segment and the general central portion of Dam 1E. The dam is supported directly by glacial till outside these former river channel zones. Glacial outwash is also present in some areas along the dam toe.

Initial construction of the main dam and Dam 1E segments consisted of a sand and gravel starter dam with an upstream clay face. The original intent was to raise the dam using downstream construction methods. However, as a result of closure activities in the 1980s, regulators required operation and construction under the previous Consensus Closure Plan when the plant restarted in the 1990s and during operation until 1996. This arrangement required placement of plant aggregate and filter material over the fine tailings beach out to a distance approximately 1,400 feet upstream of the starter dam, effectively implementing a one-time, modified centerline configuration and eliminating a significant portion the southern storage cell as required by the closure plan. In about 2003, dam construction continued following the modified centerline method, including a filter berm with plant aggregate backing material to create the dam body and fine tailings discharged inside the basin from the shoulder of the filter berm, creating a beach. The area downslope of the filter berm is constructed with plant aggregate and filter tailings overlying fine tailings. Fine tailings previously deposited by pipeline from near the original starter dam are present from near the old dam crest and extend into the basin. The plant aggregate zones are generally approximately 50 feet to 60 ft thick, and the fine-fraction tailings are generally approximately 40 feet to 55 feet thick. Initial Dam 1W construction consists of a glacial till foundation, plant aggregate body, and filter berm adjacent to fine tailings.

1.1.2 Dam 2 Background

A plan view showing the location of Dam 2 is included in Figure 1. Dam 2 is located at the northern end of the MP7 tailings basin and is currently approximately 7,100 feet long. Dam 2 consists of two zones, the main dam segment and the western progression (Dam 2W). Construction of Dam 2W commenced in 2022 and will continue westward to tie into topography at the maximum permitted dam crest elevation of 1,315 feet.

The main segment of Dam 2 was initially constructed as a sand and gravel starter dam with a glacial till cutoff incorporated as the central core in the starter dam. The starter dam was constructed on natural soils consisting of approximately 10 to 20 feet of lacustrine clay overlying glacial till soils. The fill material placed on natural ground to the existing dam elevation consists of plant aggregate, which extends upstream of the starter dam about 500 to 600 feet. After completion of the plant aggregate placement, fine tailings were discharged into the basin creating beaches.

Dam 2 was originally planned to be raised using downstream construction methods, similar to Dam 1, but following plant restart until about 1996, plant aggregate and filters were placed over the beach into the basin distance approximately 1,400 feet per the previous Consensus Closure Plan. The modified centerline method used for Dam 1 was also used for Dam 2 beginning in about 1997 and continuing in 2003, with a filter berm constructed approximately 800 feet upstream of the starter dam. The area downslope of the filter berm is raised using only plant aggregate. Future raises are also presently planned to continue to use the modified centerline method, and the downstream slope for Dam 2 (above an approximate elevation of 1,200 feet) will continue to be 6H:1V up to the maximum permitted crest elevation of 1,315 feet.

A peat deposit overlying the lacustrine clay and glacial till is present in the approximate middle portion of the dam cross section. The peat has been compressed from its original 10-foot thickness to about 3 to 5 feet thick. Previous investigations identify an alluvial channel cut into the underlying glacial till in the center of the dam site near the middle of the dam cross section. A berm consisting of plant aggregate was constructed in 1997 along the downstream toe of Dam 2 in the area of the lowest natural ground and where the dam section will be highest. The purpose of the toe berm was to increase the dam's stability by providing a means for drainage of seepage and additional weight along the toe of the dam. The western extension of Dam 2 is founded on glacial till soils and will also consist of a plant aggregate body with an upstream filter berm adjacent to fine tailings.

A seepage cutoff was initially constructed in the northeastern corner of Dam 2 in May 2012. The seepage cutoff was constructed beyond the eastern extent of the original clay cutoff to reduce the potential for a significant amount of seepage to flow along the hillside, through more permeable plant aggregate zones located in this area of the dam. The first stage consisted of a soil-cement-bentonite cutoff to elevation 1,216 feet, with the subsequent stages consisting of compacted clay till to a present surface elevation of approximately 1,248 feet. The cutoff will be extended vertically as part of subsequent raises to the maximum permitted dam height.

1.1.3 Dam 5 Background

Dam 5 is located on the eastern side of the MP7 tailings basin as depicted in Figure 1. Dam 5 was originally constructed as two dams, Dam 5A and Dam 5B, and the dams were joined as they were raised. The dam is constructed over a layer of clay at the south end, a rock knob in the middle, and a rock foundation on the north end. Blanket grouting was performed to improve the northern rock foundation, while the middle rock knob was covered by filter tailings as the dams were raised. A central cutoff was used in the initial design and has continued to be incorporated into recent raises. A buttress was initially added along the toe of the dam during the raise to crest elevation of 1,235 feet and is extended as needed within the future planned footprint to provide additional weight along the toe of the dam with subsequent raises. Historically, fine tailings have not been present against the upstream slope of Dam 5, and although the basin elevation and configuration now allow some fine tailings adjacent to the upstream slope, Dam 5 is not constructed over fine tailings. Dam 5 is planned to continue to be constructed with a central cutoff to the maximum permitted crest elevation of 1,315 feet. As the basin continues to rise, the current north end of Dam 5 will be extended further north to tie into topography at 1,315 feet.

1.1.4 Previous Investigations

Extensive subsurface investigation and laboratory testing has been performed since the 1970s at MP7. Numerous test borings and laboratory tests were performed by Klohn-Leonoff Consultants Ltd. (Klohn) of Calgary, Alberta between 1974 and 1987. The subsequent consultant, Sitka Corp. (Sitka), performed test borings, field testing, instrumentation installations, and cone penetration test (CPT) soundings in 1996 and 1997. Barr has since performed several geotechnical investigations, in some cases with field and laboratory testing.

2 Seepage and Stability Analysis Methods

2.1 Analysis Software

GeoStudio, a seepage and slope stability modeling software produced by Seequent, was used to analyze seepage and stability for the raises of Dam 1, Dam 2, and Dam 5. The SEEP/W, SLOPE/W, and SIGMA/W modules of the software package were used for these analyses. SEEP/W uses finite element methods (FEM) to model water seepage through porous material (soil and rock). SIGMA/W uses FEM to model stress-strain behavior and was used for development of end-of-construction (EOC) strength parameters. SLOPE/W models slope stability using limit equilibrium methods and was utilized to determine the factors of safety (FOS) for the downstream slopes. Spencer's Method was used to calculate the factor of safety for the slope stability analyses. This method is considered adequate because it satisfies conditions of static equilibrium and provides a factor of safety based on both force and moment equilibrium.

2.2 Model Geometry

2.2.1 Dam 1 Geometry

The geometry of the existing dam surface was based on the latest available LiDAR data at the time of the analysis, while the proposed dam raise geometry for a crest elevation of 1,265 feet is based on a continued downstream slope of 6H:1V and an upstream filter berm side slope of 2H:1V. The stratigraphy configuration of the models incorporated the results of previous test borings and CPT soundings performed within or adjacent to the dam footprint, as well as records of construction. The cross-section geometry of the western end of the dam is configured in a similar manner without the starter dam, including a plant aggregate body with an upstream filter berm adjacent to the fine tailings. The southwestern corner of Dam 1 incorporates a downstream slope of 6H:1V, transitioning to a downstream slope of 4H:1V further to the north where the dam will be shorter and founded on glacial till. The upstream filter berm slope is 2H:1V.

Two cross sections (identified as 28+40 and 35+00 have historically been identified as critical areas for the design of Dam 1. These cross sections are near the middle portion of the dam, also identified as the lowest natural ground foundation area where soft lacustrine clay deposits are present. For the current analyses, cross-sections were also reviewed at station 90+50 at the east end of Dam 1 as well as at 0+00, -10+00, and -17+00 along the western progression of Dam 1. Figures 2 through 7 depict the stratigraphy incorporated into the models for each cross section.

The models include a plant aggregate layer upstream of the dam with fine tailings present above and below. This layer is approximately 10 to 15 feet thick and is representative of plant aggregate placed over the fine tailings in the mid-1990s as part of the closure plan.

2.2.2 Dam 2 Geometry

The surface geometry for Dam 2 is based on the latest available LiDAR data at the time of the analysis, with the proposed dam raise geometry for a crest elevation of 1,270 feet based upon a continued downstream slope of 6H:1V and an upstream filter berm side slope of 2H:1V. Stratigraphy in the models was developed based on the results of previous test borings and CPT soundings performed within and adjacent to the dam in addition to records of construction.

Historically, two cross sections locations identified as 34+75 and 42+00 have been identified as critical areas for Dam 2 design. These cross sections are near the middle portion of the dam in the area of the highest potential dam raise which also corresponds to the lowest natural ground foundation areas where soft lacustrine clay and peat deposits are present. A sand seam layer has been observed at several investigation locations beneath the mid-slope at station 42+00. Conservatively, the sand seam was included in the model as a continuous layer, although this may not be the actual field conditions. Therefore, models have been developed with and without the sand seam for sensitivity analyses to evaluate the potential impact on slope stability. As described for Dam 1, plant aggregate was placed over the fine tailings in the mid-1990s as part of the closure plan, and this layer is included within the fine tailings in the models. Stratigraphy incorporated into the models for each cross section are depicted in Figures 8 and 9.

2.2.3 Dam 5 Geometry

The geometry of the existing Dam 5 dam surface was based upon the latest available LiDAR data at the time of the analysis, and the proposed dam raise geometry for a dam crest elevation of 1,270 feet is based on a continued downstream slope of 6H:1V with an upstream slope of 2.5H:1V. The stratigraphy configuration of the models incorporated the results of previous test borings performed within or adjacent to the dam footprint, as well as records of construction.

Historically, the critical cross section for Dam 5 has been located along station 14+00, which corresponds to the location of the highest area of the dam where the downstream slope has been constructed over lacustrine clay soils. Previous configurations of Dam 5 have included several construction access benches that still fall within an overall downstream slope of 6H:1V. The downstream slope for the proposed configuration for the crest at elevation 1,270 feet was revised to reflect removal of these benches in the interest of reducing the potential for long term erosion. Any remaining small benches allowing access of passenger vehicles to equipment and the dam toe remain within the overall 6H:1V slope. It was assumed an equipment access bench will be present over the dam toe and extend above the toe seepage pond water elevation, which also serves as a buttress extension. Stratigraphy included in the models for Dam 5 is shown in Figure 10.

2.3 Dam Loading Conditions

The perimeter dams raises were modeled for up to four material strength scenarios including three types of undrained strength stability analyses (USSA) and one drained or effective stress stability analysis (ESSA), which will be discussed in the following sections. Since it is common for potential failure surfaces to develop between lacustrine clay and the underlying glacial till soils, scenarios where the till layer is considered impenetrable to force a block failure were evaluated, as well.

2.3.1 Drained Conditions (ESSA)

If shear stress is applied to a soil at such a rate and/or the drainage conditions are such that excess pore water pressure is zero when failure occurs, failure is said to occur under drained conditions, where the effective stress shear strength of the soil has been mobilized. This case is typically applied to long-term, steady-state conditions, where excess pore water pressures generated due to loading have dissipated. The drained condition also applies to granular materials for short-term conditions when such materials have a high enough permeability that any excess pore water pressure is nearly immediately dissipated. The drained strength is most often described in terms of a strength envelope. The failure envelope may be linear, using the Mohr-Coulomb model to provide a drained friction angle (ϕ') and effective cohesion intercept, or it may be represented as a non-linear failure envelope.

2.3.2 Undrained Conditions (USSA)

If shear stress is applied to a soil quickly and/or if the drainage conditions are such that no shear-induced porewater pressure can dissipate when failure occurs, failure is said to occur in an undrained condition or the undrained shear strength has been mobilized. The undrained shear strength is typically applied to short-term conditions for saturated soils, for example, during or immediately after construction when construction proceeds at a fast enough rate that excess pore water pressures develop. The following sections present different USSA analyses that were performed for the perimeter dams.

2.3.2.1 Undrained Conditions – Yield Strength

It has been observed in soft soils that the undrained shear strength is often a function of effective stress. When the undrained shear strength increases linearly with increasing effective stress, the Undrained Shear Strength Ratio (USSR) is generally preferred to model the material strength. The USSR is defined as the undrained shear strength, s_u, divided by the initial effective vertical stress, σ'_{vo}. For the undrained analyses in which a USSR is applied, shear strength was modeled as a function of the overburden, hence the strength of the material increases linearly with respect to overburden (or effective stress). Shear strength represented by a USSR is applicable to the fine tailings, normally consolidated (NC) lacustrine clay, and clayey glacial till materials.

In the case of the fine tailings, the nomenclature for the USSR is further modified to $USSR_{yield}$, which refers the yield strength of the material as presented in *Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments* (Olson and Stark, 2003). For the clayey glacial till and lacustrine clays, this USSR relates to the peak strength. In the analysis presented, these strengths were used to represent the condition called End-of-Primary (EOP), which is the point in time where primary consolidation has been achieved due to load placement.

2.3.2.2 Undrained Conditions – Liquefied Strength

It is anticipated that most of the time, loading or change in loading within the fine tailings at MP7 will be slow enough for the fine tailings to be sheared under drained conditions. However, there are circumstances during which rapid changes in load and/or local stress may occur that can lead to undrained loading with liquefied conditions. Examples include rapid fill placement, significant erosion, rapid water level changes, or a seismic event that changes the state of stress within the fine tailings over a short time period. Creep can also trigger static liquefaction.

The state of a soil dictates its response to undrained loading. If the soil is in a compacted or dense state, it will exhibit dilative behavior and the particles will have to roll over each other, thereby increasing the volume of the soil mass when sheared. If drainage is not permitted, negative pore water pressures will develop. A contractive soil is in a loose state, and when loaded and sheared, the material will compress and become more compact, decreasing the volume of the soil mass. If drainage is not permitted, positive pore water pressures will develop. Contractive soils are susceptible to flow liquefaction.

Dilative materials at MP7, including glacial till, clay core, lacustrine clay, plant aggregate, and filter material are not considered subject to liquefaction, so they were not evaluated for liquefied shear strength. Liquefaction has been observed in saturated mine tailings, such as those found within the basin, which are hydraulically deposited and often exhibit contractive response (Castro, Walberg, and Perlea, 2003).

Since an unknown or combination of triggers could cause static liquefaction of the fine tailings at the site, stability analyses including liquefied fine tailings were evaluated for the raise of Dam 1 and Dam 2, as will be discussed in later sections. Liquefied fine tailings are not included in the Dam 5 analysis since no fine tailings are beneath the dam footprint and minimal fine tailings are upstream of the dam.

2.3.2.3 Undrained Conditions – Operational or EOC Conditions

As discussed in Section 2.3.2.1, the undrained shear strength of soft soils increases with increased effective stress and can be modeled with a USSR. During the Perimeter Dam raises, it is assumed that the fine tailings and clay (materials that can potentially consolidate and gain strength) will not have the time for strength gain during or immediately after construction of the raise. Therefore, the vertical effective stresses for the fine tailings and clay before the raise is constructed (current operational conditions) were estimated using the SIGMA/W module of GeoStudio and the undrained shear strengths (both yield and liquefied) corresponding to the current effective stresses were applied to each material. It is assumed that under these conditions, the increased pore-pressures have not yet dissipated. This case is typically referred to as the ”end-of-construction” (EOC) case; however, since construction at the basin consists of intermittent time periods of dam raises that will not end until the basin reaches the ultimate elevation, this case is considered operational conditions. For purposes of this report, operational and EOC are used interchangeably.

2.3.3 Railroad

To incorporate additional conservatism, a train load was applied to the middle of the railroad embankment constructed as part of the dam raises at the perimeter dams. Assuming the Cooper E-80 loading configuration, the surcharge load of 1,300 psf was applied over an 8.5-foot-long railroad tie.

3 Material Parameters

3.1 Strength Parameters

Table 3-1 presents the strength parameters used for the materials included in the slope stability models. These parameters have been developed over time incorporating data from test borings, CPT soundings, field testing, and laboratory testing by Barr and previous consultants. Materials for select scenarios are discussed in the sections below.

Table 3-1 Soil Strength Parameters

Soil Type	Moist Unit Weight (pcf)	(Φ' degrees)	Undrained (USSA)		
			USSR[1]	Mohr-Coulomb	
			s_u/σ'_v	Undrained Shear Strength (psf)	Friction Angle (deg)
Foundation Till	145	28	0.29	--	--
Plant Aggregate	144	15th percentile non-linear function [2]			
Sloughed Plant Aggregate	144	35	--	0	35
Clay Core	145	31	--	0	31
Filter Material	130	38	--	0	38
Normally Consolidated Lacustrine Clay	113	33rd percentile non-linear function[3]	0.21[4]	--	0
Overconsolidated Lacustrine Clay	113	33rd percentile non-linear function [3]	--	Dam 1: 1,660[4] Dam 2: 680[5]	0
Fine Tailings - Drained Strength	130	24	--	--	--
Fine Tailings - Yield Strength	130	--	0.24[6]	--	--
Fine Tailings - Liquefied Strength	130	--	Dam 1: 0.09[7] Dam 2: 0.10[8]	--	--

[1] Undrained Shear Strength Ratio
[2] Refer to Figure 11
[3] Refer to Figure 12
[4] Refer to Figure 13
[5] Refer to Figure 14
[6] Based on 33rd percentile of fine tailings CPT and triaxial data
[7] Based on 33rd percentile of fine tailings CPT, triaxial, and FVST test data
[8] Based on 33rd percentile of fine tailings CPT and triaxial test data

3.1.1 End of Construction Undrained Shear Strengths

The process to derive the EOC shear strength functions for the clays and fine tailings functions utilizes the SIGMA/W module of Geostudio. SIGMA/W uses finite element methods to model stress-strain behavior and was used to calculate effective stresses under existing conditions. The vertical effective stresses within the clays and fine tailings were calculated relative to node locations within the finite element geometry mesh. Stresses were determined for conditions considered "existing" for purposes and at the time of the analyses and each section. The existing conditions include crest and upstream pond elevations for each dam corresponding to the conditions prior to placement of fill to reach the next crest elevation. The vertical effective stresses were multiplied by the USSR for the clays and the $USSR_{yield}$ for the fine tailings to calculate the associated undrained shear strength for each node coordinate. The node coordinates and associated undrained shear strengths were then input as a spatial function into SLOPE/W for the EOC cases.

3.2 Seepage Parameters

Seepage parameters incorporated into seepage models have been developed based on results of field and laboratory testing data collected over the life of the basin to date. Additionally, seepage parameters for each material from previous analyses were adjusted through calibration with observed field conditions from instrumentation as part of seepage modeling for each dam raise. The calibration process includes assembling a finite element model using the SEEP/W module of GeoStudio for each cross section for steady state flow conditions with a fixed upstream pond and downstream conditions for the existing dam at the time of analyses. Resulting porewater pressure conditions from the model were then compared to observed conditions at instrumentation locations. Where total heads are inconsistent between the model and instrumentation measurements, adjustments are made to material permeability parameters in the model and results are subsequently compared again. This process is repeated until general, yet conservative, agreement between total heads is achieved for each material; as variations in actual field conditions of flow regime, stratigraphy, and material properties are present, complete agreement between the modeled and measured total heads is not realistic. Permeability values are also reviewed for consistency with a generally anticipated range for the material type. The calibration and material permeability values for each perimeter dam and cross section are discussed in subsequent sections.

3.2.1 Dam 1 Calibrations

A comparison between modeled and measured total heads was evaluated at the location of 25, 27, and 21 locations of instruments along cross sections at stations 28+40, 35+00, and 90+50, respectively. The instruments in these areas mainly consist of vibrating wire piezometers, though a few historical pneumatic piezometers from the original dam design are also present and were included in the calibration process. Calibrations for Dam 1 models at cross section locations 28+40 and 35+00 in the main segment of the dam were performed by considering the dam configuration, upstream main pond and downstream seepage recovery pond 1A water surface elevations, and piezometer data from the fall of 2022. For the cross section at 90+50 on the east end of the dam the configuration, upstream main pond and downstream seepage recovery pond 1B conditions, and piezometer data were from the fall of 2023. These comparisons were only used for model calibration.

In general, resulting modeled total heads for the calibrated permeability values were about 1 to 8 feet higher in dam fill materials (plant aggregate, filter, cutoff). Within the fine tailings beneath the offset plant aggregate zone, the model total heads were generally 1 to 4 feet higher that measured. Model total heads in the upstream fine tailings are 5 to 20 feet higher than measured; this variation is likely due to the significant difference in permeability between the fine tailings and filter/plant aggregate materials in this area. In the till and lacustrine clay foundation soils, the modeled total head was generally 2 to 8 feet higher, though in some locations beneath the dam crest the total head is about 15 to 25 feet higher. At a few locations, the modeled total head is lower than the measured. The difference at all the locations except one is less than 3 feet. For one instrument in the fine tailings, the difference is 12.7 feet; this is likely attributed to discontinuities in the fine tailings.

The majority of model total head values are greater than measured values and are considered acceptable and conservative for purposes of modeling the next dam raise for Dam 1. The resulting permeability values from the seepage calibration for stations 28+40, 35+00, and 90+50 are presented in Table 3-22. As the dam will be relatively short when the 1,265 ft crest is reached and no instruments are present yet along the western progression of Dam 1, permeabilities for the materials at the west end of Dam 1 are the same as for Station 28+40.

Table 3-2 Dam 1 Permeability Values from Seepage Calibration

Material	Station 28+40		Station 35+00		Station 90+50	
	Resulting Horizontal Permeability from Calibration (ft/sec)	Permeability Anisotropy (k_H/k_V)	Resulting Horizontal Permeability from Calibration (ft/sec)	Permeability Anisotropy (k_H/k_V)	Resulting Horizontal Permeability from Calibration (ft/sec)	Permeability Anisotropy (k_H/k_V)
Clay Cutoff	$3.28x10^{-9}$	1	$3.28x10^{-9}$	1	$1.25x10^{-7}$	1
Filter Material	$9.00x10^{-5}$	1	$3.40x10^{-4}$	1	$6.56x10^{-5}$	1
Fine Tailings	$5.00x10^{-5}$	1	$1.80x10^{-6}$	1	$1.31x10^{-6}$	1
Foundation Till	$6.00x10^{-6}$	5	$6.00x10^{-6}$	5	$5.00x10^{-5}$	9
NC Lacustrine Clay Downstream(1)	$5.45x10^{-8}$	0.7	$2.45x10^{-8}$	0.7	$6.62x10^{-7}$	0.9
NC Lacustrine Clay Upstream(1)	$1.41x10^{-8}$	0.4	$7.50x10^{-7}$	0.4	N/A (2)	N/A (2)
OC Lacustrine Clay(1)	$1.41x10^{-8}$	55.6	$3.71x10^{-6}$	55.6	$7.74x10^{-7}$	0.015
Plant Aggregate	$2.62x10^{-3}$	1	$1.62x10^{-3}$	1	$2.62x10^{-3}$	1
Sand and Gravel	$8.20x10^{-6}$	1	$3.30x10^{-3}$	1	N/A (3)	N/A (3)
Select Sand and Gravel	$8.20x10^{-6}$	1	$1.67x10^{-3}$	1	N/A (3)	N/A (3)
Bedrock	N/A (4)	N/A (4)	N/A (4)	N/A (4)	$3.00x10^{-10}$	1

Notes:
(1)NC is normally consolidated clay and OC is overconsolidated clay
(2)The permeability for NC lacustrine clay is not differentiated between upstream and downstream at Station 90+50
(3)Sand and gravel and select sand and gravel are not present at Station 90+50
(4)Bedrock is not included in models for Stations 28+40 and 35+00

3.2.2 Dam 2 Calibration

For Dam 2, modeled total heads were compared to measured total heads at the location of 19 and 21 instruments along cross sections at stations 34+75 and 42+00 respectively. The instruments in these areas mainly consist of vibrating wire piezometers, though a few pneumatic piezometers are also present and were included in the calibration process. Calibrations for Dam 2 models were performed by considering the dam configuration, upstream main pond surface elevation, and piezometer data from the fall of 2020.

Resulting model total heads using the calibrated permeability values are generally 1 to 5 feet greater than measured at piezometer locations within dam fill materials and fine tailings. In the dam foundation materials, resulting model total heads are also generally 1 to 5 feet higher, though the modeled head is 5 to 12 feet higher at a few select locations.

For purposes of this phase of modeling for the Dam 2 crest elevation raise to 1,270 feet, the model head values are considered acceptable. The resulting permeability values estimated from the seepage calibration for both station 34+75 and 42+00 are presented in Table 3-3.

Table 3-3 Dam 2 Permeability Values from Seepage Calibration

Material	Station 34+75		Station 42+00	
	Resulting Horizontal Permeability from Calibration (ft/sec)	Permeability Anisotropy (k_H/k_V)	Resulting Horizontal Permeability from Calibration (ft/sec)	Permeability Anisotropy (k_H/k_V)
Clay Cutoff	$1.25x10^{-8}$	1	$1.25x10^{-8}$	1
Filter Material	$6.56x10^{-5}$	1	$6.56x10^{-5}$	1
Fine Tailings	$7.31x10^{-7}$	1	$8.31x10^{-7}$	1
Foundation Till	$3.99x10^{-6}$	9	$6.62x10^{-6}$	9
NC Lacustrine Clay(1)	$1x10^{-8}$	0.9	$2.62x10^{-9}$	0.9
OC Lacustrine Clay(1)	$1x10^{-5}$	66.7	$4.74x10^{-7}$	66.7
Peat	$2.35x10^{-8}$	1	$3.00x10^{-10}$	1
Plant Aggregate	$2.62x10^{-3}$	1	$2.62x10^{-3}$	1
Sand Seam	N/A(2)	1	$5.00x10^{-4}$	1
Bedrock	$3.00x10^{-10}$	1	$3.00x10^{-10}$	1

Notes:
(1)NC is normally consolidated clay and OC is overconsolidated clay
(2)A significant sand seam has not been observed at the toe of Dam 2 at station 34+75

3.2.3 Dam 5 Calibration

At Dam 5, measured and modeled total heads were compared at 9 vibrating wire piezometer instrument locations along the cross section at station 14+00. The modeled heads for instruments on the upstream side of the crest are minimal to approximately 2.5 feet higher than measured total heads. At the toe, the modeled heads are 0.3 feet higher than measured. For one instrument, the modeled head is the same as measured, while for three of the four instruments on the downstream side of the crest, modeled total heads are approximately 5 to 12 feet lower than measured. These instruments are co-located with a damaged inclinometer and will be investigated as part of the inclinometer replacement.

For purposes of this phase of modeling for the Dam 5 crest elevation raise to 1,270 feet, the model head values are considered acceptable, as the excess pressures downstream of the till cutoff are a temporary condition. The resulting permeability values estimated from the seepage calibration for station 14+00 are presented in Table 3-4. For this location, it has been observed in the past that the permeability of the till soils near the midpoint of the cross section vary with depth. To capture this variation, foundation till with three different permeabilities has been incorporated into the model.

Table 3-4 Dam 5 Permeability Values from Seepage Calibration

Material[1]	Resulting Horizontal Permeability from Calibration (ft/sec)	Permeability Anisotropy (k_V/k_H)
Clay Cutoff	$2.00E^{-6}$	1.0
Filter Material	$6.56E^{-5}$	1.0
Fine Tailings	$1.50E\text{-}^{7}$	1.0
Foundation Till	$4.50E^{-6}$	9
Foundation Till (lower permeability)	$1.41E^{-8}$	9
Foundation Till (higher permeability)	$9.00E^{-6}$	5.3
NC Lacustrine Clay	$1.41E^{-8}$	0.9
Plant Aggregate	$2.62E^{-3}$	1.0
Bedrock	$3.28E^{-5}$	1.0

[1]NC = normally consolidated clay

4 Dam Stability Analyses

4.1 Scenarios

Slope stability analyses were performed for the proposed raise of each dam by incorporating the results of seepage analyses with the slope stability modeling module SLOPE/W within the GeoStudio software. Analyses were performed for the downstream slope of Dam 1 and Dam 2 including the following material strength configurations:

- Drained (ESSA)
- Drained (ESSA) with a block failure
- Undrained (USSA) using USSR for yield fine tailings strength with and without block failure
- Undrained (USSA) using USSR for liquefied fine tailings strength with and without a block failure
- Undrained (USSA) using end of construction (EOC) strengths, including yield strength for the fine tailings, with and without block failure
- Undrained (USSA) using EOC strengths, including liquefied strength for the fine tailings, with and without block failure.

For Dam 5, fine tailings are not present within the dam footprint; hence, the stability model scenarios include:

- Drained (ESSA)
- Drained (ESSA) with a block failure
- Undrained (USSA) with and without block failure
- Undrained (USSA) using EOC strengths, with and without block failure
- Undrained (USSA) using EOC strengths, including liquefied strength for the fine tailings, with and without block failure.

The upstream tailings pond applied for each of these configurations for steady state seepage conditions are as follows:

- A maximum normal operating pool elevation of 1,255 feet for Dam 1 and 1,260 feet for Dam 2 and Dam 5 matching the minimum of 10 feet of freeboard relative to the respective dam crest for steady state seepage conditions.

- Flood pool elevation of 1,262 feet for Dam 1 and 1,267 feet for Dam 2 and Dam 5 matching the minimum of 3 feet of freeboard relative to the dam crest or steady state seepage conditions.

- Estimated pool elevation of 1,234.0 feet at the end of the construction of the dam for steady state seepage conditions.

4.2 Factors of Safety

The minimum required factor of safety USSA conditions at the MP7 site is 1.3. For ESSA conditions, a minimum required factor of safety is 1.5. The minimum required factor of safety is 1.1 for Dam 1 and Dam 2 scenarios including the liquefied strength of fine tailings.

4.3 Results

The results of slope stability analyses are summarized for each dam in the following sections.

4.3.1 Dam 1

The slope stability results for the scenarios considering the normal operating and flood pools for Station 28+40 are shown in Table 4-1. The results of the EOC scenario with the estimated upstream pond at 1,234.0 feet are shown in 4-2.

Table 4-1 Dam 1 Station 28+40 Stability Results

Slope Location and Material Configuration	Dam 1 Station 28+40 Factors of Safety for Crest Elevation 1,265 feet		Minimum Required FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	2.27	2.24	1.5
Downstream Slope, ESSA Block Failure	1.93	1.90	1.5
Downstream Slope, USSA, Fine Tailings Yield Strength	1.43	1.42	1.3
Downstream Slope, USSA, Fine Tailings Yield Strength, Block Failure	1.40	1.39	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	1.28	1.25	1.1
Downstream Slope, USSA, Fine Tailings Liquefied Strength, Block Failure	1.28	1.25	1.1

Table 4-2 Dam 1 Station 28+40 Stability Results for End of Construction Conditions

Material Scenario	Factor of Safety[(1)]		Minimum Required FOS
	USSA	USSA BF	
EOC with Fine Tailings Yield Strength	1.44	1.32	1.3

The slope stability results for the scenarios considering the normal operating and flood pools for Station 35+00 are shown in 4-3. The results of the EOC scenario with the estimated upstream pond at 1,234.0 feet are shown in Table 4-4.

Table 4-3 Dam 1 Station 35+00 Stability Results

Slope Location and Material Configuration	Dam 1 Station 35+00 Factors of Safety for Crest Elevation 1,265 feet		Minimum Required FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	2.61	2.48	1.5
Downstream Slope, ESSA Block Failure	2.19	2.06	1.5
Downstream Slope, USSA, Fine Tailings Yield Strength	1.58	1.50	1.3
Downstream Slope, USSA, Fine Tailings Yield Strength, Block Failure	1.62	1.52	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	1.51	1.43	1.1
Downstream Slope, USSA, Fine Tailings Liquefied Strength, Block Failure	1.49	1.41	1.1

Table 4-4 Dam 1 Station 35+00 Stability Results for End of Construction Conditions

Material Scenario	Factor of Safety[(1)]		Minimum Required FOS
	USSA	USSA BF	
EOC with Fine Tailings Yield Strength	1.58	1.58	1.3

The slope stability results for the scenarios considering the normal operating and flood pools for Station 90+50 are shown in Table 4-5. The results of the EOC scenario with the estimated upstream pond at 1,234.0 feet are shown in Table 4-6

Table 4-5 Dam 1 Station 90+50 Stability Results

Slope Location and Material Configuration	Dam 1 Station 90+50 Factors of Safety for Crest Elevation 1,265 feet		Minimum Required FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	2.97	2.86	1.5
Downstream Slope, ESSA Block Failure	3.24	3.12	1.5
Downstream Slope, USSA, Fine Tailings Yield Strength(4)	2.24	2.16	1.3
Downstream Slope, USSA, Fine Tailings Yield Strength, Block Failure	2.49	2.43	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	2.15	2.08	1.1
Downstream Slope, USSA, Fine Tailings Liquefied Strength, Block Failure	2.33	2.29	1.1

Table 4-6 Dam 1 Station 90+50 Stability Results for End of Construction Conditions

Material Scenario	Factor of Safety(1)		Minimum Required FOS
	USSA	USSA BF	
EOC with Fine Tailings Yield Strength	2.25	2.40	1.3

Analyses were also completed for three stations along the western progression of Dam 1 for a crest elevation of 1,265 feet. The stability results for these locations are summarized the tables below. Block failure scenarios were not reviewed for these sections since lacustrine clay is not present. EOC scenarios were also not reviewed for Stations -10+00 or -17+00 since these areas are founded completely on glacial till soils.

Table 4-7 Dam 1 Station 0+00 Stability Results

Slope Location and Material Configuration	Dam 1 Station 0+00 Factors of Safety for Crest Elevation 1,265 feet		Minimum Acceptable FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	2.95	2.91	1.5
Downstream Slope, ESSA Block Failure	3.01	2.97	1.5
Downstream Slope, USSA, Fine Tailings Yield Strength	2.18	2.15	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	2.18	2.15	1.1

Table 4-8 Dam 1 Station 0+00 Stability Results for End of Construction Conditions

Material Scenario	Factor of Safety		Minimum Required FOS
	USSA	USSA BF	
EOC with Fine Tailings Yield Strength	2.83	3.92	1.3

Table 4-9 Dam 1 Station -10+00 Stability Results

Slope Location and Material Configuration	Dam 1 Station -10+00 Factors of Safety for Crest Elevation 1,265 feet		Minimum Acceptable FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	5.28	3.67	1.5
Downstream Slope, ESSA Block Failure	5.96	4.43	1.5
Downstream Slope, USSA, Fine Tailings Yield Strength	4.21	2.86	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	4.21	2.86	1.1

Table 4-10 Dam 1 Station -17+00 Stability Results

Slope Location and Material Configuration	Dam 1 Station -17+00 Factors of Safety for Crest Elevation 1,265 feet		Minimum Acceptable FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	3.46	2.16	1.5
Downstream Slope, ESSA Block Failure	6.02	4.02	1.5
Downstream Slope, USSA, Fine Tailings Yield Strength	2.13	2.10	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	2.13	2.10	1.1

As shown in Table 4-1 through Table 4-10 the resulting stability factors of safety cross sections at stations along the main dam, east end of the dam, and western extension meet or exceed the corresponding minimum factor of safety for all of the ESSA and USSA scenarios evaluated and the dam is considered stable under the conditions that were analyzed. The EOC scenarios analyzed are considered conservative as it is anticipated construction of the Dam 1 raise will be performed in stages, hence, underlying materials will have some period of time to gain strength between interim dam raise stage elevations.

4.3.2 Dam 2

Slope stability results for the scenarios considering the normal operating and flood pools for stations 34+75 and 42+00 at Dam 2 are shown in Tables**Error! Reference source not found.** 4-11 and 4-13. The results of the EOC scenarios with the estimated upstream pond at 1,234.0 feet are shown in Tables 4-12 and 4-14.

Table 4-11 Dam 2 Station 34+75 Stability Results

Slope Location and Material Configuration	Dam 2 Station 34+75 Factors of Safety for Crest Elevation 1,270 feet		Minimum Required FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	2.79	2.78	1.5
Downstream Slope, ESSA Block Failure	2.86	2.86	1.5
Downstream Slope, USSA	1.76	1.76	1.3
Downstream Slope, USSA, Block Failure	1.73	1.73	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	1.76	1.76	1.1
Downstream Slope, USSA, Fine Tailings Liquefied Strength, Block Failure	1.73	1.73	1.1

Table 4-12 Dam 2 Station 34+75 Stability Results for End of Construction Conditions

Material Scenario	Factor of Safety(1)		Minimum Required FOS
	USSA	USSA BF	
Downstream Slope, EOC	1.74	1.72	1.3

Table 4-13 Dam 2 Station 42+00 Stability Results

Slope Location and Material Configuration	Dam 2 Station 42+00 Factors of Safety for Crest Elevation 1,270 feet		Minimum Required FOS
	Tailings Pond at 1,255 feet (10-foot freeboard)	Tailings Pond at 1,262 feet (3-foot freeboard)	
Downstream Slope, ESSA	2.95	3.03	1.5
Downstream Slope, ESSA Block Failure	3.22	3.32	1.5
Downstream Slope, USSA	1.85	1.87	1.3
Downstream Slope, USSA, Block Failure	1.78	1.82	1.3
Downstream Slope, USSA, Fine Tailings Liquefied Strength	1.85	1.87	1.1
Downstream Slope, USSA, Fine Tailings Liquefied Strength, Block Failure	1.78	1.82	1.1

Table 4-14 Dam 2 Station 34+75 Stability Results for End of Construction Conditions

Material Scenario	Factor of Safety[1]		Minimum Required FOS
	USSA	USSA BF	
Downstream Slope, EOC	1.87	1.82	1.3

For the cross-section locations reviewed along Dam 2, the resulting stability factors of safety meet or exceed the corresponding minimum required factor of safety for all of the ESSA and USSA scenarios evaluated. The dam is considered stable under the conditions that were analyzed. Since it is anticipated the dam raise will be constructed in stages, allowing some period of time for strength gain between stage elevations, the EOC scenarios are considered conservative.

4.3.3 Dam 5

Slope stability results for the scenarios considering the normal operating and flood pools for station 14+00 at Dam 5 are shown in Table 4-15. The results of the EOC scenarios with the estimated upstream pond at 1,224.8 feet are shown in Table 4-16

Table 4-15 Dam 5 Station 14+00 Stability Results

Slope Location and Material Configuration	Dam 5 Station 14+00 Factors of Safety for Crest Elevation 1,270 feet		Minimum Required FOS
	Tailings Pond at 1,260 feet (10-foot freeboard)	Tailings Pond at 1,267 feet (3-foot freeboard)	
Downstream Slope, ESSA	2.55	2.52	1.5
Downstream Slope, ESSA Block Failure	2.26	2.23	1.5
Downstream Slope, USSA	1.73	1.69	1.3
Downstream Slope, USSA, Block Failure	1.54	1.51	1.3

Table 4-16 Dam 5 Station 14+00 Stability Results for End of Construction Conditions

Material Scenario	Factor of Safety(1)		Minimum Required FOS
	USSA	USSA BF	
Downstream Slope, EOC	1.9	1.5	1.3

5 Dam Toe Uplift Analyses

5.1 Background

Lacustrine clay deposits have been encountered in subsurface investigations at the toe of all the dams in the areas of the design cross sections at the MP7 TSF. Generally, the stratigraphy in these areas consists of lacustrine clay overlying glacial till and bedrock. Plant aggregate and filter material is present above the lacustrine soils within the extents of the dam footprints. The glacial till composition varies from soils with a high fines content, such as lean clay and silt, to silty sand and poorly graded sand and/or gravel outwash soils. The lacustrine soils typically consist of lean to fat clay, and due to lower permeability relative to the underlying glacial till, act as a confining layer. If seepage pressures within the permeable layer are greater than the overburden pressure of the confining layer, a rupture may occur through the confining layer, compromising stability (USBR, 2014). Since total heads higher than grade at the dam toe can develop with this stratigraphy sequencing, uplift was reviewed at the toe of each dam.

5.2 Uplift Evaluation Methods

The Total Stress Method, an approach suggested by the US Bureau of Reclamation (USBR, 2014), is typically used to evaluate the uplift stability of dams and was chosen for the MP7 TSF. For the Total Stress Method, the stresses acting on the blanket are evaluated in terms of total stresses where the FOS is the ratio of the saturated unit weight of the confining layer to the hydrostatic pressure at the base of the confining layer. It should be noted that uplift calculations are based on idealized conditions and do not account for other significant factors such as soils strength (Pabst, et.al 2013). Neglecting soil strength, the computed factors of safety are considered conservative assessments. High seepage pressures must also exceed the strength and stiffness of the overlying clays which are not accounted for in the analyses. Furthermore, any excess pressures along the toe of the dam have been included in the seepage and subsequent stability analyses as discussed in previous sections. The FOS for uplift is defined as follows:

$$FOS = {}^{\gamma_t * t}/_{\gamma_w * \ h_p} \qquad \text{(USBR, 2014)}$$

where:

$\gamma_t = \textit{total unit weight of confining layer soil}$
$t = \textit{vertical thickness of confining layer}$
$\gamma_w = \textit{unit weight of water}$
$h_p = \textit{pressure head at base of confining layer}$

The pressure head at the base of the lacustrine clay was estimated from the VW piezometers installed at and/or near the respective toes of Dam 1 and Dam 2 in the critical cross section areas. The total head at the base of the lacustrine clay was taken to be equal to the value measured in the uppermost glacial till piezometer in each nest, neglecting any potential head loss in the formation that may result in a reduced pressure at the base of the confining layer. For Dam 5, the uplift pressures at the toe of the dam were estimated based on the porewater pressure results of the finite element SEEP/W model.

For pressures to lift or rupture the confining layer, the shear strength of the lacustrine clay in addition to the overburden pressures would need to be overcome. The uplift FOS calculations conservatively neglect the clay shear strength due to the number of unknowns, e.g., the size of a potential rupture area. For existing sites with sufficient piezometric information, the USBR (2014) considers a FOS of 1.5 appropriate to account for errors in piezometric information and variable stratigraphy and soil properties.

5.3 Uplift Evaluation Results

Calculated uplift FOSs for the piezometers at the toe of Dam 1 are summarized in Table 5-1 while uplift FOSs for piezometers at the toe of Dam 2 are summarized in Table 5-2.

Table 5-1 Calculated FOS Against Uplift at Dam 1 Toe

Instrument Location (1)	Calculated Uplift FOS at Base of Lacustrine Clay(2)
D1-2400R1200	2.0
D1-2550R1200	1.9
D1-2750R1200	1.9
D1-2840R1200	2.4
D1-3050R1200	2.8
D1-3300R1200(3)	2.0
D1-3500R1200	2.0
D1-3600R1200	2.0
D1-9050R975	1.9

(1) Instrument locations are denoted with the format of dam number, station, and distance downstream of the dam centerline.
(2) For data between January through October 2024
(3) Uplift FOS calculated for lean clay glacial till layer above silty sand glacial till.

Table 5-2 Calculated FOS Against Uplift at Dam 2 Toe

Instrument Location (1)	Calculated Uplift FOS at Base of Lacustrine Clay(2)
D2-3475R1000	2.1
D2-3800R1000	2.1
D2-4200R1000	1.5
D2-4500R1000	2.5
D2-5000R1000	1.5

(1) Instrument locations are denoted with the format of dam number, station, and distance downstream of the dam centerline.
(2) For data between January and December 2024

For Dam 5, uplift was computed at the dam toe at the location of the cross section at Station 14+00. The FOS at the base of the lacustrine clay at the toe of Dam 5 was calculated to be 1.7. Based on the uplift analyses, the minimum uplift FOS is met for Dam 1, Dam 2, and Dam 5.

6 Summary

Seepage and stability analyses were performed to evaluate the raise of Dam 1 to a crest elevation of 1,265 feet, Dam 2 to a crest elevation of 1,270 feet, and Dam 5 to a crest elevation of 1,270 feet. The results of the analyses indicate the stability FOSs are greater than the minimum required FOSs for the site of 1.5 for drained (ESSA) scenarios; 1.3 for undrained (USSA) scenarios, including the yield strength of fine tailings for Dam 1 and Dam 2; and 1.1 for undrained (USSA) scenarios including the liquefied strength of fine tailings for Dam 1 and Dam 2. Toe uplift was also evaluated for each dam. The FOSs against uplift meet the minimum of 1.5 based on guidance from USBR (2014).

7 References

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Pabst, Mark et. al. "Heave, Uplift, and Piping at the Toe of Embankment Dams." *The Journal of Dam Safety*, Association of State Dam Safety Officials (ASDSO). 2013, Vol. 11, Issue 2.

Olson, Scott M., and Timothy D. Stark. Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments. August 2003, Vol. 129, 8, pp. 727-737.

U.S. Department of Interior Bureau of Reclamation (USBR). *Design Standards No. 13 Embankment Dams*. January 2014.

Figures

FIGURE 1: DAM LOCATIONS

Figure
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 1, Station 28+40 Geometry

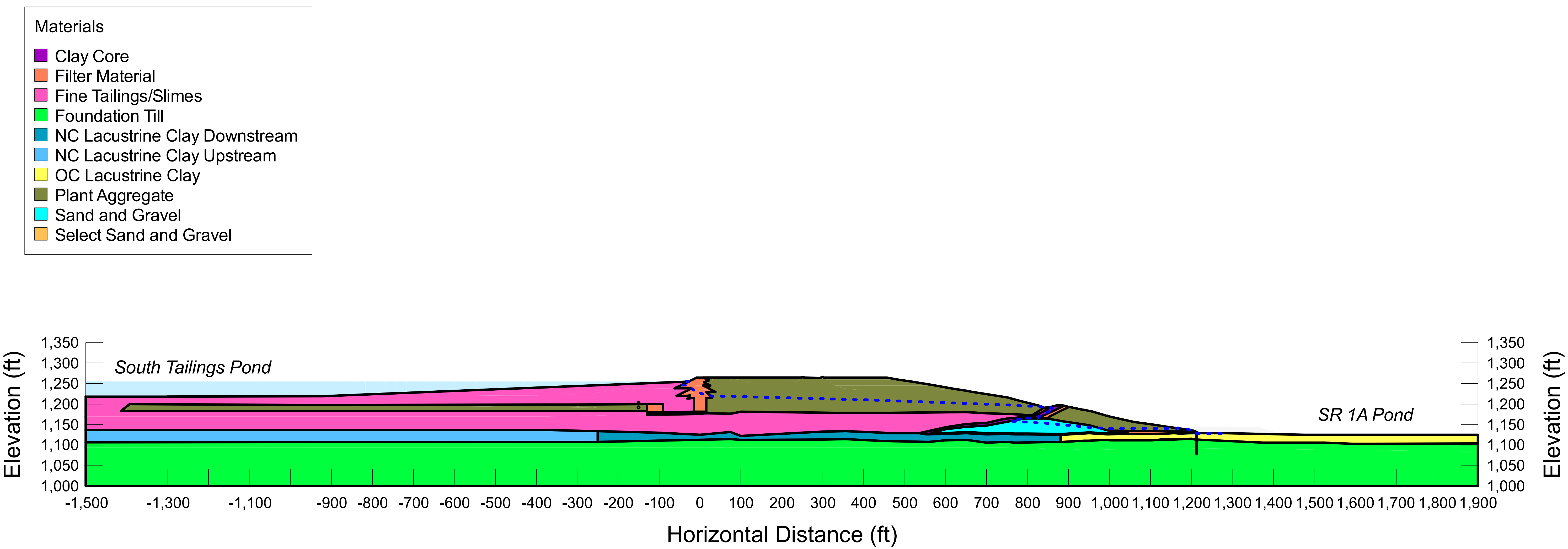


Figure
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 1, Station 35+00 Geometry

Materials

- Clay Core
- Filter Material
- Fine Tailings
- Foundation Till
- NC Lacustrine Clay Downstream
- NC Lacustrine Clay Upstream
- OC Lacustrine Clay
- Plant Aggregate
- Sand and Gravel
- Select Sand and Gravel

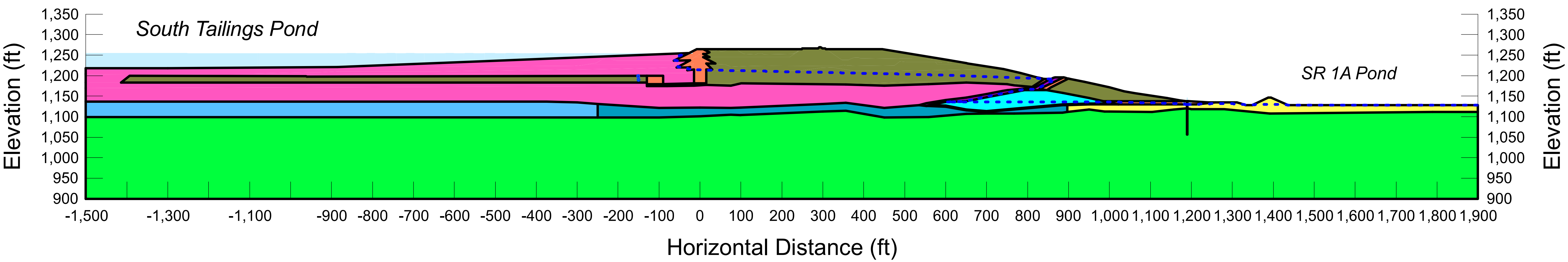


Figure
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 1E, Station 90+50 Geometry

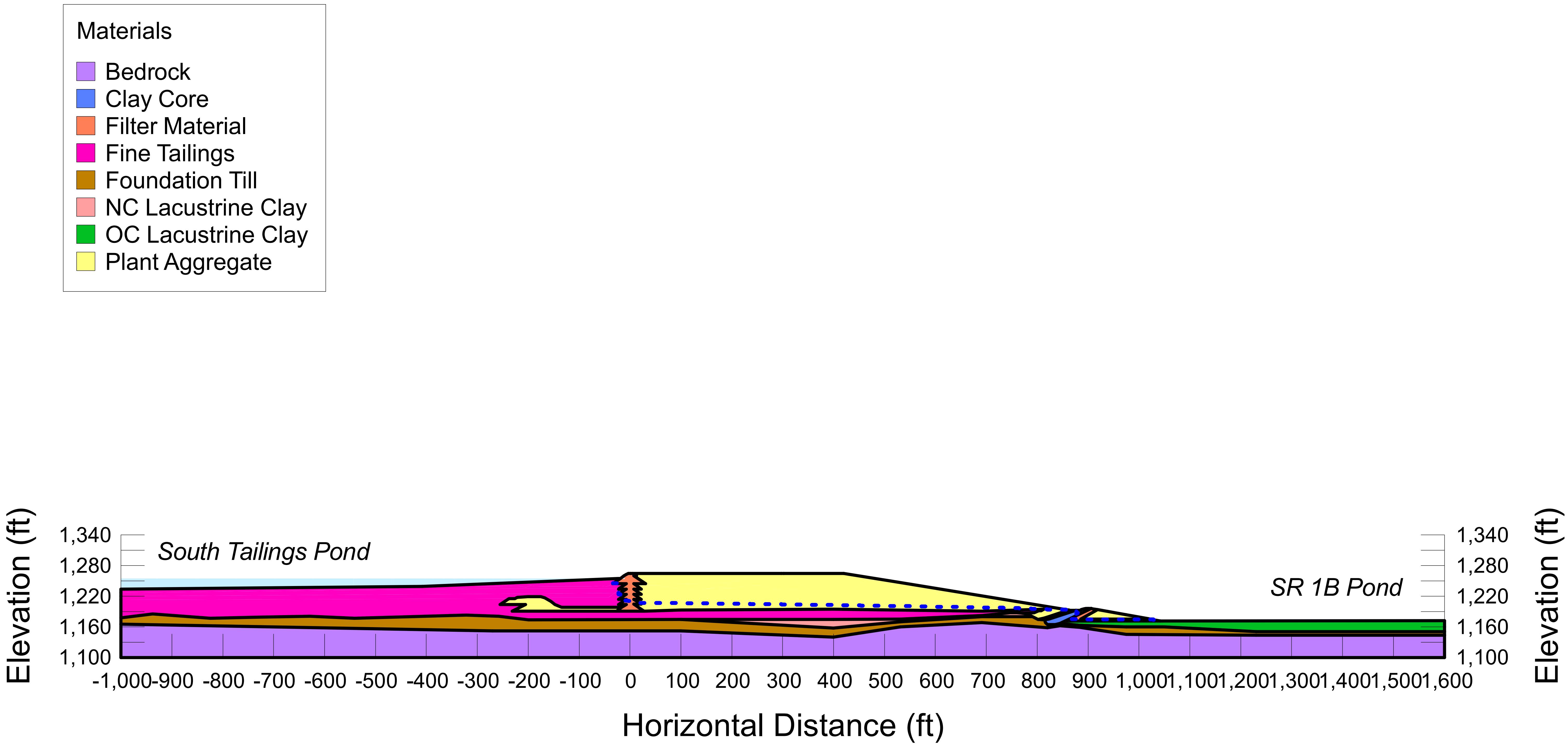


Figure
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 1W, Station 0+00 Geometry

Color	Name
	Bedrock
	Filter Berm
	Fine Tailings
	Foundation Till
	Plant Aggregate

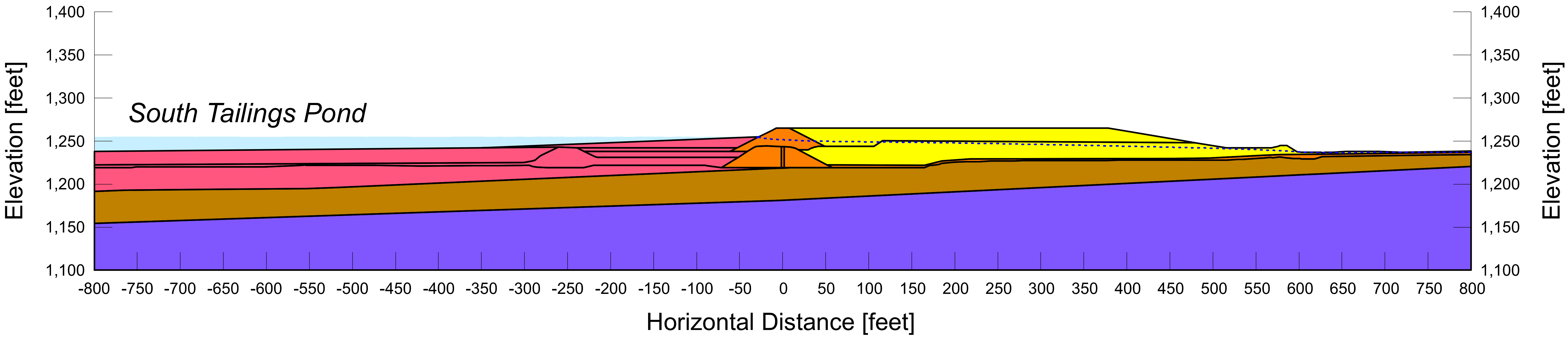


Figure 6
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 1W, Station -10+00 Geometry

Color	Name
	Bedrock
	Filter Berm
	Fine Tailings
	Foundation Till
	Plant Aggregate

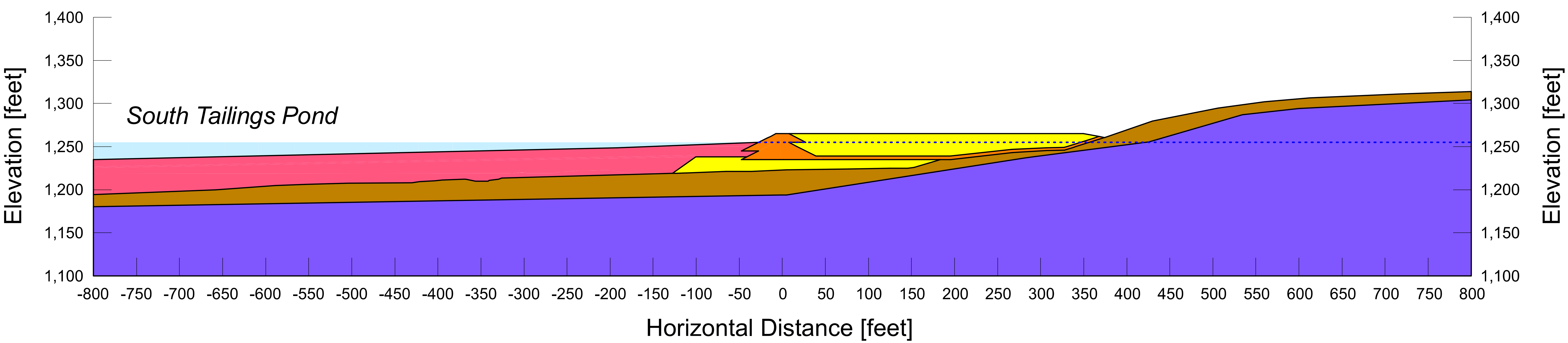


Figure 7
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 1W, Station -17+00
Geometry

Color	Name
	Bedrock
	Filter Berm
	Fine Tailings
	Foundation Till
	Plant Aggregate

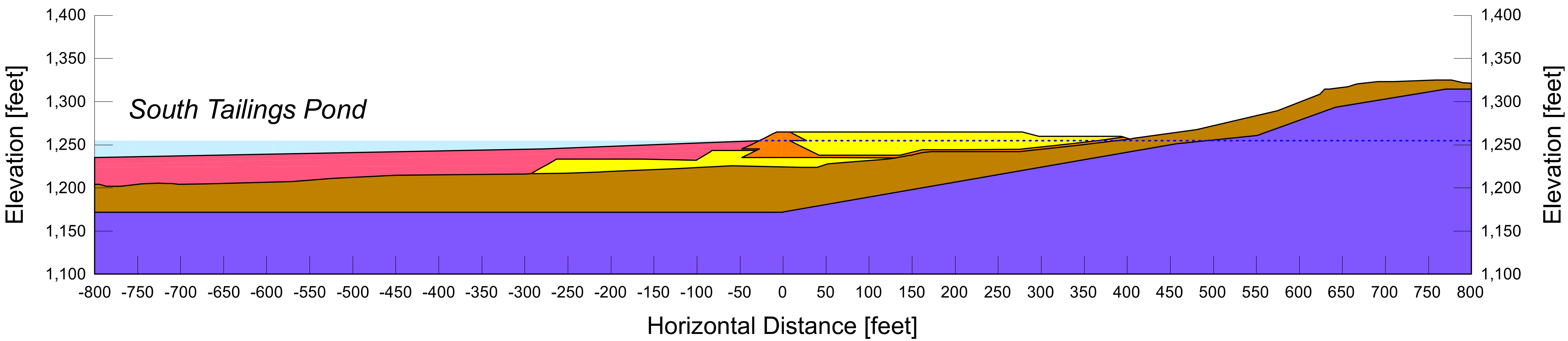


Figure
Northshore Mining Company
Milepost 7 Tailngs Basin
Dam 2, Station 34+75 Geometry

Materials

- Bedrock
- Clay Core
- Filter Material
- Fine Tailings
- Foundation Till
- NC Lacustrine Clay
- OC Lacustrine Clay
- Peat
- Plant Aggregate

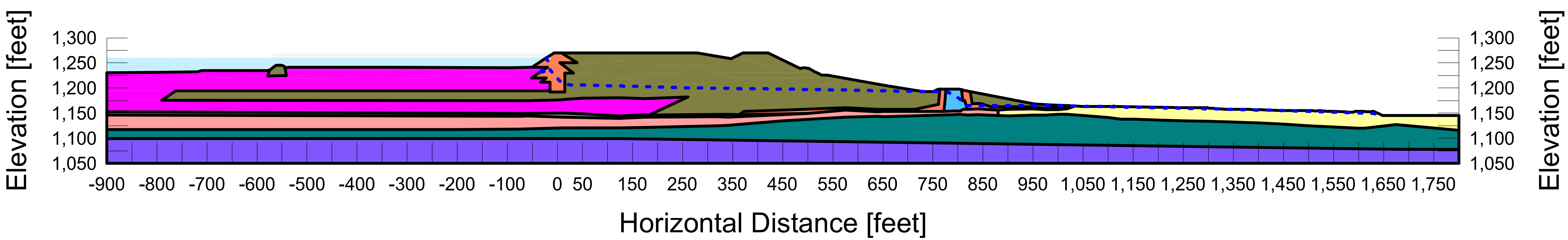


Figure
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 2, Station 42+00 Geometry

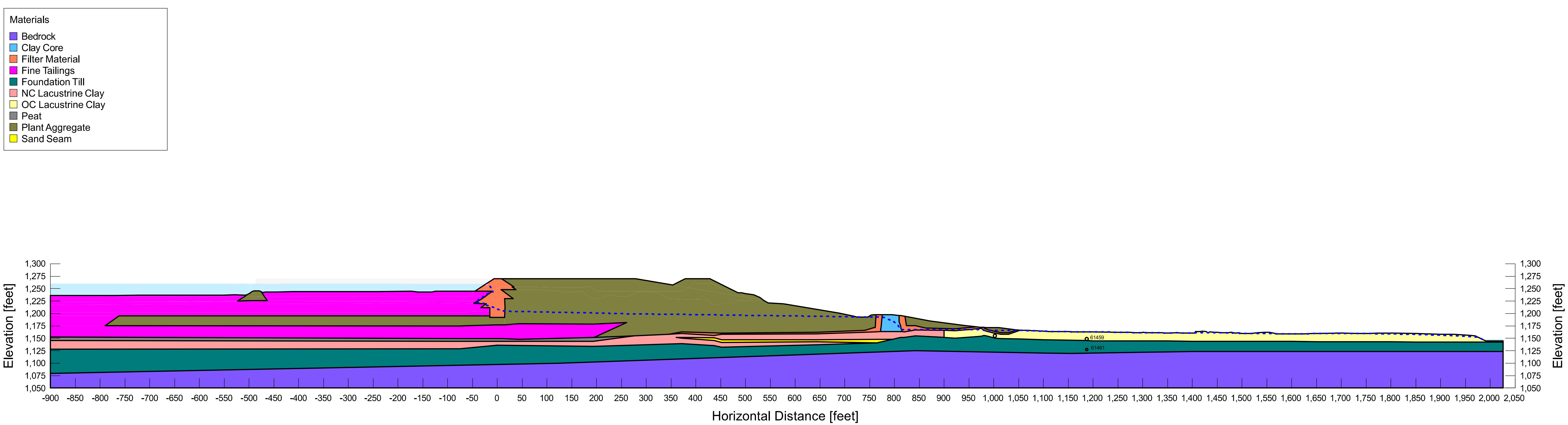


Figure
Northshore Mining Company
Milepost 7 Tailings Basin
Dam 5, Station 14+00 Geometry

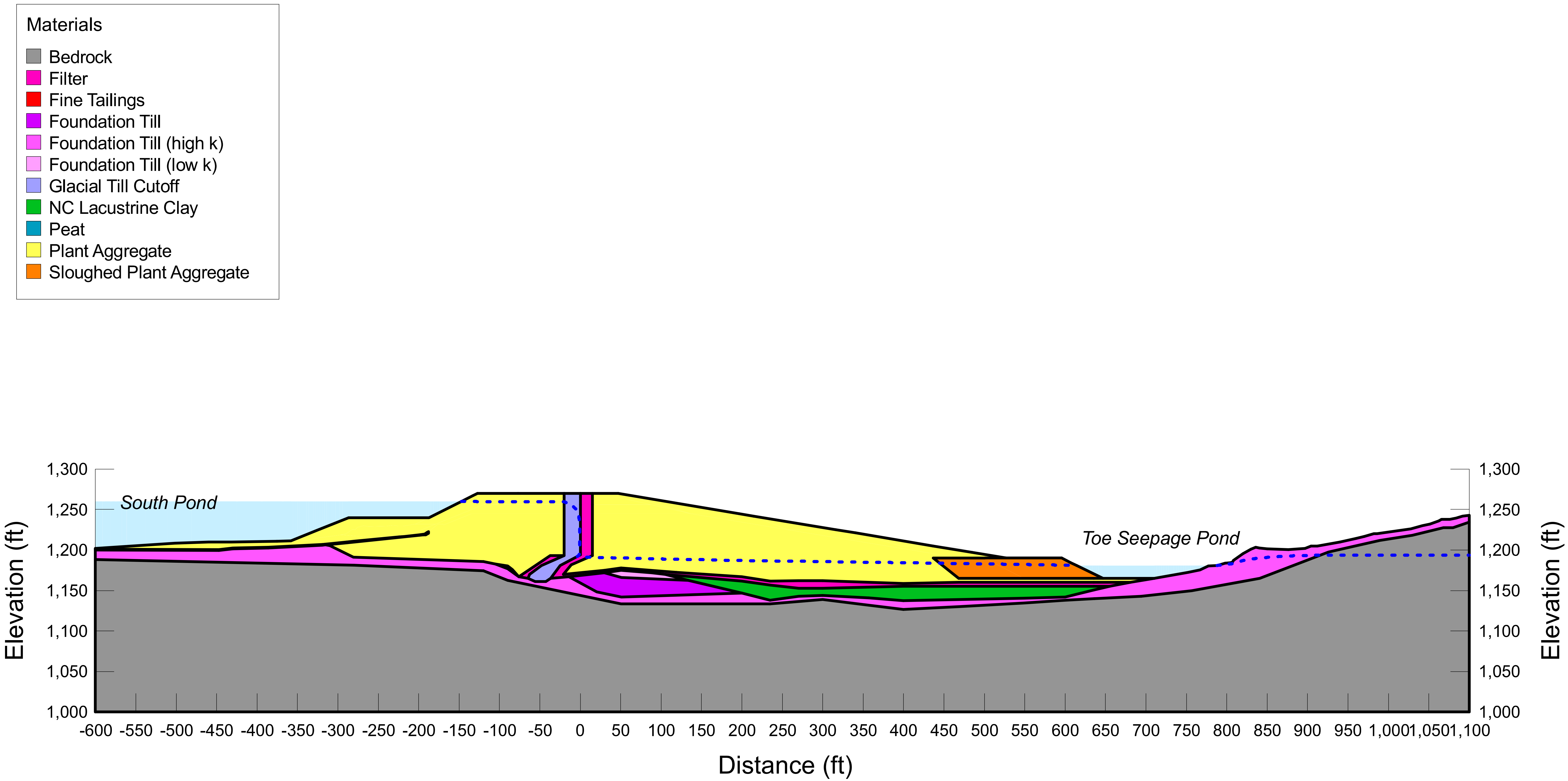


FIGURE 11
Plant Aggregate Strength from Direct Shear and CPT Data
Northshore Mining Company Milepost 7 Tailings Basin

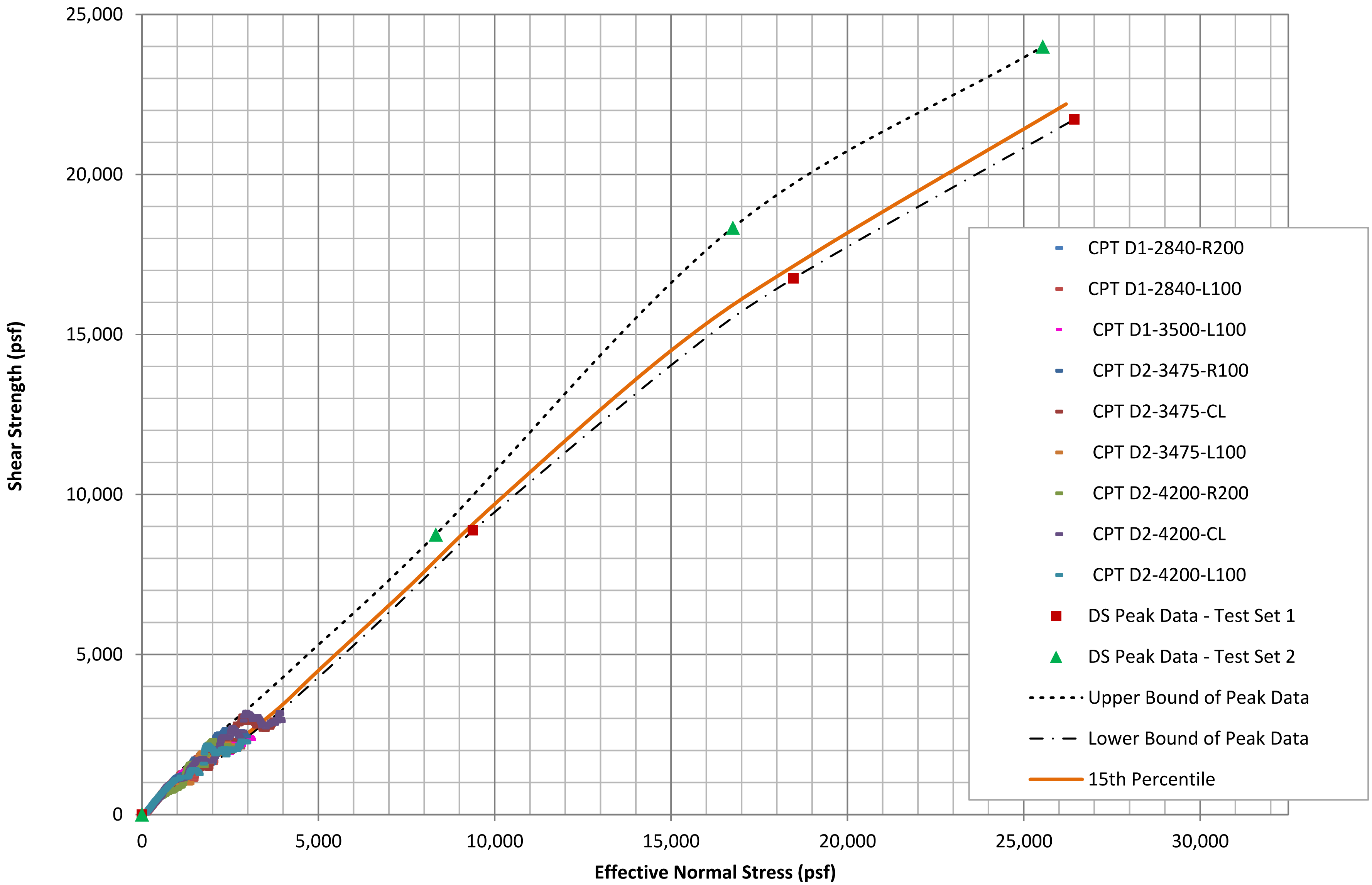
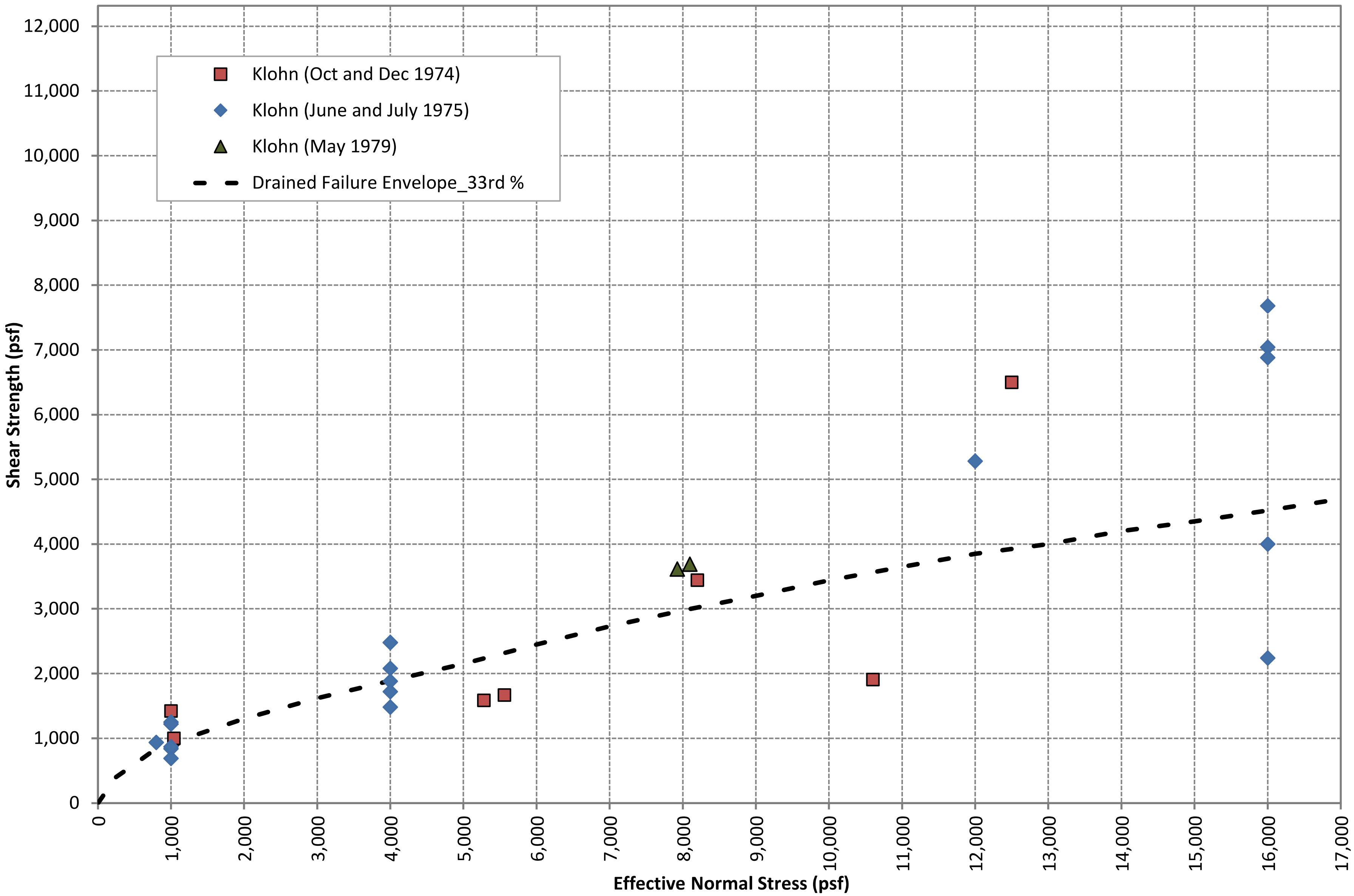


FIGURE 12

Lacustrine Clay Drained Shear Strength - Direct Shear Data

Northshore Mining Company Milepost 7 Tailings Basin



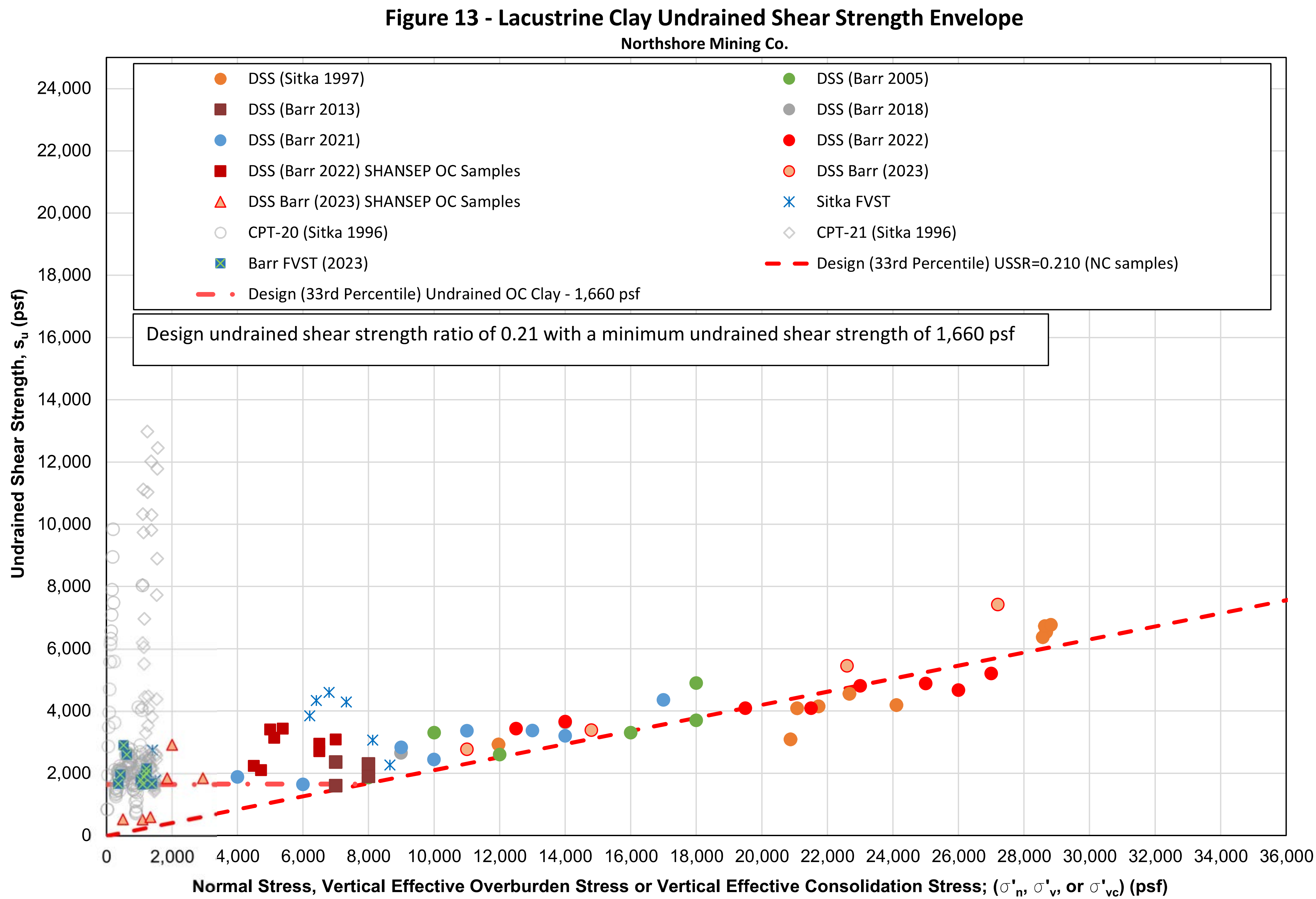


Fig 4 ID23 Analysis

FIGURE 14
Lacustrine Clay Undrained Shear Strength - Overconsolidated, Dam 2
Northshore Mining Company Milepost 7 Tailings Basin