Waterman Lake Dam - Main Dam Study

Seepage Investigation & Recommendation Report

March 2022

Smithfield, Rhode Island

Dam Name: **Waterman Lake Dam - Main Dam Study**

ID#: **111**

Owner: **Citizens for the Preservation of Waterman Lake**

Town: **Smithfield**

Consultant: **Pare Corporation**

PREFACE

The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigations and analyses involving topographic mapping, subsurface investigations, testing and detailed computational evaluations are beyond the scope of this report.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection, along with data available to the inspection team. In cases where an impoundment is lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions, which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is critical to note that the condition of the dam is evolutionary in nature and depends on numerous and constantly changing internal and external conditions. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

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APPENDICES

1.0 PROJECT BACKGROUND

In a letter dated May 20, 2021, the State of Rhode Island Department of Environment Management (RIDEM) Bureau of Environmental Protection Office of Compliance and Inspection issued a response to findings of the 2018 regulatory inspection. requiring the Citizens for the Preservation of Waterman Lake, Inc (CPWL), amongst other items to:

1. Retain a professional engineer fully registered in the State of Rhode Island, who is experienced with dam inspections, to complete a detailed investigation of the area of leakage with possible sediment transport, and submit a report of the investigation findings to DEM. The report must specify and additional actions necessary to return the dam to a safe condition and include a schedule to complete the work. For any proposed repairs to the dam, the report must include an application prepared in accordance with Part 1.108.
2. The investigation program must complete in accordance with the Rules and Regulations for Dam Safety. Part 1.10B.

In June 2020, a regulatory dam inspection was completed by Pare personnel at the request of the RIDEM. The inspection was performed in accordance with the RIDEM Rules and Regulations for Dam Safety. As indicated within the 2020 inspection report, a boil area was observed within the wooded downstream area at the right end of the Main Dam. Per a review of previous inspections, seepage and saturation at the downstream toe of the Main Dam between the right abutment and the outlet have been a recurrent problem. The continued presence of the seepage, along with the observation of sediment within one of the seepage areas prompted the RIDEM to issue a letter requiring the CPWL to investigate the seepage and submit a report of the findings as well as potential repair approaches to prevent continued development. CPWL contracted Pare Corporation to complete the investigation and report.

1.1 Site Description

The following is paraphrased from information contained within the 1977 ACOE Phase I Inspection Report and updated to reflect additional observations and current local terminology:

> The Waterman Lake Dam is an old, long, earthen structure impounding a relatively shallow regulating reservoir used formerly for industrial water supply and currently used for recreation. The entire structure consists of a main dam with an outlet structure (located at the left end of the system), an earthen dike (referred to as Booker’s Dam), short concrete flood walls in the area of the Marina between Bookers Dam and Pine Ledge Road, an earthen dike with a concrete core wall located between the Marina and West Greenville Road, and an earthen dam with a concrete core wall and an overflow spillway structure located right of West Greenville Road.

For the purposes of this Conceptual Design Report, only descriptions of the Main Dam will be included.

> *MAIN DAM*
>
> The main dam is approximately 19 feet high at its highest point at the gatehouse and 430 feet long abutting into high ground at each end. The dam is basically an earthen embankment section that formerly included a dry masonry wall along the upstream slope but now appears to consist of a slope with dumped stone slope protection. The upstream slope is an approximately 2H:1V slope with 3 to 6-inch diameter stone protection. The crest elevation is estimated at El. 334± MSL, is

approximately 10 feet wide, and consists of a grassed surface. The downstream side is variable. From 30 feet right of the outlet structure to the left abutment, the downstream side consists of a short, grassed slope supported by a dry set stone masonry wall with areas of the wall buttressed with 3 to 6-inch diameter stone. Right of this wall, the downstream side is a 1H:1V grassed slope leading to a grassed toe that transitions to a mildly sloped wooded area that leads to the downstream channel.

An outlet structure, located at about the midpoint of the main dam, consists of a 4-foot wide by 6-foot-high stone masonry culvert with gunite facing that extends through the dam. A stone masonry gatehouse structure is located at the upstream of this culvert that houses two timber gates that were formerly used as the primary closure for this culvert. The gates are no longer used as the primary means of operations and are typically left in the open position; however, they are operable if they needed to be used. Upstream of the gatehouse is a rectangular concrete channel formed by 1.5-foot-thick concrete walls and an apparent concrete floor that extends 10 feet upstream. The upstream end of the structure is open to the impoundment and is equipped with removable steel plates (each approximated at 12 inches high by ½ an inch thick set within steel channels within two vertical steel I-beams that are permanently affixed to the upstream end of the walls of the structure. The plates serve as the primary means of closure/operations of the outlet structure and can be added by hand and removed by use of a steel chain and pully system that is supported by the I-beams. A trash rack is present upstream of the steel plates that appears to be welded to the upstream face of the I-beams. The culvert daylights through an apparent concrete retaining wall along the downstream side of the dam. Downstream of the wall, the downstream channel is formed by granite block wing walls with a gunite facing and a gunite faced channel floor. The floor transitions to a natural downstream channel approximately 15 feet downstream of the downstream end of the culvert.

The CPWL dam maintenance committed takes responsibility for operations and maintenance at the dam, including undertaking routine maintenance of vegetation along the length of the dam, completing routine inspections, implementing dam improvement programs as budgets permit, completing other maintenance as required, and adding/removing steel plates from the outlet structure at the main dam as pool levels and anticipated storm events warrant. Pond levels are measured and recorded daily and are available on the CPWL's website (www.cpwl.org).

Per the State of Rhode Island Dam Safety Rules and Regulations, Waterman Lake Dam is classified as a **High** hazard potential dam.

1.2 Study Section

For the purposes of this Conceptual Design Report, the study area is limited to a 150 feet foot long section of embankment adjacent to the right abutment of the Main Dam. The study section consists of an apparent section of manmade fill, forming the dam embankment, and an apparent section of native terrain.

The dam embankment consists of a riprap covered upstream slope, a level and vegetated crest, and a vegetated downstream slope. On the downstream side, the dam embankment is considered to extend from the top of the crest to approximately El. 326.0. The dam embankment was constructed atop an apparent section of native terrain. The native terrain appears to extend from elevation 326.0-325.0 to the downstream discharge channel formed by discharges from the low-level outlet. The native terrain consists of a roughly level bench extending 5 to 25 feet downstream of the dam before sloping at approximately 3.5H:1V to the downstream channel (approximately elevation 317.0). While the bench is generally covered with grasses, the downstream slope has numerous developed trees ranging in size from sapling to 48 inches diameter.

The transition elevation between the dam embankment and native terrain was further defined based upon soil layering and samples collected during the subsurface exploration, further described in Section 2.0 of this report). While material noted herein as native deposits is so noted due to its matching the description of the native soils within the USGS Surficial Geology Maps, there remains the potential that native soils were used to construct what is herein described as native terrain.

2.0 DATA COLLECTION

2.1 Site Survey

A limited topographic survey of the dam at the study area was completed on October 8, 2021, by Pare. The limits of the survey extended from near the right abutment to approximately 150 feet left of the right abutment, inclusive of the area of seepage; and from the normal upstream water level to approximately 80-feet downstream, to the edge of the downstream tailwater. A supplemental survey was completed using a handheld GPS on December 23, 2021 once the winter drawdown of the impoundment was completed to supplement data along portions of the upstream slope normally under water.

The survey was completed using Trimble R10-2. Horizontal coordinates on the drawing are based on the Rhode Island grid system North American Datum of 1983. Vertical elevations are based on the North American Vertical Datum 1988. Two benchmarks were established. One was set at the downstream left corner of the spillway gatehouse, one was set at the right abutment, near the crest upstream edge. The following tables provides details of the benchmarks:

Table 1-1 Benchmarks

ID	Location on Crest	North	East	Elevation (ft)	Description
B21-1	Right Abutment at Main Dam	289465.071	307123.076	332.88	Nail on wooden stake
B21-2	Downstream gatehouse left corner	289685.979	307165.965	333.76	

Topographic survey completed by Pare was utilized to develop the existing conditions plan, as presented in Figure 3.1: Study Section.

A limited visual inspection at the study area of the dam was performed during this survey. The following summarizes the deficiencies observed at this location:

- Active seepage and saturated ground were noted along the downstream toe and along the slope of the natural terrain downstream of the dam.
- Seepage was observed to have heavy iron oxide staining and evidence of sediment transport.
- The downstream slope of the dam is steep with soft surface soils.
- Numerous trees ranging between 12 and 36 inches in diameter were noted along the downstream toe along the native ground and along the downstream slope of the native ground. Seepage was observed discharging from beneath the root systems of numerous trees throughout the downstream area.

2.2 Subsurface Investigation

2.2.1 Regional Geology Setting

Surficial Geology: Based on the "Geologic Map of the Georgiaville Quadrangle, Rhode Island Surficial Geology" by Gerald M. Richmond (1953), the area is identified as "Kame Deposits". Kame consists of "pebble to cobble gravel and sand, poorly sorted with thin layers of silt and clay".

Bedrock Geology: According to the “Geologic Map of the Georgiaville Quadrangle, Rhode Island Bedrock Geology” by Gerald M. Richmond (1952), the bedrock within the study area consists of Porphyroblastic (apg) of the Absalona formation. Apg consists of “dark gray to black, medium to coarse grained, containing inclusions of quartz-biotite schist and quartz-epidote nodules. Rock is composed of albite (25 to 40 percent), microperthite (15 to 30 percent), quartz (10 to 25 percent), biotite (10 to 20 percent), amphibole (1 to 2 percent), chlorite (5 to 10 percent), and trace of epidote, garnet, magnetite, allanite, rutile, sphene, zircon, apatite, and caloite”.

2.2.2 Subsurface Program

A subsurface investigation program was performed by Northern Drill Service, Inc of Northborough, Massachusetts on September 20, 2021, and observed by Pare personnel. The purpose of this subsurface investigation was to explore, sample, and characterize the subsurface soil, install a open pipe piezometer to allow for the measurement of groundwater elevation, and to determine engineering soil properties for use within seepage and slope stability models of the existing embankment. Pare personnel provided field observation and coordination for the subsurface exploration program. Field personnel observed the exploration conditions, collected split spoon samples, visually identified, and sampled the soil strata encountered, and took groundwater measurements.

A total of two borings, B21-1 (OW) and B21-2, were completed at the study area of the dam. Borings B21-1 and B21-2 were advanced using 4-inch casing and wash and drive drilling methods to 24.0 feet (B21-1) and 24.5 feet (B21-2) below the existing ground surface. The boring locations are shown on Figure 3.1: Study Section. Boring logs are included in Appendix B: Subsurface Exploration Data.

Table 2-2 Boring Summary

Boring ID	Location on Crest	Depth (feet)	Instrumentation
B21-1	Upstream of Crest Centerline	24.0	Monitoring Well
B21-2	Downstream of Crest Centerline	24.5	None

During the explorations, subsurface soils were visually classified utilizing the Burmister Classification System. This system describes soil composition based upon the percentage of soil particle size present by weight in the sample with the major soil particle size listed first followed by other soil components described as “trace” indicating 0-10% by weight, “little” indicating 10-20% by weight, “some” indicating 20-35% by weight or “and” indicating 35-50% by weight.

2.2.3 General Subsurface Conditions

The material at the proposed site generally consists of TOPSOIL overlying EMBANKMENT FILL overlying SAND overlying GLACIAL DEPOSITS overlying presumed BEDROCK. The following describes the general subsurface soil profile encountered:

TOPSOIL was encountered in all borings. TOPSOIL is generally be described as moist, brown, fine SAND and SILT, trace roots.

EMBANKMENT FILL was encountered in all borings. EMBANKMENT FILL is generally described as moist to wet, very loose to loose, brown, SILT, “little” to “and” amount of fine to coarse SAND, “trace” to “and” amount of fine to coarse GRAVEL, “trace” amount of organics (roots).

SAND was encountered in boring B21-1 below the EMBANKMENT FILL, in B21-2 between the GLACIAL DEPOSITS. SAND is generally described as wet, medium dense to very dense, fine to coarse SAND.

GLACIAL DEPOSITS were encountered in both borings. GLACIAL DEPOSITS is generally described as moist to wet, light brown to tan to gray, fine to coarse GRAVEL, "some" to "and" amount of fine to coarse sand, "trace" amount of silt. A layer of BOULDER was encountered within the GLACIAL DEPOSITS in B21-2 between 17.5 feet to 18.5 feet.

POSSIBLE BEDROCK was encountered within B21-2. Spit spoon refusal was encountered at 19.5 feet. The driller advanced the roller-bit to 24.5 feet to confirm bedrock. Note that after the winter drawdown was completed, an area of bedrock outcrop was observed approximately 150 feet left of the right abutment.

Variations in depths to, and thickness of the deposit should be anticipated between and away from the borings. The layering and relative density of the subsurface deposits encountered in the borings are summarized in tabular form below:

Table 2-3 Generalized Subsurface Conditions

Stratum	B21-1		B21-2	
	Thickness	Notes	Thickness	Notes
Topsoil	2 inches		2 inches	
Embankment Fill	12 feet	very loose to loose fill	8 feet	very loose to loose fill
Sand	2 feet	medium dense	1 foot	very dense
Glacial Deposits	8 feet	medium dense	9 feet	very dense
Bedrock	Did Not Encounter		Presumed weathered bedrock encountered approximately 19.5 to 24.5 feet below the ground surface, drill advanced by roller bit. Coring was not performed.	
Depth to Water	4.0		4.0	
Water Elevation	328.3		328.6	

2.2.4 Groundwater Readings

A monitoring well was installed at B 21-1 during the subsurface exploration. Groundwater readings were taken and are summarized in the table below.

Table 2-4 Monitoring Well Readings

Boring ID	Date	Depth to Water	Well Water Elevation[1]	Impoundment Level[1]	Tailwater Elevation[1]
B21-1	9/20/2021	4.0	328.3	329.6 +/-	Not Measured
B21-1	10/8/2021	6.2	326.4	329.6	317.2
B21-1	11/18/2021	7.7	324.9	327.6	316.7
B21-1	12/23/2021	10.4	322.2	324.5	317.4

Note that groundwater levels fluctuate due to local and regional factors including, but not limited to, precipitation events, seasonal changes, period of wet or dry weather, and the impoundment level.

[1] NAVD 88

3.0 EVALUATIONS & ASSESSMENTS

3.1 Laboratory Testing

The laboratory testing program included mechanical grain size determinations performed on samples of the strata encountered during the investigation. The results of the laboratory testing are summarized below. The data sheets are included in Appendix B.

3.1.1 Grain Size Analysis

Three grain size determination tests were performed by Pare in general accordance with ASTM D-422 on samples recovered during the subsurface investigation with descriptions and results presented as follows:

Table 3-1 Results of Grain Size Analyses

Test No.	Boring No.	Sample No.	Depth (Ft.)	Representative Soil	Water Content %	% Gravel	% Sand	% Fines
1	B21-1	S-4	6 – 8	Embankment Fill	20.2	34.9	47.3	17.8
2	B21-2	S-2	2 - 4	Embankment Fill	15.6	20.6	41.7	37.8
3	B21-2	S-7	14 – 16	Glacial Deposits	10.0	40.1	49.3	10.6

3.2 Liquefaction Evaluation

In general accordance with the ASCE 7-16 and using the N-Bar Method, the dam site has a Site Class “D” (i.e. Stiff Soil with 15<N-bar<50)[1]

The seismic parameter for the site was developed in accordance with RI SBC-1-2019 and ASCE 7-16. Based upon available information, a modified peak ground acceleration for liquefaction potential coefficient of 0.169 was selected for the site.

Liquefaction is the tendency for a soil type, particularly fine sands, to lose a significant amount of strength and behave like a liquid in the event of an earthquake or sufficient vibrations. Liquefaction analyses generally relate SPT N values, corrected for overburden, hammer efficiency, fines content, and measured groundwater levels to the liquefaction potential of the materials in question. In general, for liquefaction to occur, three conditions must be met simultaneously. These are: 1.) The presence of loose, clean sandy soils, 2.) Saturated conditions, and 3.) Vibration.

The liquefaction analyses completed during the preparation of this report was performed in accordance with the summary report from the 1996 NCEER (National Center for Earthquake Engineering Research) and 1998 NCEER/NSF (National Science Foundation) workshops on “Evaluation of Liquefaction Resistance of Soil” prepared by Youd et al. (2001). The analysis considers the soil and groundwater conditions encountered at the time of the subsurface exploration program. Fluctuations in groundwater levels could have a significant effect upon the liquefaction potential of soils. If the groundwater is observed to change during the construction process or future explorations, Pare should be contacted as it may be necessary to re-analyze the soil for liquefaction potential.

[1] N-bar is the average Standard Penetration Test N-value for the upper 100 feet of soil.

Based upon the observed relative densities, groundwater elevation, and material composition, it appears that the in-situ soils are not susceptible to liquefaction at this time.

3.3 Seepage & Slope Stability Modeling

Seepage and slope stability models to evaluate the current embankment performance were developed utilizing the SEEP/W and SLOPE/W modules within the GeoStudio 2021 R2 Version 11.1.0.22070 finite element software. The finite element model was used to model seepage, potential seepage breakout, exit gradients, and seepage flows through the dam embankment at normal and maximum pool levels. An existing conditions model was developed, and the initial model results were compared to observed seepage rates; appropriate calibration of the model was completed to provide general agreement between observed conditions and predicted flow rates.

One cross section was located approximately 60 feet left of the right abutment to develop and existing condition model in the area of the observed boil. This location was deemed the critical section due to the steepest downstream slope and the highest elevation of seepage breakout downstream. Using information from the collected soil samples, grain size analyses, and Standard Penetration Tests, geotechnical strength parameters were calculated and used to populate the slope and seepage stability models. Geotechnical parameters used are summarized in Table 3-2.

3.3.1 Soil Parameters

Based on a review of available information, results of laboratory testing, engineering judgement based on previous projects with similar material types, and available geotechnical calculations and correlations, the following soil properties were developed for us in the analysis of the existing embankment geometry.

Table 3-2 Soil Properties of Existing and conceptual Embankment Materials

Soil Layer	(N_160) (Blows/ft)	D_r (%)	Angle of Internal Friction (º)	Sat. Unit Weight (pcf)	Porosity Saturated Water Content (%)	Saturated Hydraulic Conductivity (ft/sec)	Residual Water Content
Embankment Fill	8	20	29	127	20	3.3E-5	0.03
Sand	26	54	34	131	20	3.28E-3	0.02
Glacial Deposits	100	95	39	137	10	3.28E-5	0.03
Riprap	N/A	N/A	40	145	10	0.1	0.005
Weathered Bedrock	N/A	N/A	N/A	N/A	N/A	1.0E-8	N/A

3.3.2 Model Results

Table 3-3 presents the results of the seepage stability analyses for the existing condition. Factors of safety (FOS) are based on U.S. Army Corps of Engineers dam safety recommendations. Bolded values indicate cases where the minimum requirements were not met.

Table 3-3 Result of Seepage Analyses for Existing Conditions

Design Case	Pool Level	Construction Embankment Section		Native Terrain Slope	
		Breakout height Above Toe (ft)	Calculated FOS	Breakout height Above Toe (ft)	Calculated FOS
Steady State	Drawdown	None	N/A	At toe	5.0
	Normal	None	**3.0**	1.5	**3.6**
	Max	At downstream slope toe	**2.0**	4.0	**3.0**

Downstream slope: *The 1.5H:1V area between the crest and the downstream, approximately between El. 332 to El. 326.*
Native Terrain slope: *The area downstream of the downstream slope., approximately from El. 326 to the downstream channel. The area ranging between 3H:1V to 10H:1V.*

Seepage models suggest that the current seepage performance of the dam during Normal and Maximum pool conditions do not meet required factors of safety and seepage breakout requirements. In general, seepage breakout should be at or beneath the toe. In cases where a tailwater is present, such as this study area) seepage breakout should not occur above the tailwater elevations. A more critical evaluation of seepage stability is the factor of safety (FOS) against piping. Piping is the ability for seepage waters to transport individual soil particles due to buoyancy and water pressure, leading to a uncontrollable internal erosive condition. The predicted FOS against piping ranges from 2.0 to 3.6; typical factors of safety against piping should be above 5.0.

Table 3-4 presents the results of the slope stability analyses for the existing conditions at the dam. Factors of safety are based on U.S. Army Corps of Engineers dam safety recommendations. Bolded values in the “Calculated Factor of Safety” columns indicate cases where the minimum requirements were not met.

Table 3-4 Result of Slope Stability Analyses for Existing Conditions

Loading Case	Required FOS	Upstream Slope	Downstream Slope
Drawdown from Max	1.1	1.7	NR
Drawdown from Normal Pool	1.2	1.8	NR
Steady State at Normal Pool	1.5	1.6	**1.2**
Steady State at Maximum Pool	1.4	NR	**0.7**
Earthquake at Normal Pool	1.0	1.0	**0.8**

NR – Not Required through USACE.

Slope stability models indicated factors of safety less than required along the downstream slope under all loading conditions considered (Maximum, Normal, and Earthquake).

3.4 Implications of Findings

During the investigations the condition described during the 2020 report was confirmed:

> “50 feet downstream of the toe, and near the right abutment of the mild wooded slope of the downstream area, several areas (3±) of seepage were noted, each approximated between 1-3 GPM, some with iron oxide staining and one with a possible accumulation of sand downstream of the seepage area.”

Under normal pool conditions, soils at the toe of the dam embankment were saturated and areas of seepage were observed breaking out from the downstream slope above the elevation of the tail water. Several areas of concentrated seepage were present in close proximity to the root systems of the many trees along the downstream slope of the native terrain. Seepage was generally flowing clear; however, iron oxide staining and flocculent was present at all seepage locations. Sediment present in the seepage areas appeared to be old, as the iron oxide staining and flocculent was developed over the material; however, sediment was present indicating the potential for internal erosion to have occurred. It was unclear if the sediment deposition was merely the result of localized erosion at the point of seepage breakout, or if the sediment was sourced from deeper within the embankment/native terrain as a result of internal erosion. While factors of safety against piping within the SEEP/W model were calculated to be greater than 2, there is the potential that localized conditions, such as, soils loosened from tree roots or non-homogenous soils could result in areas where the factor of safety could be less than 1, causing internal erosion to develop locally.

It was apparent through both seepage modeling and field observations prior to and during the impoundment drawdown, that the seepage rates are directly influenced by the level of the impoundment. As impoundment levels decreased during the regular winter drawdown implemented during the study timeframe, both the water level within the B21-1 monitoring well and the seepage breakout height decreased. Field observations correlated well with predictions of performance by the SEEP/W program.

4.0 REPAIR RECOMMENDATION

For the purposes of this study, the existing conditions seepage and slope stability models were modified to develop conceptual embankment cross sections which would address the seepage and slope stability concerns identified as part of this evaluation. A single cross section was chosen to model the critical condition under conceptual conditions and develop a repair approach that would meet current embankment design standards.

4.1 Recommended Repairs Approach

Based on the observed conditions at area of study at the Waterman Lake Main Dam and completed modeling, the following repair approaches are recommended:

1. **Tree Clearing Program:** Based on observations made during the various site assessments it is apparent that the mature tree growth along the native terrain downstream of the dam is affecting the seepage patterns through the dam. Seepage breakout at several locations was concentrated beneath the roots systems of nearby trees. While root systems of trees do provide a degree of surface erosion protection, they cause detrimental effects to a dam or other water retaining structure, such as locally loosen soils as a result of rotting roots, displace soil if the tree is subjected to high wind loading, and loosen soils as the root systems pull against the soil matrix to stabilize the tree. Once soil along a root system has loosened, the space between the root and the intact soil becomes a preferential pathway for water to travel, leading to increased seepage and potentially piping erosion.

   Pare recommends clearing trees, brush, and other unwanted vegetation from the native terrain between the dam and the downstream tailwater. Stumps and root systems larger than 0.5 inches in diameter should be grubbed within the footprint of the slope. Resulting voids should be filled with approved material and compacted in lifts, no larger than 12-inches thick, to subgrade for the specific treatment (i.e. loam and seed, riprap, etc.). Once cleared, grubbed, and backfilled, disturbed surfaces should be covered with loam and seeded. Completion of the work, especially grubbing, during a normal or elevated pool level could result in uncontrolled seepage and internal erosion of the native foundation soils and a rapid loss of the dam. Given the observed seepage, this work should only occur during the winter drawdown under the observation of an engineer practiced in dam engineering.

2. **Seepage Mitigation and Slope Stability Improvements:** Pare evaluated several measures to address the observed and modeled conditions of seepage along the toe of the dam and areas of slope instability. Each of the following recommendations could be installed singularly to address the observed conditions; however, combinations of these measures may allow for a more economical solution. The following approaches for repair have been conceptualized as follows:

   a. **Regrade the Downstream Slope and Install a Rock Toe:** This approach would include regrading the downstream slope of the dam and downstream native terrain to 3.5H:1V slope and installing a graded filter/rock toe to approximately 11 feet above the tail water downstream toe and extended 4 feet into the embankment.

      The graded filter/rock toe would act as a blanket drain. While the blanket drain would not lower the phreatic surface, it would intercept it and allow the seepage water to be to be filtered, collected, and discharge into the downstream channel. The blanket drain would be designed to have factors of safety against piping greater than 5.

Regrading of the downstream slope would provide a uniform, maintainable slope, provide the phreatic surface with additional cover preventing premature breakout, and increase overall slope stability.

b. **Install Trench Drain System:** This approach includes the excavation of an approximately 3-foot wide by 10-foot-deep trench located about 5 feet downstream of the dam toe, within the native terrain. A collection drain would be installed within the trench and the trench backfilled with free draining soils/stone to allow the collection of seepage waters. The fill material would be designed to filter seepage flows without sediment transport from the surrounding soils.

The drain would lower the phreatic surface beneath the existing ground surfaces and allow water to be filtered and collected. The drain would allow for a singular point to monitor seepage flows from the dam. This drain would likely require frequent maintenance due to the need for the outfall to be at or below the tailwater elevation, which could cause frequent clogging with debris.

c. **Sheet Pile Wall and Toe Drain:** This approach includes a driven sheet pile cutoff wall installed along the upstream shoulder of the crest to form a continuous impermeable barrier along the length of the dam. The cutoff wall would be anticipated to extend from the top of the dam crest (Average EL. 333.0) to top of bedrock. Based on the subsurface investigation, bedrock elevation is variable between the two borings locations. This option may have limited effectiveness locally due to the variable bedrock elevation, the potential for gaps to be present between adjacent sheet pile sections, and the ability for seepage waters to penetrate these gaps. Given the length of the study area, the current cost of steel, mobilization costs for the equipment required to install the sheeting, and the limited capacity for a sheet pile to adequately cutoff seepage along a variable bedrock surface, this option was not considered economical and would not fully address downstream slope stability concerns; as such, it was not evaluated in additional detail.

Result of seepage analyses under these alternatives are included in Appendix C: Seepage and Slope Results.

4.2 Additional Construction Considerations

Site Access: Access to the right side of the dam is via a dirt road off West Greenville Road and will require travelling through the downstream area of the dam for approximately 0.2 mile before reaching the right abutment. Improvement to the dam access would likely be required prior to travel along the road by heavy construction equipment. The contactor would have to plan activities and deliveries along the roadway as the pathway is only wide enough for a single lane of travel.

Staging Areas: Temporary staging for limited amounts of equipment and materials could be set up at the right abutment of the Main Dam, right of the working area. The crest of the dam near the low-level outlet could be used for material storage, however the ability to stage equipment on the crest is limited. If additional staging is needed, it is likely that an access/easement agreement would need to be made with the adjacent property owner as the property owned by the CWPL beyond the limits of the dam is limited. Additional staging areas could be set up along the access road to the dam.

Control and Diversion of Water: As with any dam repair program, controlling the level of the impoundment and the effect of water upon the completion of the work is a critical component to the success of the project, both in the interest of the quality of the work as well as the safety of the work. Traditionally, two primary methods for controlling water are considered, including temporary drawdowns and/or cofferdams. The extent and type of control of water requirements will be largely dictated by the selected repair approach and time of year during with the work occurs.

If drawdown is not preferred, a cofferdam would be required. The cofferdam may consist of stacked bulk sandbags or a steel frame and tarp system (i.e., Port-A-Dam). Bulk sandbags and steel frame and tarp cofferdams are usually limited to heights of 10 to 12 feet, inclusive of freeboard and settlement into pond sediment; based upon available information, it appears that either of these options would be suitable for work at the dam. If additional height is required, driven sheet piles could be installed; however, this type of system is typically more expensive and conditions at the dam do not appear to warrant installation of such a system.

Water control systems would be required to be in place for the grubbing of stumps and root systems after tree clearing, trench drain installation, or during any other activities that require excavation beyond the existing surface of the dam embankment. Additionally, it may be prudent to implement water controls during the installation of a sheet pile wall, as the weight of the equipment used to install such a system in combination with hydrostatic forces from the impoundment and vibrations from the sheet pile installation may stress the existing embankment into an unstable condition.

4.3 Schedule for Completion

Recommendation #1 – Tree Removal: Pare recommends that actions be promptly taken by the Owner to remove the trees from the downstream slope and native terrain downstream of the dam. Trees pose two types of risk to an embankment:

1) Uprooting: Trees pose a blow over hazard which could result in the disturbance of embankment/foundations soils if a tree were to become uprooted. This loss of soil mass may result in significant reductions to seepage path lengths, leading to potential seepage related concerns developing rapidly.

2) Soil Loosening: Cyclical load of trees due to wind tend to loosen the soil beneath the tree and around root systems. This loading and loosening may cause loose soil conditions at the base of root systems resulting in the discharge of sediment in seepage areas, as is the potential case for the Study Area. As wind loads on trees are increased once the trees leaf out, removal of the trees is recommended in the Spring of 2022. There are an estimated 20 trees greater than 6-inches diameter in the study area that are recommended for removal.

In discussions with the CPWL, budgetary constraints may limit the extent to which they can remove the trees and grub the associate root balls. Therefore, to remove the immediate risk, it is recommended trees be cut such that 2 to 3 feet of the stump remain. This will allow for the easy identification of cut trees during future grubbing work and allow for a future grubbing Contractor the option of removing the tree and associated root balls by anchoring to the tree stump. *Cutting of the stump flush with the ground surface or grinding the stumps below the ground surface IS NOT recommended for a delayed grubbing program, as these actions may limit the ability to locate previously cut trees and will limit the options available to future contractors to adequately grub the root systems.*

If tree cutting is completed this spring, a grubbing program is recommended to be completed within 2 years (i.e by Spring of 2024) as the root systems will begin to deteriorate and may develop preferential seepage pathways if not adequately grubbed and backfilled.

After removal of the trees, it is recommended that a monitoring program be implemented by the CPWL to track changes in the seepage conditions at the dam. The monitoring program should include weekly visits to the study area to visually assess seepage flows, identify any potential sediment releases, measure the level of the impoundment, photograph conditions, and generally assess any changes in condition at the structure. Additional site visits should be completed after rainfall events of 1-inch or more in a single occurrence, 1-inch or more in a single week, during pond refill, and after an earthquake event.

Recommendation #2 – Seepage Collection System: Based on the results of the seepage analyses tree removal, grubbing of the root systems, and filling of disturbed areas, may be sufficient to address immediate concerns with the observed seepage. Given the CPWL's limited budget, it may be possible to petition the Dam Safety Program to allow for the implementation of a monitoring program while a Capitol Improvements Program is developed to further evaluate, design, and construct one of the improvement alternatives recommended to address seepage stability concerns. A monitoring program has been outlined in Section 4.4. It is recommended that the CPWL discuss the development of a Capitol Improvement Program with members and discuss appropriate timelines with the State.

Table 4-1 Recommended Schedule for Repairs

Action	Date
Cut trees to within 2-3 feet of base	Spring 2022
Grub Root systems of cut trees and fill voids	No later than Spring 2024
Implement Monitoring Program	Spring 2022 through Implementation of Recommendation #2; Continuance beyond this period is dependent upon design and observations.
Develop a Capitol Improvement Plan to implement one alternative in Recommendation 2	Begin Plan development Spring 2022
Further evaluate, design, and construct one alternative in Recommendation 2	To be determined

4.4 Monitoring Program Outline

A typical weekly site visit as part of the monitoring program should include the following:

- Measure the elevation of the impoundment.
- Note locations of seepage on a site sketch.
- Estimate seepage flows:
    - In areas where seepage is present, but flow cannot be measured due to low flow, indicate trace flow present.
    - In areas where concentrated flow is present from a single point source, the flow should be funneled to a collection point (i.e. bucket or bottle of known volume) and the time to fill the container measured to estimate flow rates.
    - In areas where flow is present over a wide area, the observer should estimate overall flows and take a video of the flow so that it can be compared to subsequent data points.
- Photograph locations of seepage breakout.
    - Overview photos should be taken of the downstream slopes from both the left and right sides.

  - Close up photos should be taken of individual seepage locations.

  Photographs should be taken from the same locations so that comparisons can be made from subsequent data points.
- Look for sediment discharge from seepage locations. If sediment is observed:
  - Estimate sediment quantities; Document if sediment is on top of or beneath any newly fallen leaves.
  - Take additional photographs of the sediment discharge and extents.
- Upload photographs, videos, and site sketches onto a data server so that information can be stored and tracked.
- Review and comment on any changes from one week to another within a monitoring program document.

The following table provides a list of potential observations and actions to be taken:

Table 4-2 Proposed Monitoring Program Actions for Observed Conditions

<table>
<tr><th>Condition</th><th>Observation</th><th>Action</th><th>Additional Notes</th></tr>
<tr><td>1</td><td>New Seepage Area Develops</td><td>• Increase monitoring frequency to 2x per week for this location until stable conditions develop for a 2 week period.
• If conditions do not stabilize within 2 weeks or conditions rapidly develop, contact an engineer to assess the condition.</td><td></td></tr>
<tr><td>2</td><td>Seepage Rates Increase/Decrease – (No Additional Erosion)</td><td>• Increase monitoring frequency to 2x per week for the study area until stable conditions develop and prevail for 2 weeks.
• Note any changes in the upstream / downstream water levels.</td><td rowspan="2">• Small changes in seepage rates are expected due to varying water surface elevations, and in response to periods of rainfall or drought. However, increases/decreases in seepage rates over periods of time for no apparent reason may be indicative of internal erosion and shifting internal soil structures that require immediate assessment by an Engineer.</td></tr>
<tr><td>3</td><td>Seepage Rates Increase/Decrease – Erosion Observed</td><td>• Contact an engineer to assess the condition
• Increase monitoring frequency to daily for entire study area.
• Place stone within the erosion area to prevent further erosion. DO NOT PLUG SEEPAGE POINTS OR ATTEMPT TO BUILD A CHECK DAM TO CONTAIN SEEPAGE OR EROSION without prior review by an Engineer.
• If seepage rates continue to fluctuate with no apparent cause, take steps to lower the level of the impoundment</td></tr>
<tr><td>4</td><td>Cloudy Discharge is Observed</td><td>• Contact an engineer to assess the condition
• Increase monitoring frequency to daily for entire study area
• If condition persists for more than 24-hours take steps to lower the level of the impoundment</td><td>• Cloudy discharge may occur after rain events due to surficial soils entering just above the seepage breakout zone. This condition should stop a day or two after the rain event.</td></tr>
<tr><td>5</td><td>Sediment Discharge is Noted</td><td>• Contact an engineer to assess the condition
• Estimate Sediment
• Increase monitoring frequency to daily for entire study area</td><td>• Based upon assessment, steps to lower the impoundment may be required</td></tr>
</table>

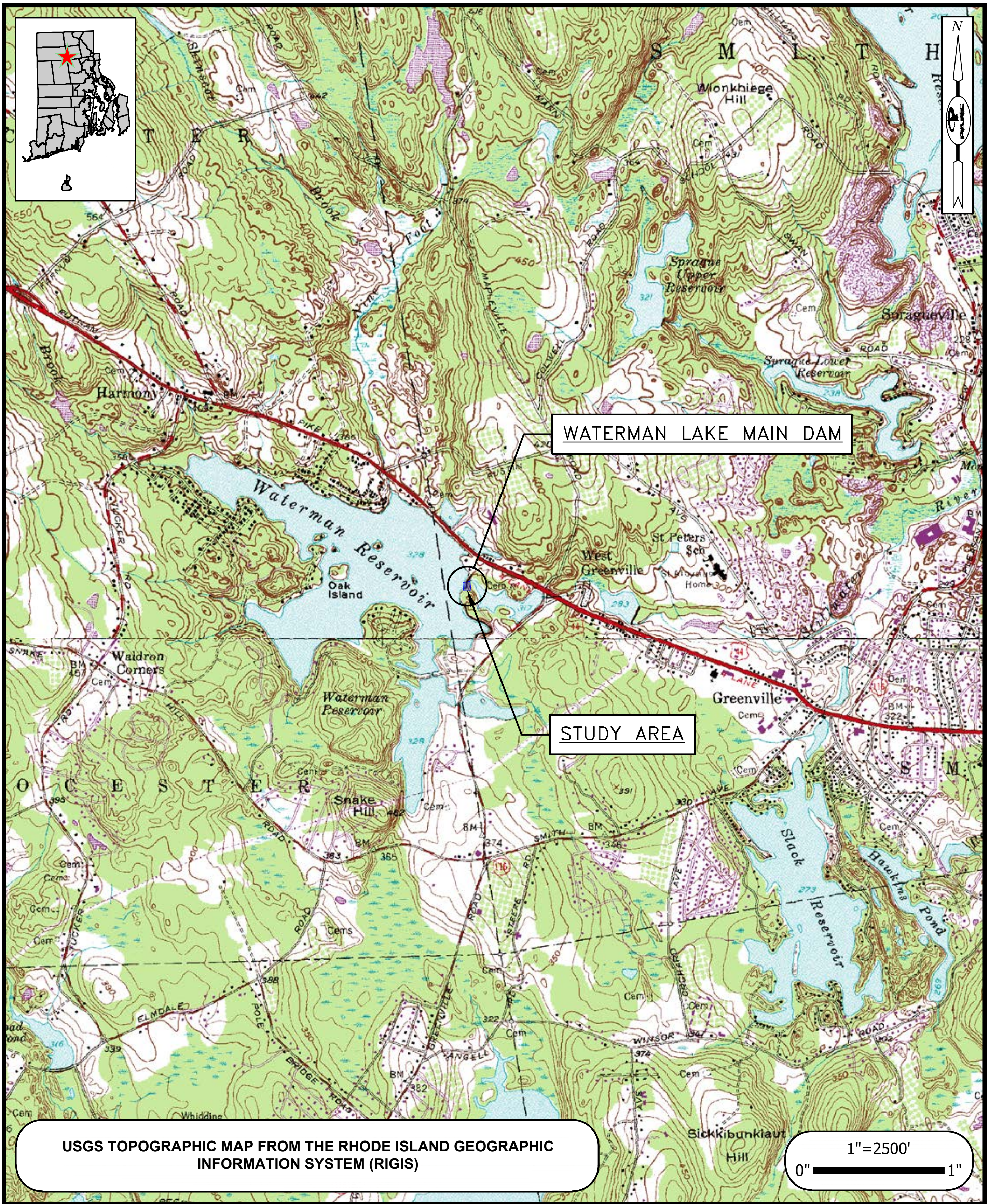
6	Rainfall of 1-inch or more in 24 hrs	• Increase monitoring frequency to 2 times for that week.	• Additional visits as required by additional conditions that develop. • For Conditions 6-7, photographs are not required during additional visits beyond the weekly visit unless a Condition 1-5 is observed. Logging of the additional visit in the monitoring program document should still occur.
7	Rainfall of 1-inch or more over 7 days	• Increase monitoring frequency to 2 times for that week.	
8	During Pond Refill	• Increase monitoring frequency to once per every 6 inches of rise.	
9	After an Earthquake Event	• Contact an engineer to assess the condition • Increase monitoring frequency to daily for entire study area for one week after the event.	

Note that conditions may develop, which after further review by an Engineer or the State, could warrant implementing a drawdown of the impoundment until further corrective action (as outlined in Recommendation #2) can be implemented.

FIGURES

Waterman Lake Dam - Main Dam Study

Smithfield, RI



USGS TOPOGRAPHIC MAP FROM THE RHODE ISLAND GEOGRAPHIC INFORMATION SYSTEM (RIGIS)

PARE CORPORATION

WATERAMN LAKE DAM

RI DAM No. 111

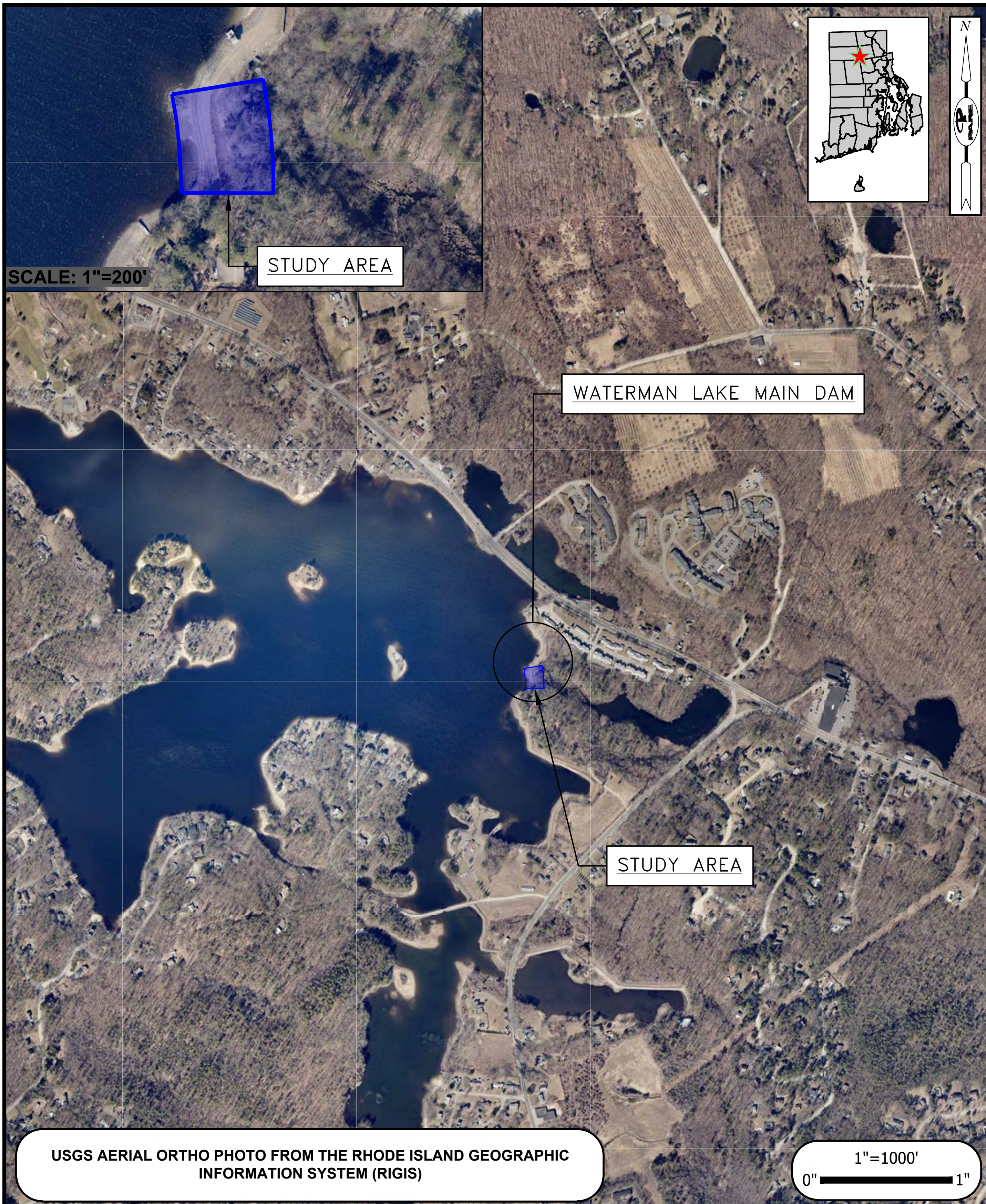
GLOCESTER/SMITHFIELD, RHODE ISLAND

OWNER: CITIZENS FOR THE PRESERVATION OF WATERMAN LAKE, INC

LOCUS PLAN

FEBRUARY 2022

FIGURE 1



PARE CORPORATION

WATERAMN LAKE DAM

RI DAM No. 111

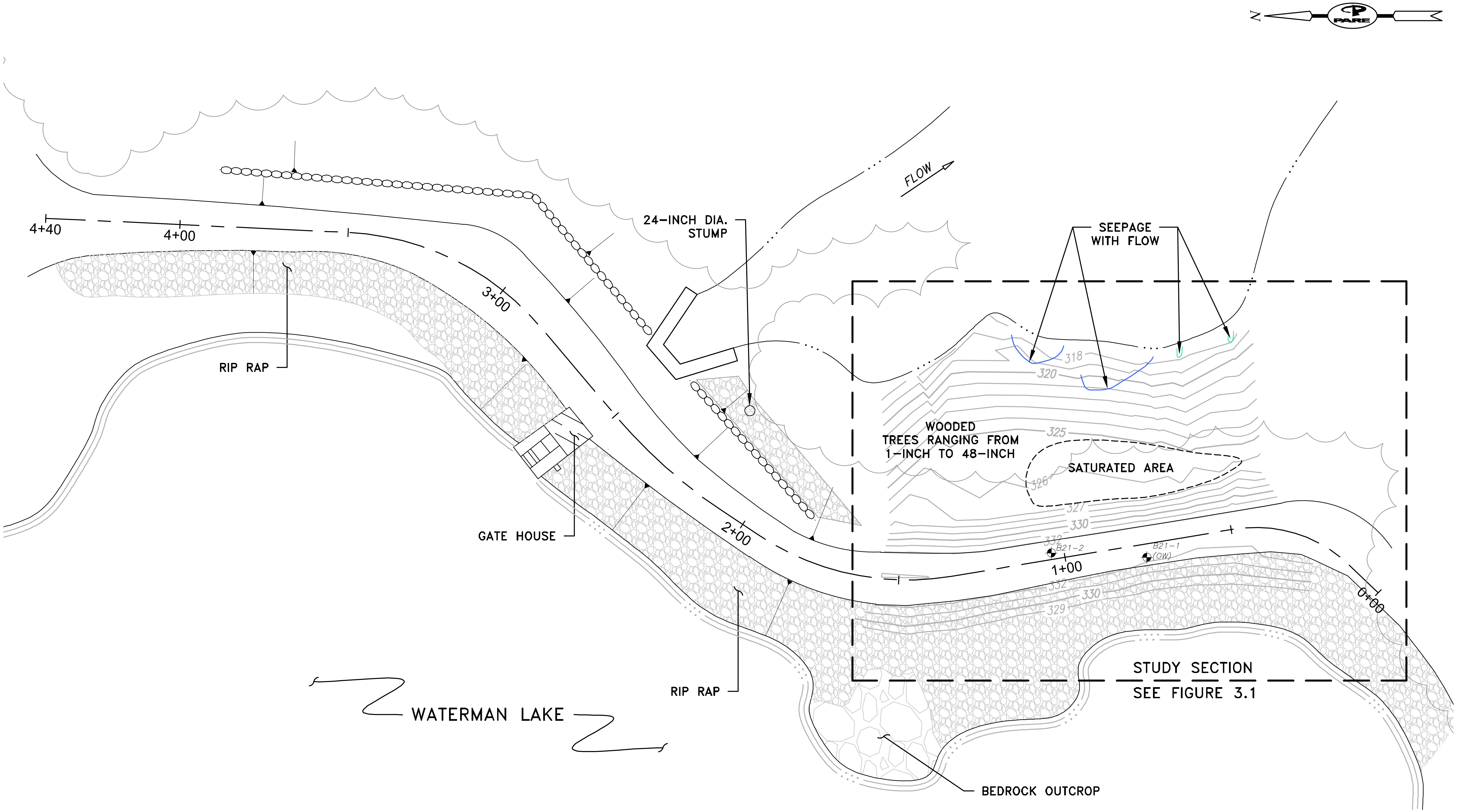
GLOCESTER/SMITHFIELD, RHODE ISLAND

OWNER: CITIZENS FOR THE PRESERVATION OF WATERMAN LAKE, INC

AERIAL PLAN

FEBRUARY 2022

FIGURE 2



MAIN DAM SITE SKETCH

SCALE: 1"=30'±

NOTES:

1. PLAN DEVELOPED FROM NOTES TAKEN DURING PREVIOUS INSPECTION AND AVAILABLE AERIAL IMAGERY FROM RIGIS. INFORMATION IS PROVIDED FOR REFERENCE PURPOSES ONLY.
2. THE STUDY SECTION WERE DEVELOPED FROM A LIMITED SURVEY PERFORMED BY PARE ON OCTOBER 8, 2021 AND DECEMBER 23, 2021.
3. BORINGS WERE COMPLETED BY NORTHERN DRILL SERVICE, INC, AND OBSERVED BY PARE PERSONNEL ON SEPTEMBER 20, 2021.

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SCALE ADJUSTMENT GUIDE
0" 1"
BAR IS ONE INCH ON ORIGINAL DRAWING.

WATERMAN LAKE DAM

RI DAM No. 111

GLOCESTER/SMITHFIELD, RHODE ISLAND

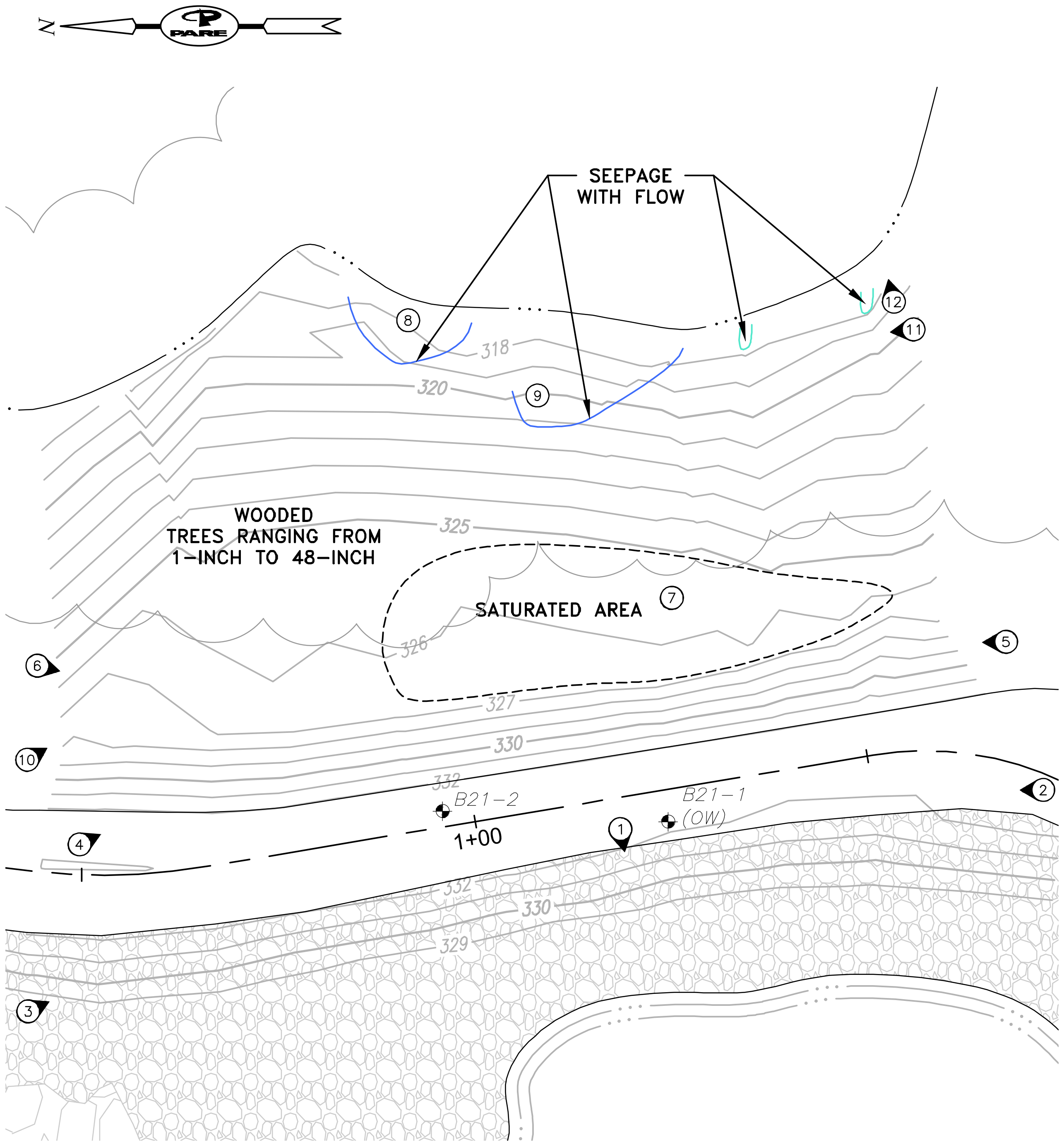
OWNER: CITIZENS FOR THE PRESERVATION OF WATERMAN LAKE, INC.

REVISIONS:

PROJECT NO.:	21137.00
DATE:	FEBRUARY 2022
SCALE:	AS NOTED
DESIGNED BY:	MLP
CHECKED BY:	DRC
DRAWN BY:	LMC
APPROVED BY:	ARO

MAIN DAM SITE SKETCH

FIGURE NO.: 3.0

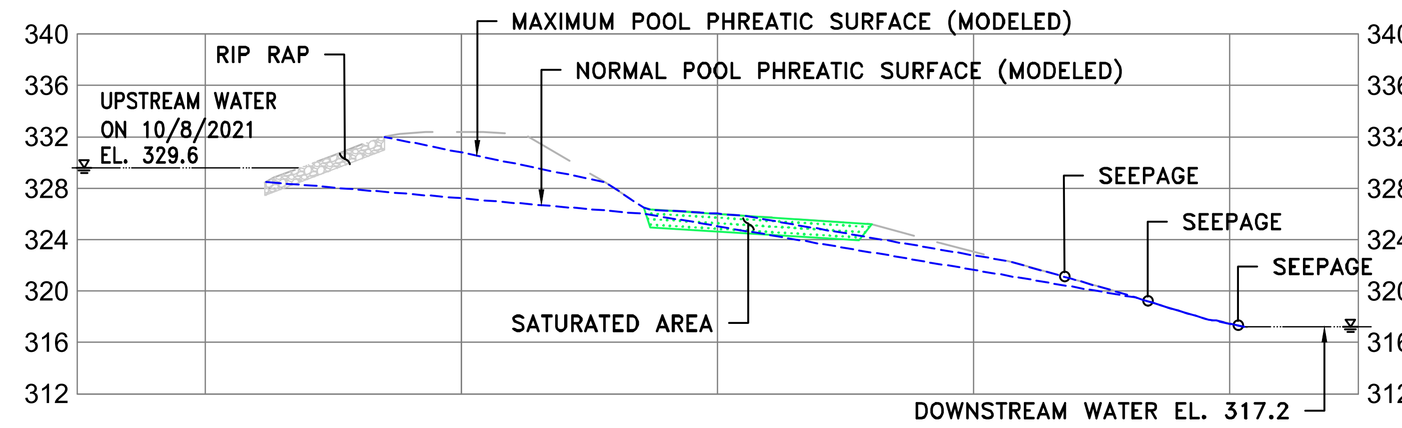


EXISTING STUDY SECTION

SCALE: 1"=20'±

LEGEND

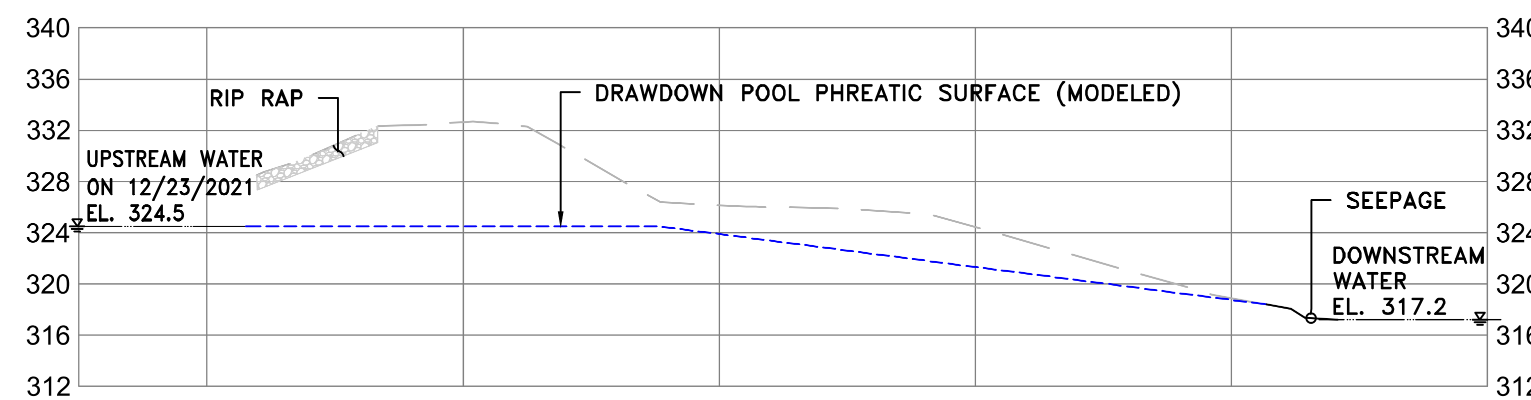
- SEEPAGE OBSERVED AT NORMAL POOL
- SEEPAGE OBSERVED AT NORMAL POOL AND AT DRAWDOWN
- PHREATIC SURFACE (BASED ON SEEPAGE MODELING)
- CONTOUR MAJOR
- CONTOUR MINOR
- PHOTO LOCATION
- B21-1 BORING LOCATION



OBSERVED SEEPAGE DURING NORMAL POOL

TYPICAL DAM SECTION

SCALE: 1"=15'



OBSERVED SEEPAGE DURING DRAWDOWN

TYPICAL DAM SECTION

SCALE: 1"=15'

NOTE:

1. PHREATIC SURFACES DEPICTED WITHIN THE DAM EMBANKMENT ARE THE MODELED LOCATIONS OF THE PHREATIC SURFACE PER THE SEEP W/MODEL, AND ARE LABELED AS SUCH. ALL OTHER NOTED SEEPAGE AREAS OR SATURATED AREAS WERE OBSERVED IN THE FIELD,

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SCALE ADJUSTMENT GUIDE

0" 1"

BAR IS ONE INCH ON ORIGINAL DRAWING.

WATERMAN LAKE DAM

RI DAM No. 111

GLOCESTER/SMITHFIELD, RHODE ISLAND

OWNER: CITIZENS FOR THE PRESERVATION OF WATERMAN LAKE, INC.

REVISIONS:

PROJECT NO.:	21137.00
DATE:	FEBRUARY 2022
SCALE:	AS NOTED
DESIGNED BY:	MLP
CHECKED BY:	DRC
DRAWN BY:	LMC
APPROVED BY:	ARO

STUDY SECTION

FIGURE NO.:

3.1

Y:\JOBS\21 Jobs\21137.00 CPWL-WatermanLakeDamNOVResponse-RI\dwg\Fig 3.0-3.1 - Site-Section.dwg

APPENDIX A:
Site Photographs
Waterman Lake Dam - Main Dam Study
Smithfield, RI

Photo No. 1.: View of the impoundment form the dam crest looking upstream.

Photo No. 2.: Upstream slope of the study area during normal pool.

Photo No. 3.: Upstream view of the study area from the bedcrock outcrop looking right after drawdown of the impoundment.

Photo No. 4.: Crest of the dam from study area looking right.

Photo No. 5.: Downstream slope of the dam embankment from near the right abutment looking left.

Photo No. 6.: Overview of the study area downstream slope from the toe of the dam looking right. Note the steep slope and uneven surface throughout.

Photo No. 7.: Saturated area along the downstream toe of the dam (atop the native terrain) during normal pool (looking right).

Photo No. 8.: Active seepage breakout with iron oxide staining along the downstream toe of the native terrain.

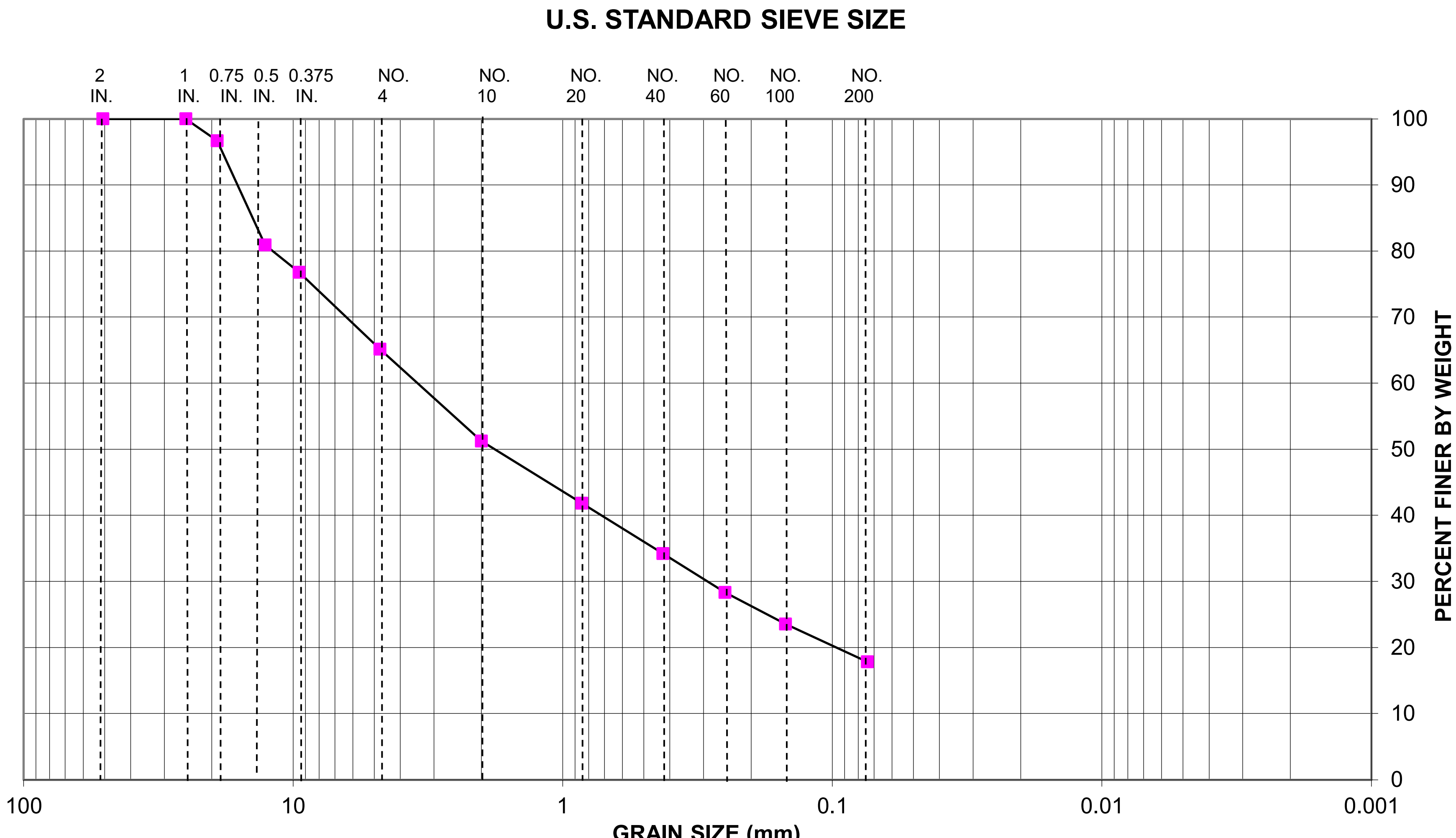
Photo No. 9.: Closer view of the seepage at the downstream toe of the native terrain.

Photo No. 10.: Overview of the downstream area of the dam (native terrain) after drawdown. Note trees ranging between 4-inch to 36-inch diameter growing atop the native terrain and in close proximity to the dam embankment.

Photo No. 11.: Seepage at the downstream toe of the native terrain after drawdown (solid arrows). The dashed arrow indicates the approximate location of the seepage from the slope during normal pool levels (see Photo No. 9)

Photo No. 12.: Overview of the downstream channel.

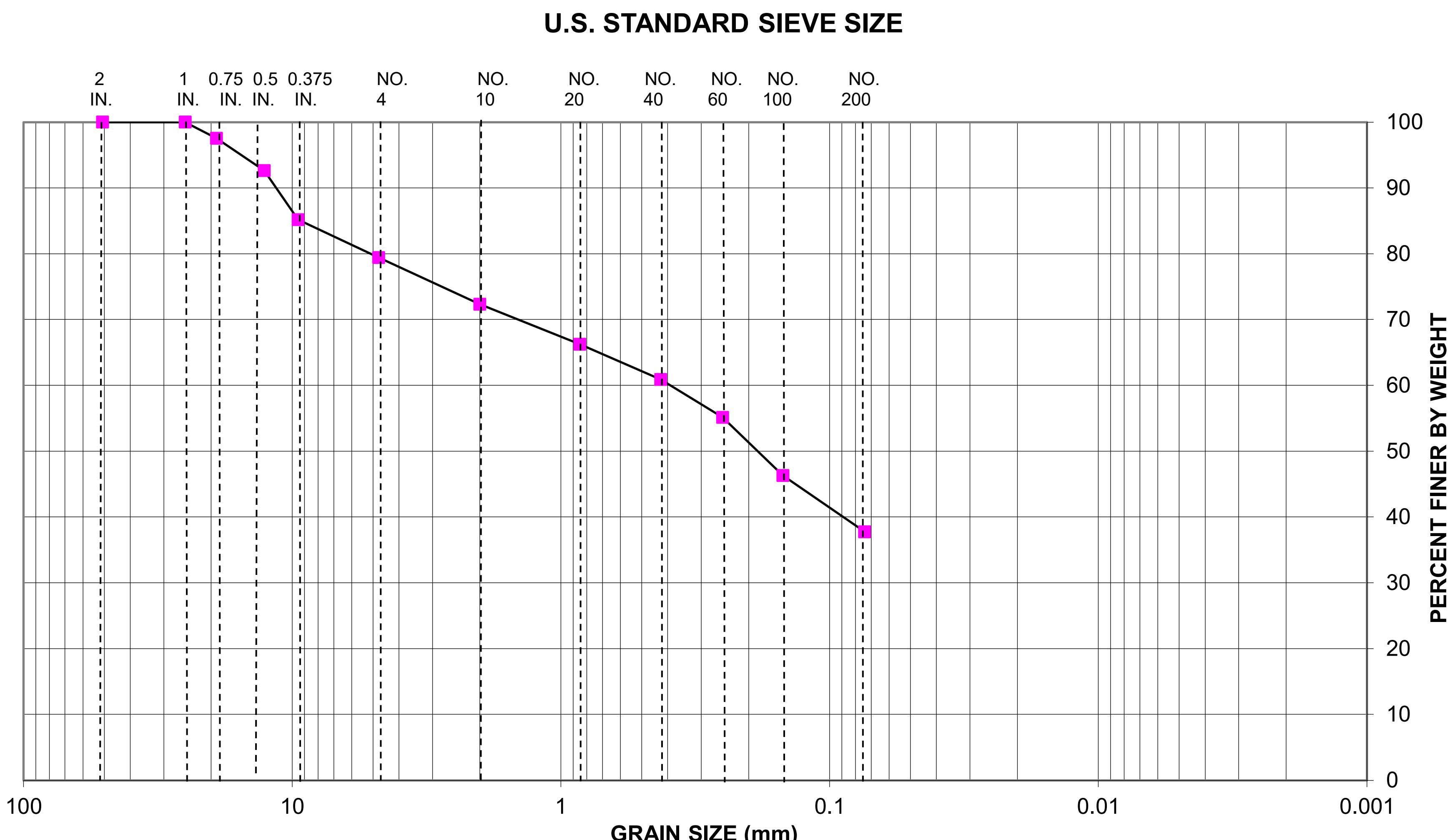
APPENDIX B:
Subsurface Exploration Data
Waterman Lake Dam - Main Dam Study
Smithfield, RI



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

TEST NO.	MATERIAL SOURCE	REMARKS
1	**B21-1 (S-4)** **Depth: 6 feet to 8 feet** **"EMBANKMENT FILL"**	**Burmister Fine to coarse SAND, some fine gravel, little silt.** **Unified Soil Classification System - (SM) Silty SAND with gravel** TESTED BY:__**AKC**__DATE:__**10/12/21**__CHECK BY:____DATE:____.

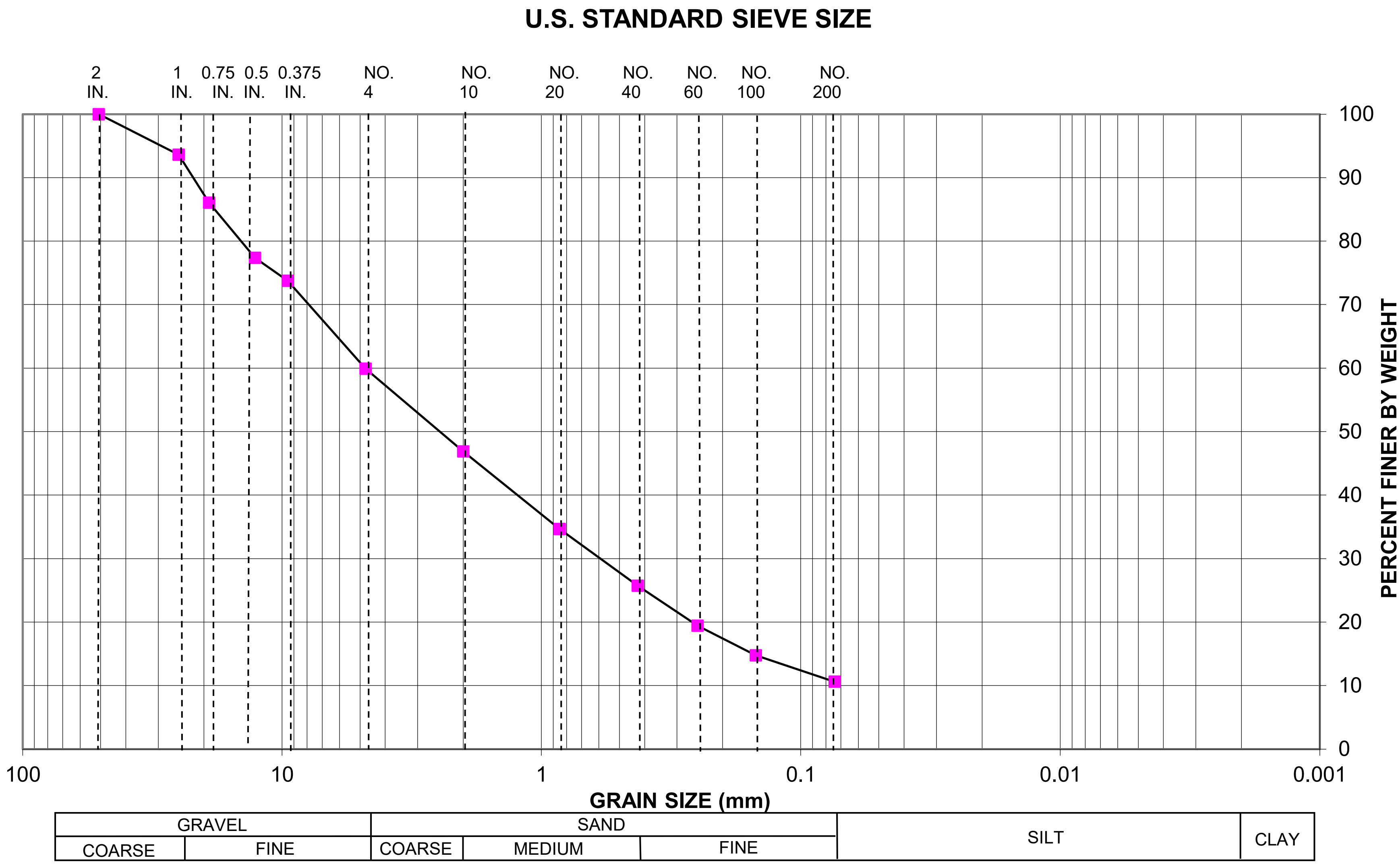
PARE CORPORATION



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

TEST NO.	MATERIAL SOURCE	REMARKS
1	**B21-2 (S-2)** **Depth: 2 feet to 4 feet** **"EMBANKMENT FILL"**	**Burmister - fine to coarse SAND, and silt, trace fine gravel.** **Unified Soil Classification System - (SM) Sand with Fines** TESTED BY: **AKC** DATE: **10/14/21** CHECK BY: **MLP** DATE: 12/8/2021

PARE CORPORATION



TEST NO.	MATERIAL SOURCE	REMARKS
1	**B21-2 (S-7)** **Depth: 14 feet to 16 feet** **"GLACIAL DEPOSITS"**	**Burmister - fine to coarse SAND, some fine gravel, trace coarse gravel, little silt** **Unified Soil Classification System - (GW-GM) Well graded GRAVEL with silt and sand** TESTED BY: **AKC** DATE: **10/14/21** CHECK BY: MLP DATE: 12/8/21

PARE CORPORATION

GENERAL INVESTIGATION NOTES

GENERAL

1. All depths are given in feet measured from the ground surface unless otherwise noted. Depth of angled borings is measured along the axis of the boring.
2. The identification and description of soils is based on visual inspection of the retrieved samples using the Burmister Classification System. Descriptions of boring logs apply only at the specific boring locations and at the time the borings were made. They are not warranted to be representative of subsurface conditions at other locations or times.
3. Water levels are observed at the end of boring (E.O.B.) or/and on a long-term basis through the use of strategically placed observation wells. The indicated levels may not reflect the actual groundwater levels. Fluctuations in groundwater levels can occur due to variations in precipitation, season, tidal fluctuation, adjacent construction activity and construction dewatering systems, and other factors.

SOIL DESCRIPTION

1. The Standard Penetration (SPT) test is performed in general accordance with ASTM D-1586. The standard penetration resistance (N) is defined as the number of blows required to drive a 2-inch O.D., 1 3/8-inch I.D. split-spoon sampler by 12 inches by dropping a 140-lb hammer through a vertical distance of 30 inches. The sampler is normally driven 3 (for 18-inch long sampler) or 4 (for 24-inch long sampler) successive 6-inch increments. The first 6-inch is considered to be a seating drive, therefore the sum of the second and third increments are used in determining the N value.

2. Consistency/Condition

Coarse-Grained Soils	Relative Density (%)	N (blows per foot)
Very loose	0-15	0-4
Loose	15-35	4-10
Medium dense	35-65	10-30
Dense	65-85	30-50
Very dense	85-100	>50

Fine-Grained Soils	Unconfined Compressive Strength, q_u (tsf)	N (blows per foot)	Field Identification
Very Soft	<0.25	0-2	Exudes between fingers when squeezed in hand
Soft	0.25-0.50	2-4	Molded by light finger pressure
Medium	0.50-1.00	4-8	Molded by strong finger pressure
Stiff	1.00-2.00	8-15	Indented by thumb
Very Stiff	2.00-4.00	15-30	Indented by thumbnail
Hard	>4.00	>30	Difficult to indent by thumbnail

Grain Size

Boulders – >12 in.
Cobbles – 3 in. - 12 in.
Gravel – Coarse, ¾ in. - 3 in.
– Fine, 0.19 in. (#4) to ¾ in.
Sand – Coarse, 0.079 in. (#10) to 0.19 in. (#4)
– Medium, 0.017 in. (#40) to 0.079 in. (#10)
– Fine, 0.0029 in. (#200) to 0.017 in. (#40)
Silt – 0.0002 in. to 0.0029 in. (#200)
Clay - <0.0002 in.

Descriptive Adjective	
Trace	0-10%
Little	10-20%
Some	20-35%
And	35-50%

Percent by Weight

ROCK DESCRIPTION

1. Core recovery is the total length of rock core recovered from a core run divided by the length of the run, expressed as a percentage.

2. Rock Quality Designation (RQD) is the total length of hard, sound pieces of rock core greater than 4-inches from a core run divided by the length of the run, expressed as a percentage.

RQD (%)	Description	Approximate Equivalent Fracture Spacing (feet)	
0-25	Very Poor	Very close	(<0.2)
25-50	Poor	Close	(0.2-1)
50-75	Fair	Moderately wide	(1-3)
75-90	Good	Wide	(3-10)
90-100	Excellent	Very wide	(>10)

3. "Weathering" refers to the degree of alteration observed in the rock core, which is produced by chemical and/or mechanical processes.

Grade	Symbol	Recognition
Fresh	F	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	WS	Slight discoloration inwards from open fractures, otherwise similar to F.
Moderately Weathered	WM	Discoloration throughout. Weaker minerals such as feldspar decomposed. Strength somewhat

		less than fresh rock but cores cannot be broken by hand or scraped by knife. Texture preserved.
Highly Weathered	WH	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Completely Weathered	WC	Minerals decomposed to soil but fabric and structure preserved (Saprolite). Specimens easily crumbled or penetrated.
Residual Soil	RS	Advance state of decomposition resulting in plastic soils. Rock fabric and structure completely destroyed. Large volume change.

4. "Hardness" is an estimate of the rock strength that is a function of lithology and the degree of weathering.

Class	Hardness	Field Test	Approximate Range of Uniaxial Compression Strength kg/cm^2 (tons/ft^2)
I	Extremely Hard	Many blows with geologic hammer required to break intact specimen.	>2,000
II	Very Hard	Hand held specimen breaks with hammer end of pick under more than one blow.	2,000 – 1,000
III	Hard	Cannot be scraped or peeled with knife, hand held specimen can be broken with single moderate blow with pick.	1,000 – 500
IV	Soft	Can just be scraped or peeled with knife. Indentation 1 mm to 3 mm show in specimen with moderate blow with pick.	500 – 250
V	Very Soft	Material crumbles under moderate blow with sharp end of pick and can be peeled with a knife, but is too hard to hand-trim for triaxial test specimen.	250 – 10

5. Discontinuity Descriptions

Rock Continuity: Any break in a rock whether or not it has undergone relative displacement.

Extremely Fractured – Drill core stem less than 1 in.
Moderately Fractured – Drill core stem 1 in. to 4 in.
Slightly Fractured – Drill core stem 4 in. to 8 in.
Sound – Drill core stem greater than 8 in.

Texture: Terminology used to identify size, shape and arrangement of constituent elements.

Amorphous – Too small to be seen with naked eye.
Fine Grained – Barely seen with naked eye.
Medium Grained – Barely seen with naked eye to 1/8 in.
Coarse Grained – $^1/_8$ in to ¼ in.
Very Coarse Grained > ¼ in.

Discontinuities: Surface representing breaks or fractures separating the rock moss into discrete units.

Crack – A partial or incomplete fracture.
Joint – A simple fracture along which no shear displacement has occurred. May form joint sets.
Shear – A fracture along which differential movement has taken place parallel to the surface sufficient to produce slickendsides or polishing. May be accompanied by a zone of fractured rock up to a few inches wide.
Fault – A major fracture along which there has been appreciable displacement and accompanied by gouge and/or a severely fractured adjacent zone.
Shear or Fault Zone – A band or zone of parallel, closely spaced shears or faults.

Fractures, Bedding, and Foliation, Spacing and Attitude:

Fractures	Bedding and Foliation	Spacing	Attitude	Dip Angle
Very Close	Very Thin	Less than 2 in.	Horizontal	0 – 5
Close	Thin	2 in. – 1 ft.	Shallow or low angle	5 – 35
Moderate	Medium	1 ft. – 3 ft.	Moderately dipping	35 – 55
Wide	Thick	3 ft. – 10 ft.	Steep or high angle	55 – 85
Very Wide	Very Thick	More than 10 ft.	Vertical	85 – 90

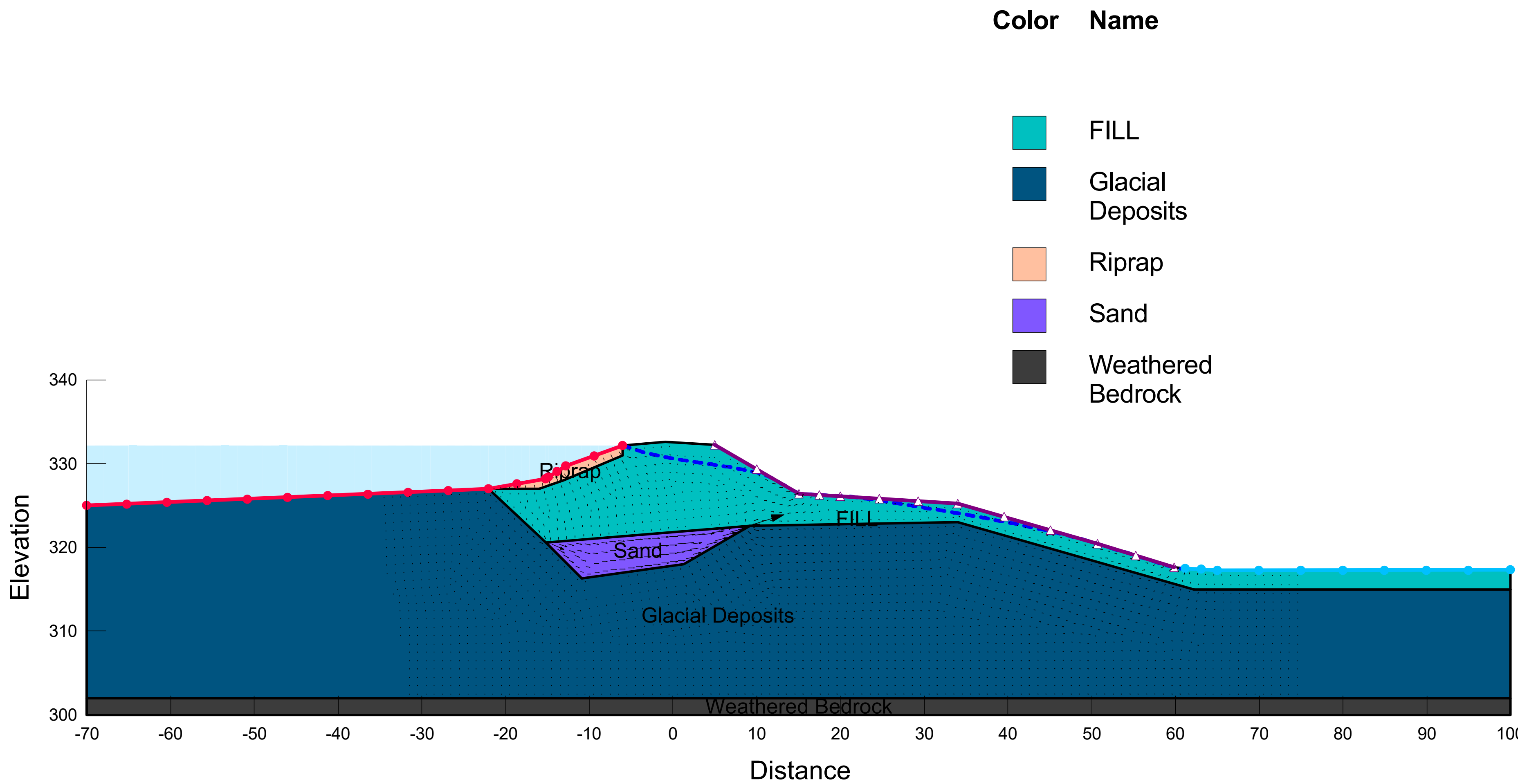
DRILLING CODES

HSA	Hollow Stem Auger	SS	Split Spoon Sample
C/A	Casing Advancement	AS	Auger Sample
BX	Rock Cored with BX Core Barrel (Produces 1 $^5/_8$"-diameter core)	ST	Shelby Tube Sample
		WS	Washed Sample
NX	Rock Cored with NX Core Barrel (Produces 2 $^1/_8$"-diameter core)	NR	No Recovery

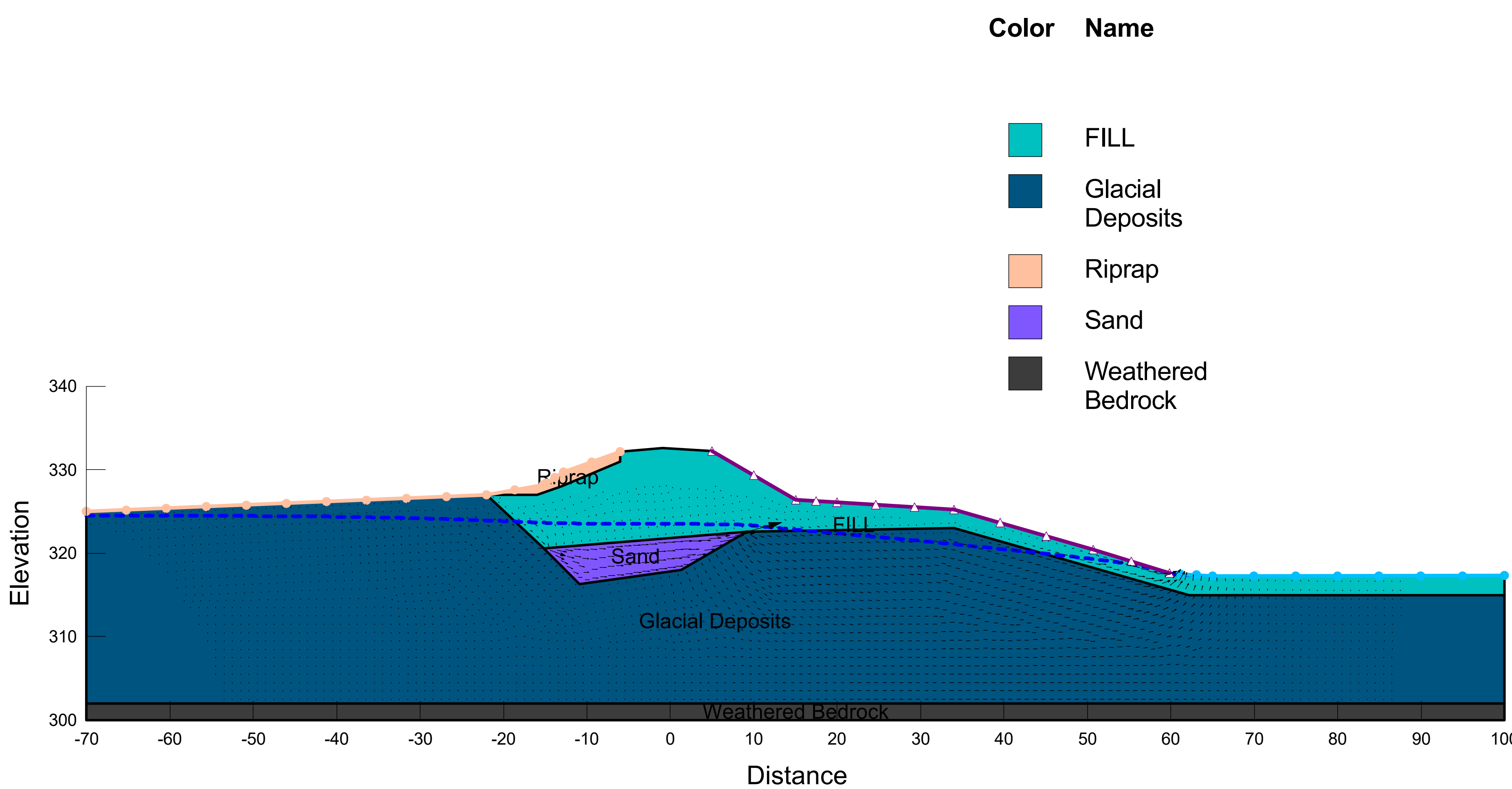
APPENDIX C:
Seepage and Slope Stability Analyses Results
Waterman Lake Dam - Main Dam Study
Smithfield, RI

EXISTING CONDITION

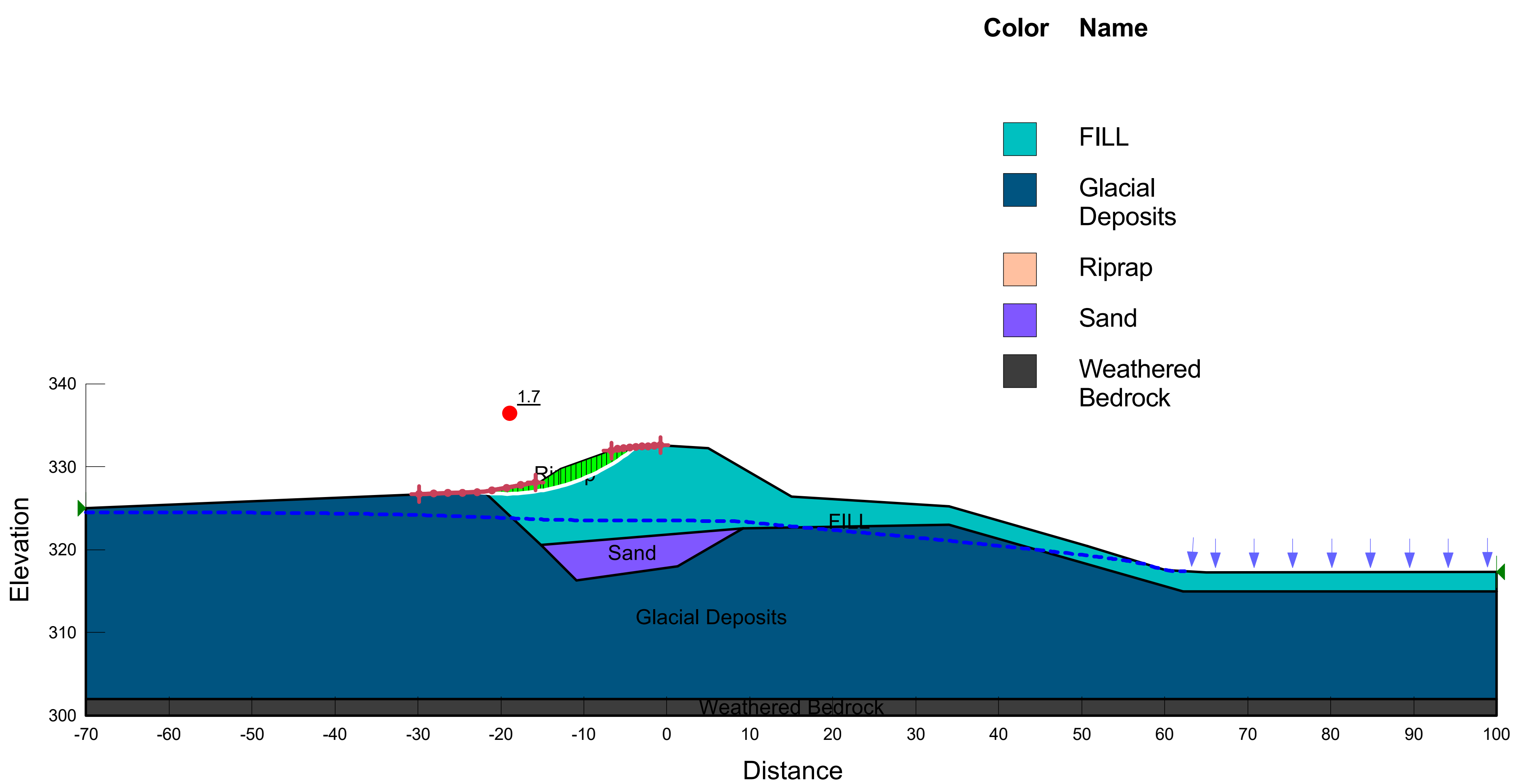
Existing Condition Maximum Pool



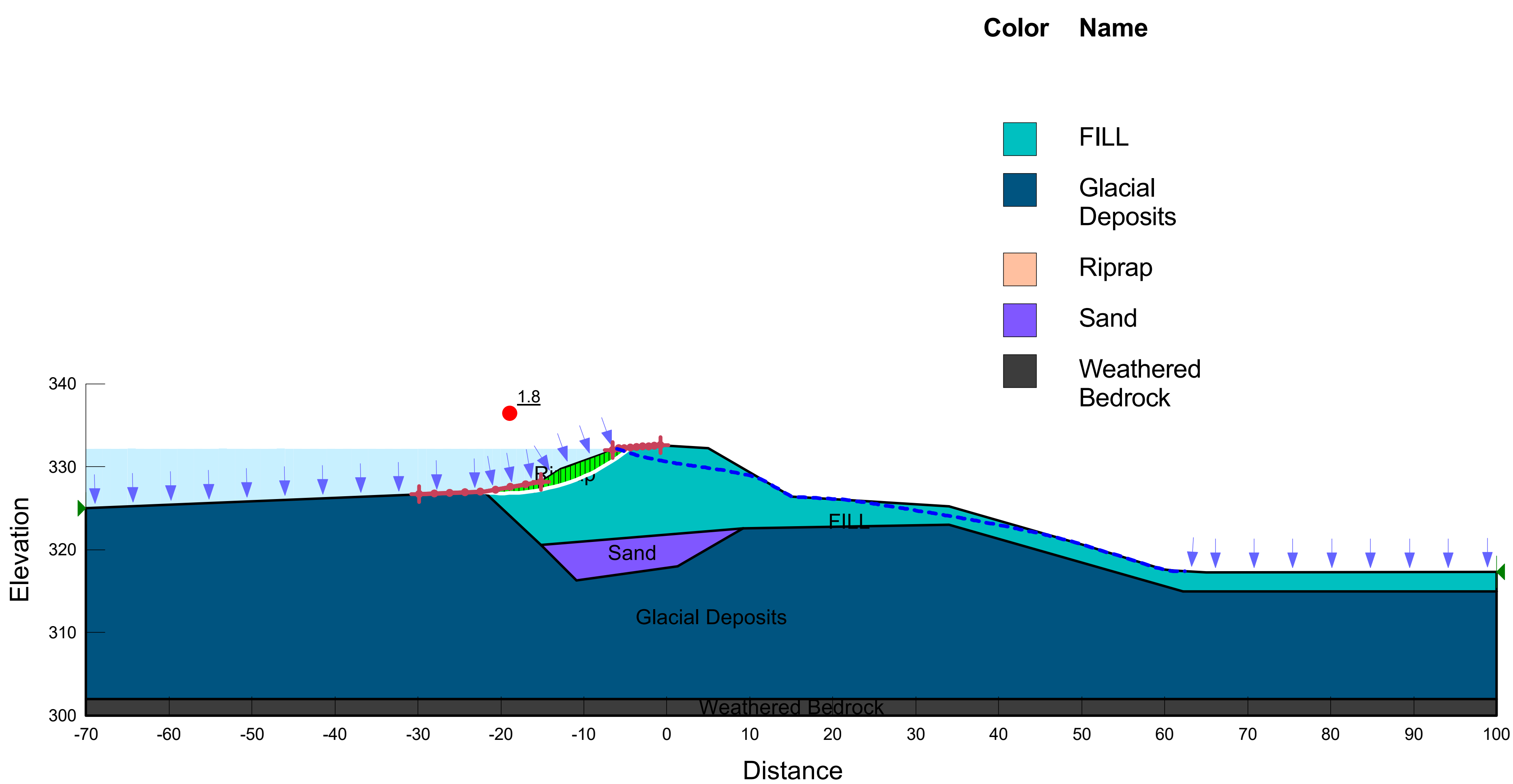
Existing Condition - Drawdown from Max



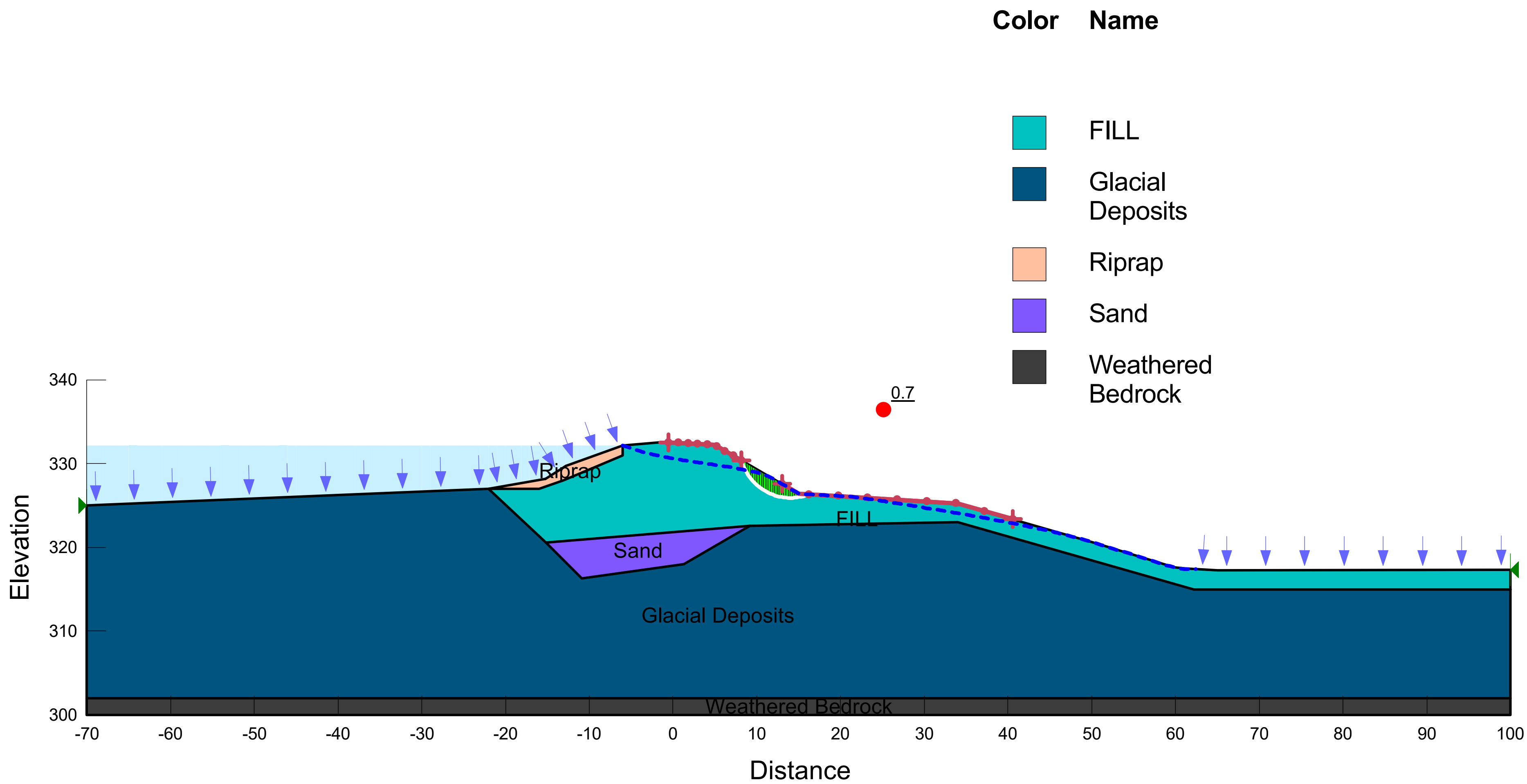
Drawdown Max - US Slope



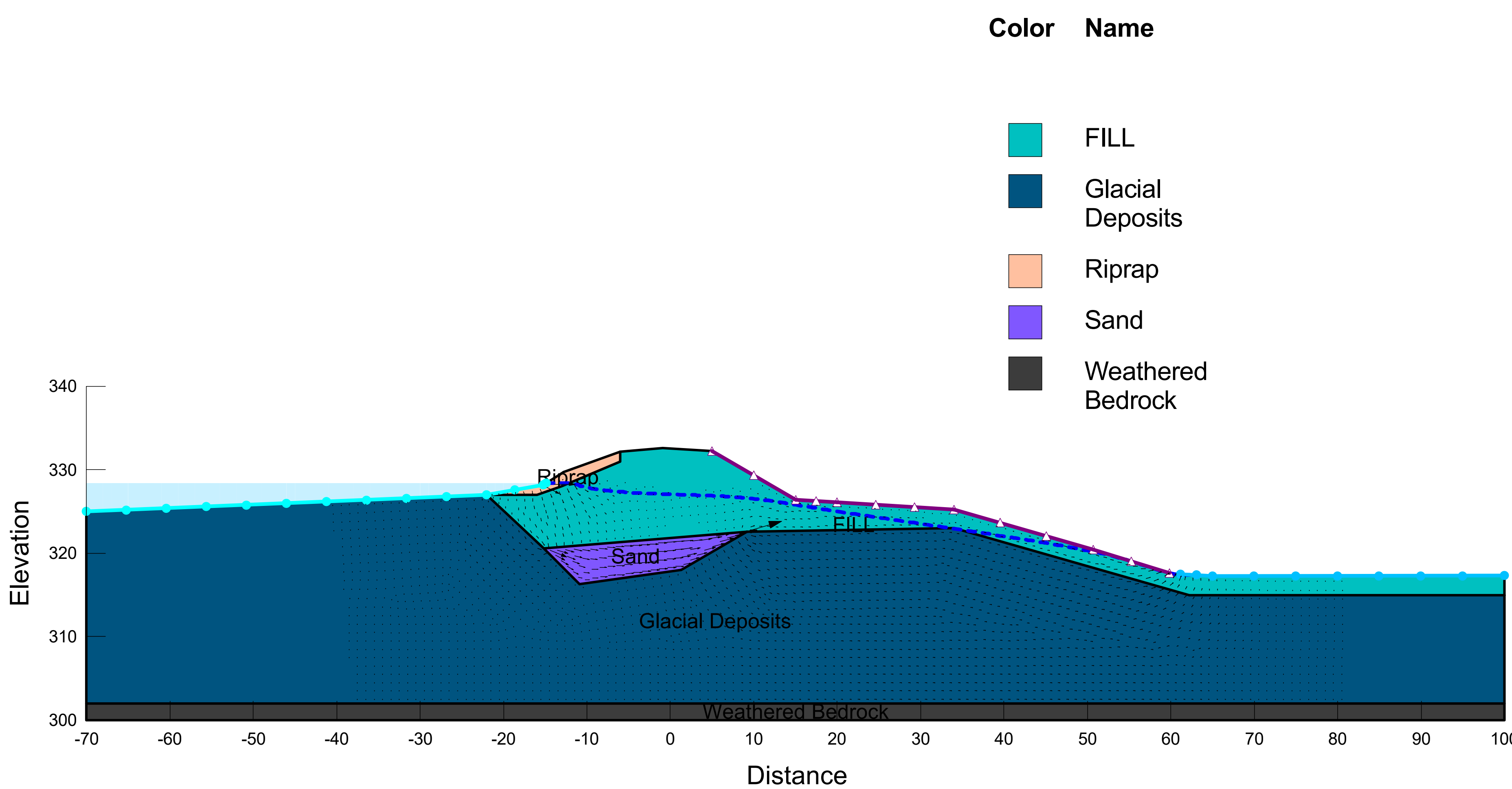
Maximum Pool Upstream Slope



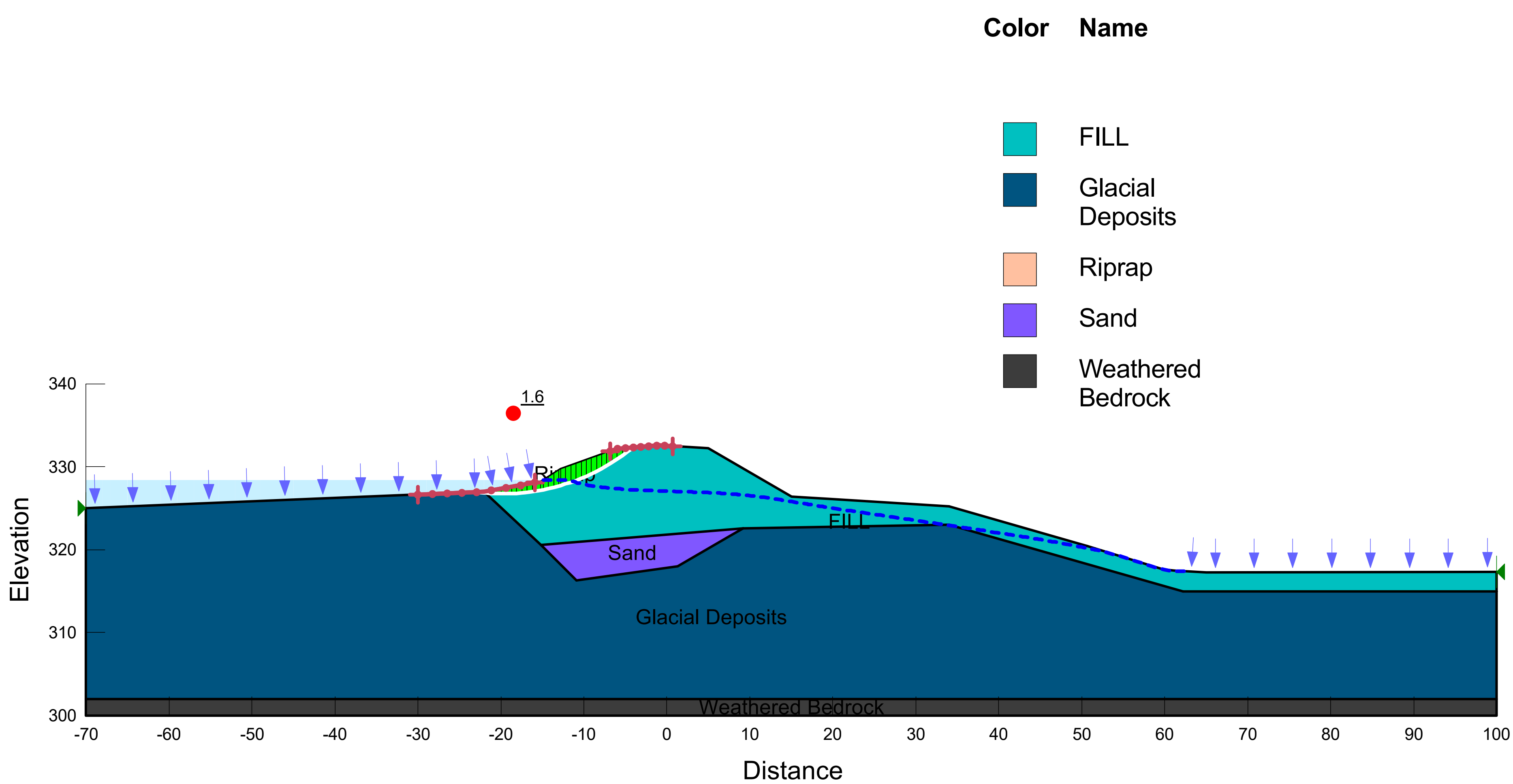
Maximum Pool Downstream Slope



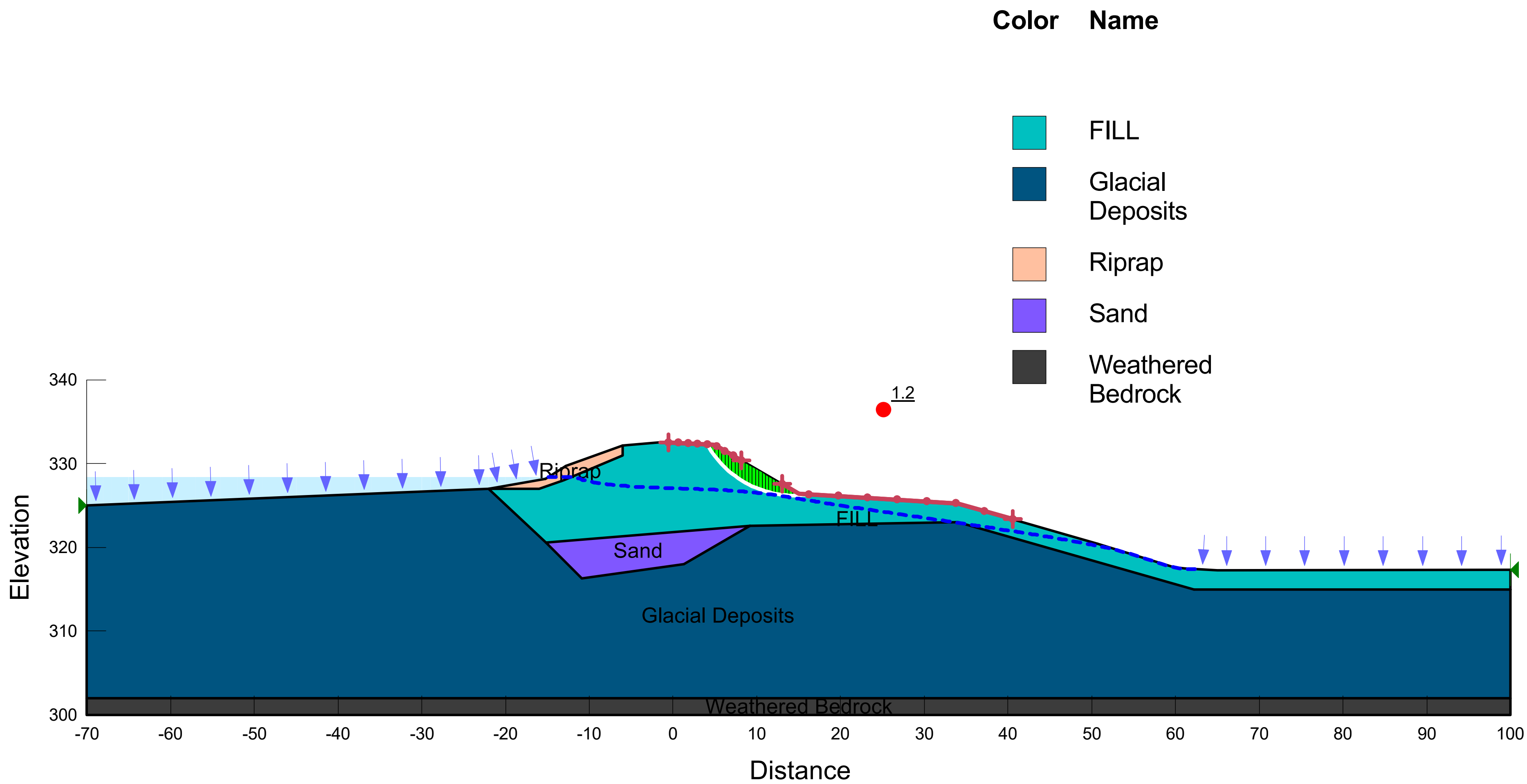
Existing Condition - Normal Pool



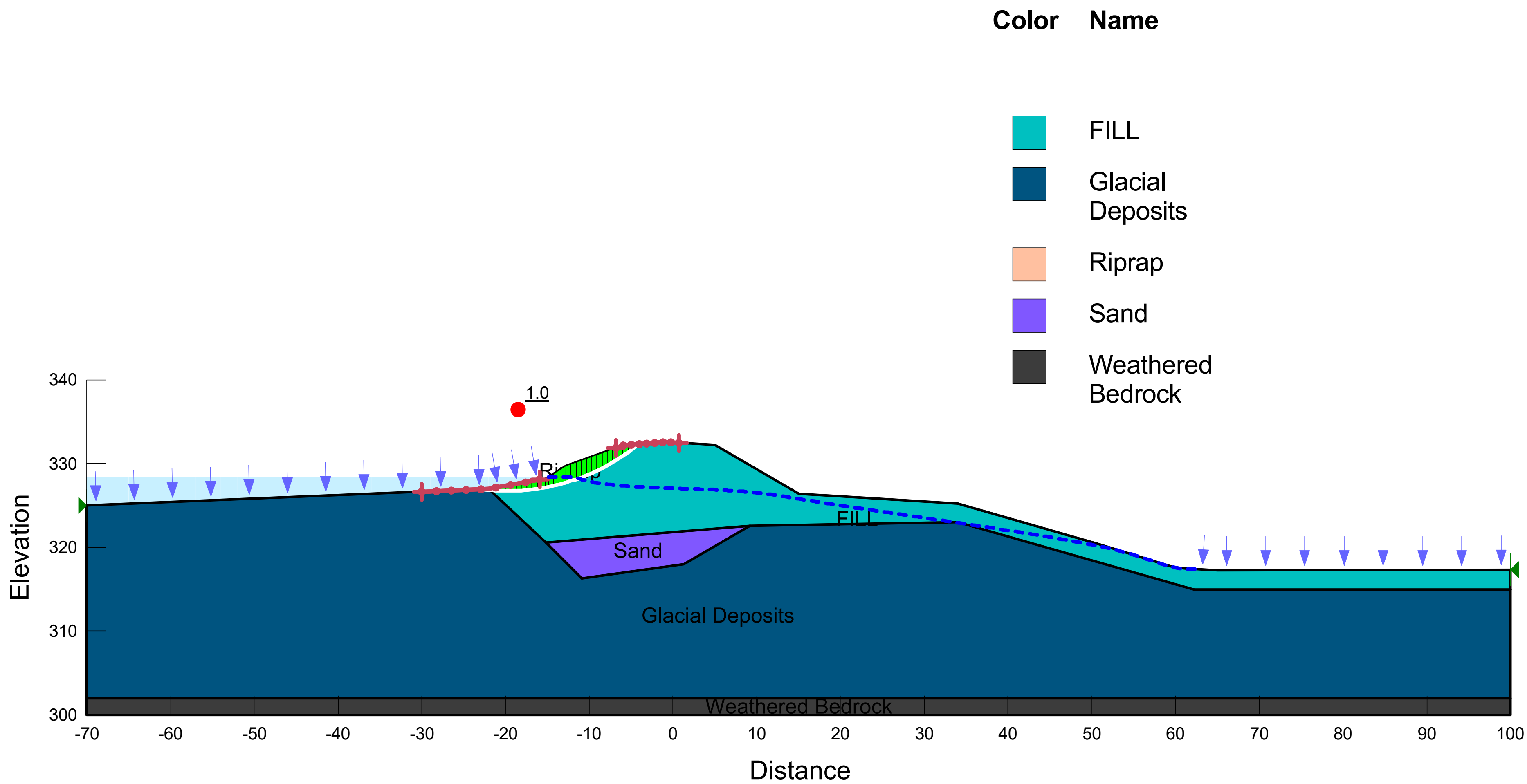
Normal Pool Upstream Slope



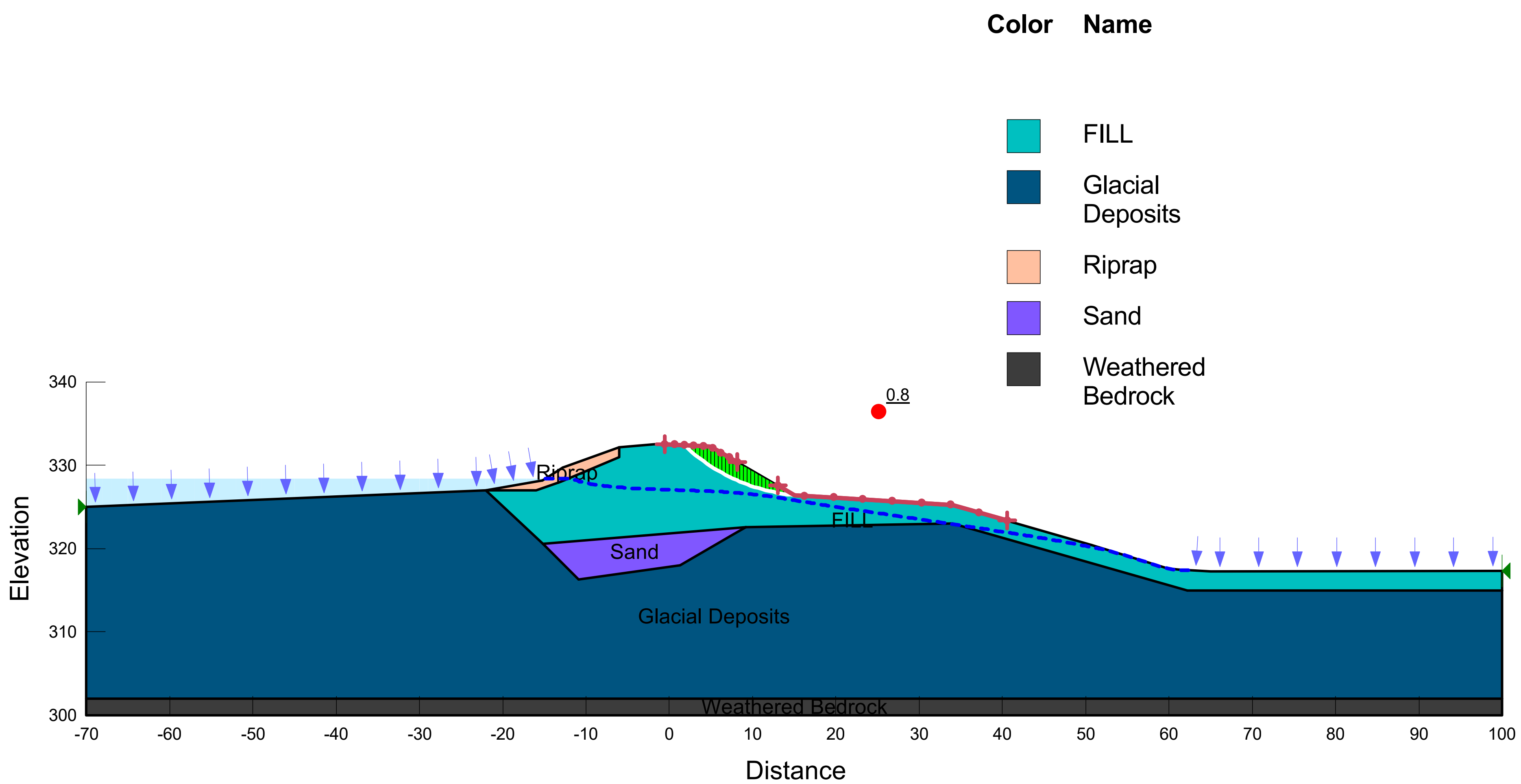
Normal Pool Downstream Slope



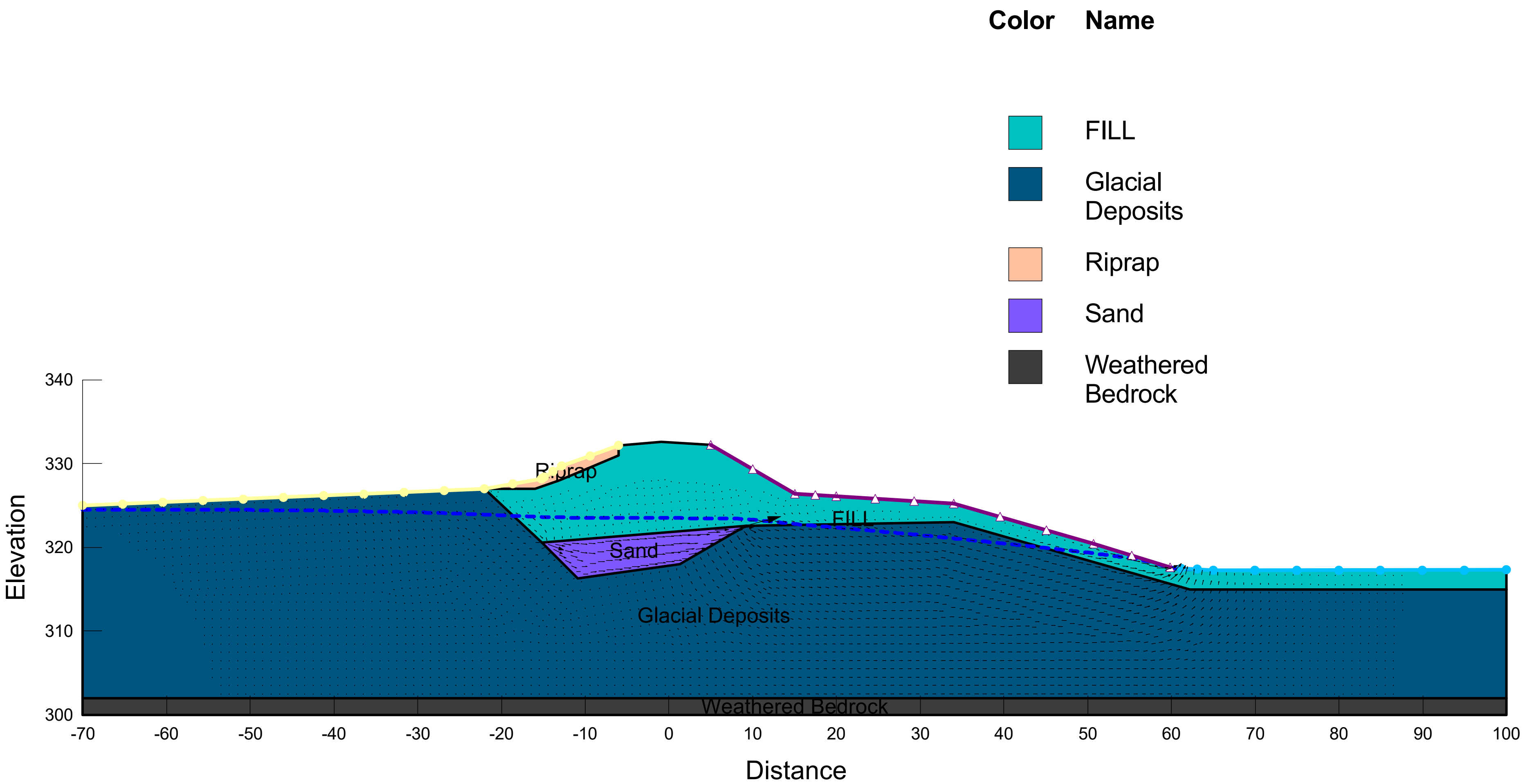
Normal Pool Upstream Slope Quake



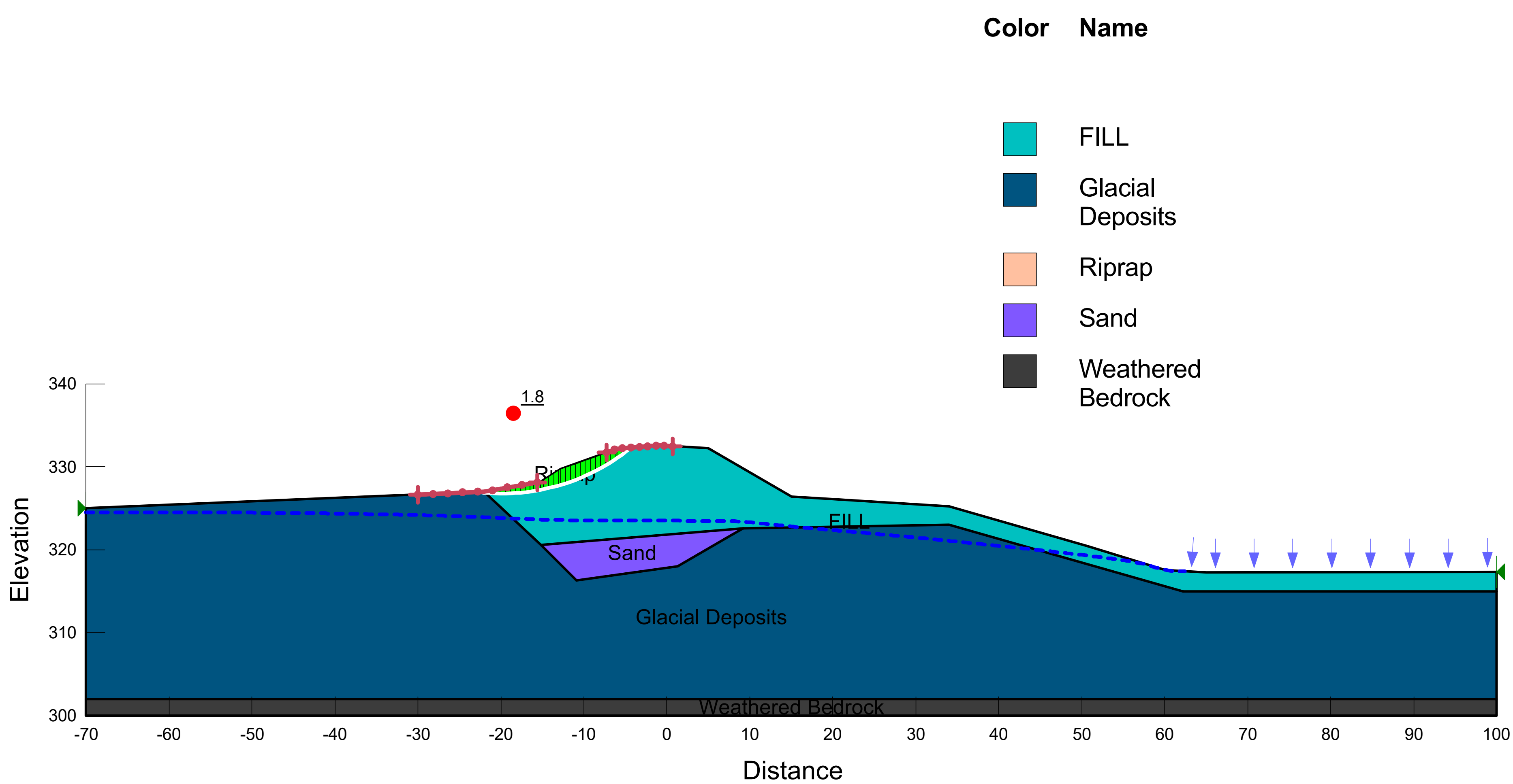
Normal Pool Downstream Slope Quake



Existing Condition - Drawdown from Normal Pool

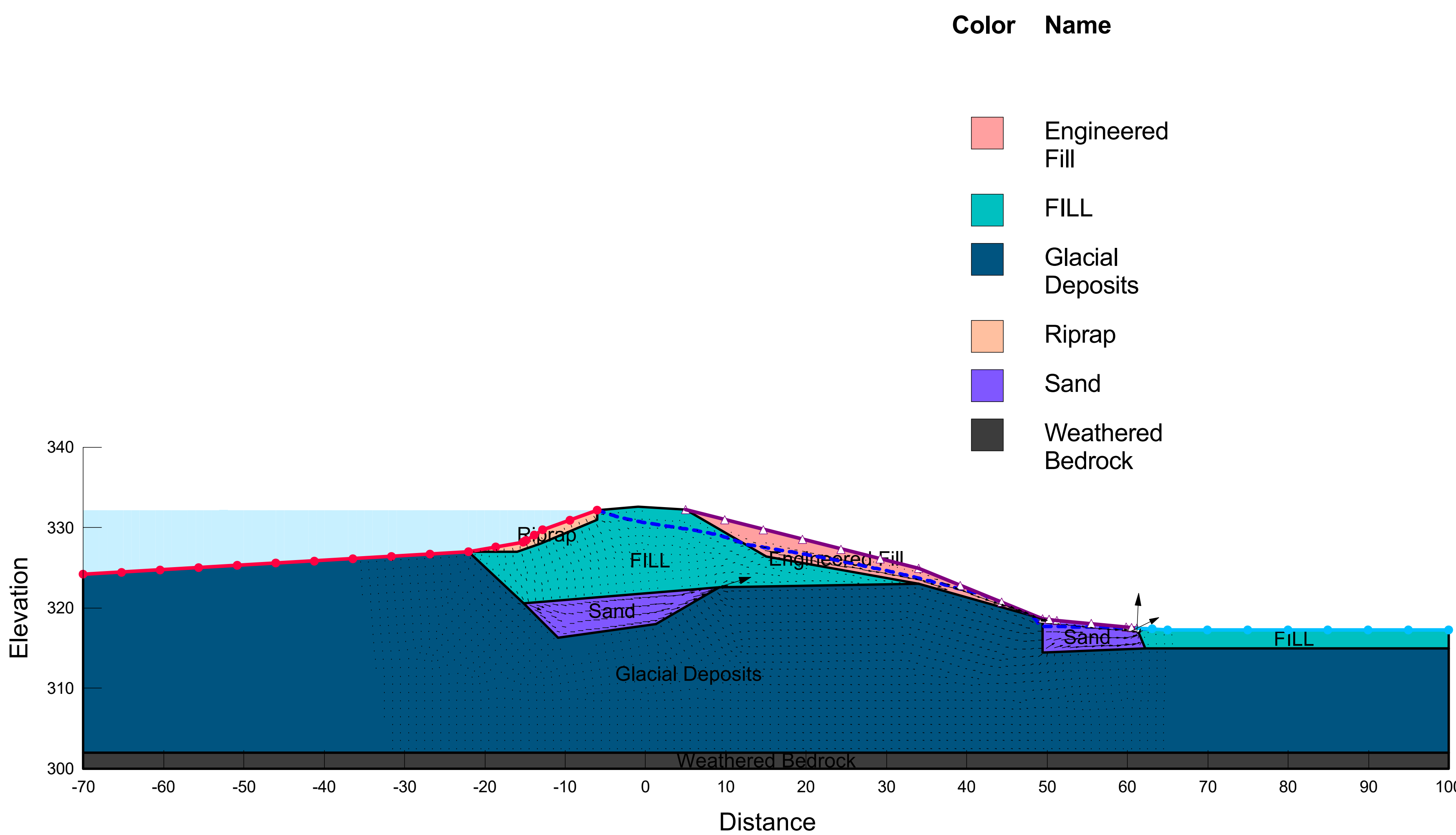


Existing Condition Drawdown NP - US Slope

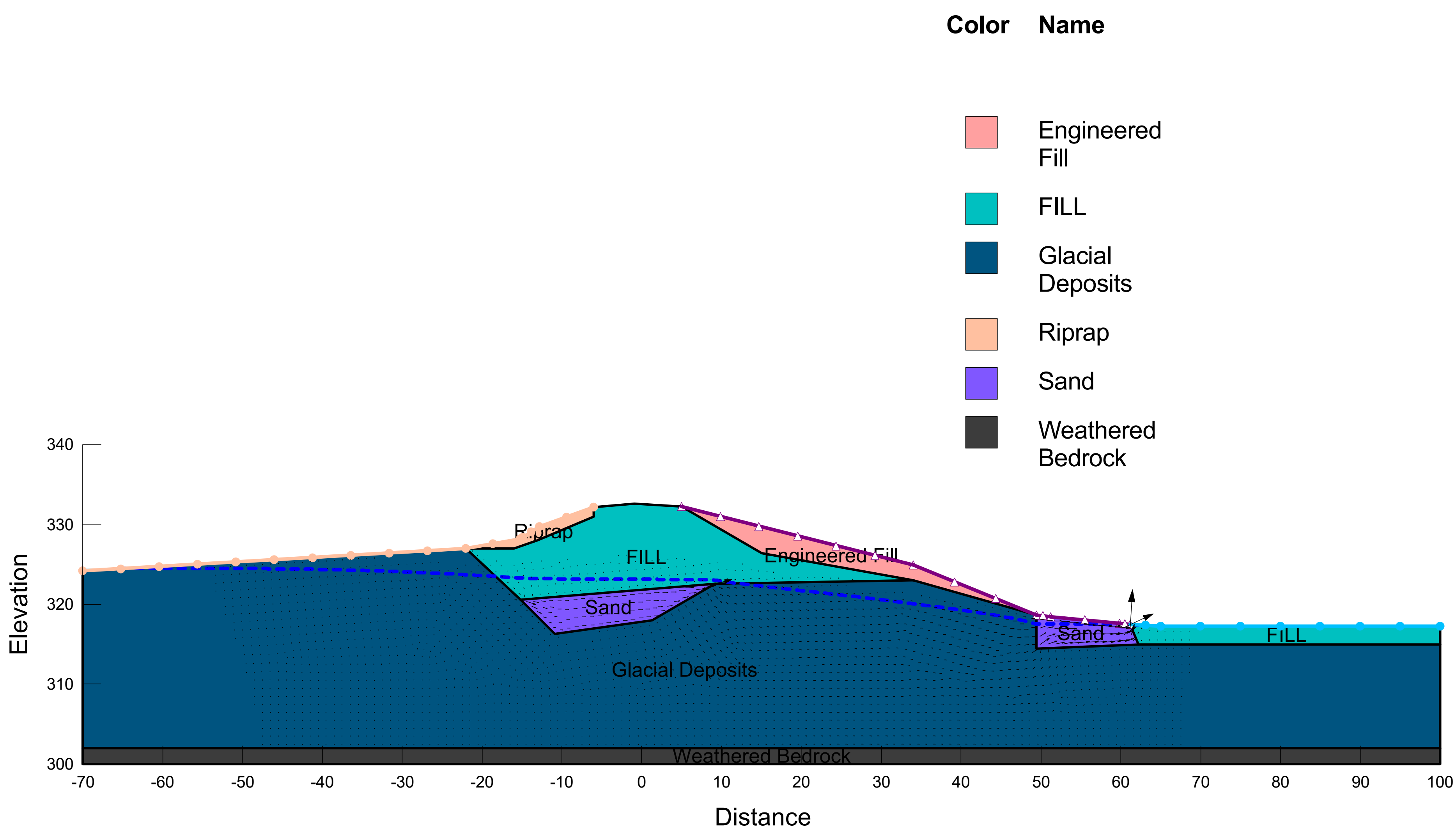


PROPOSED ROCK TOE

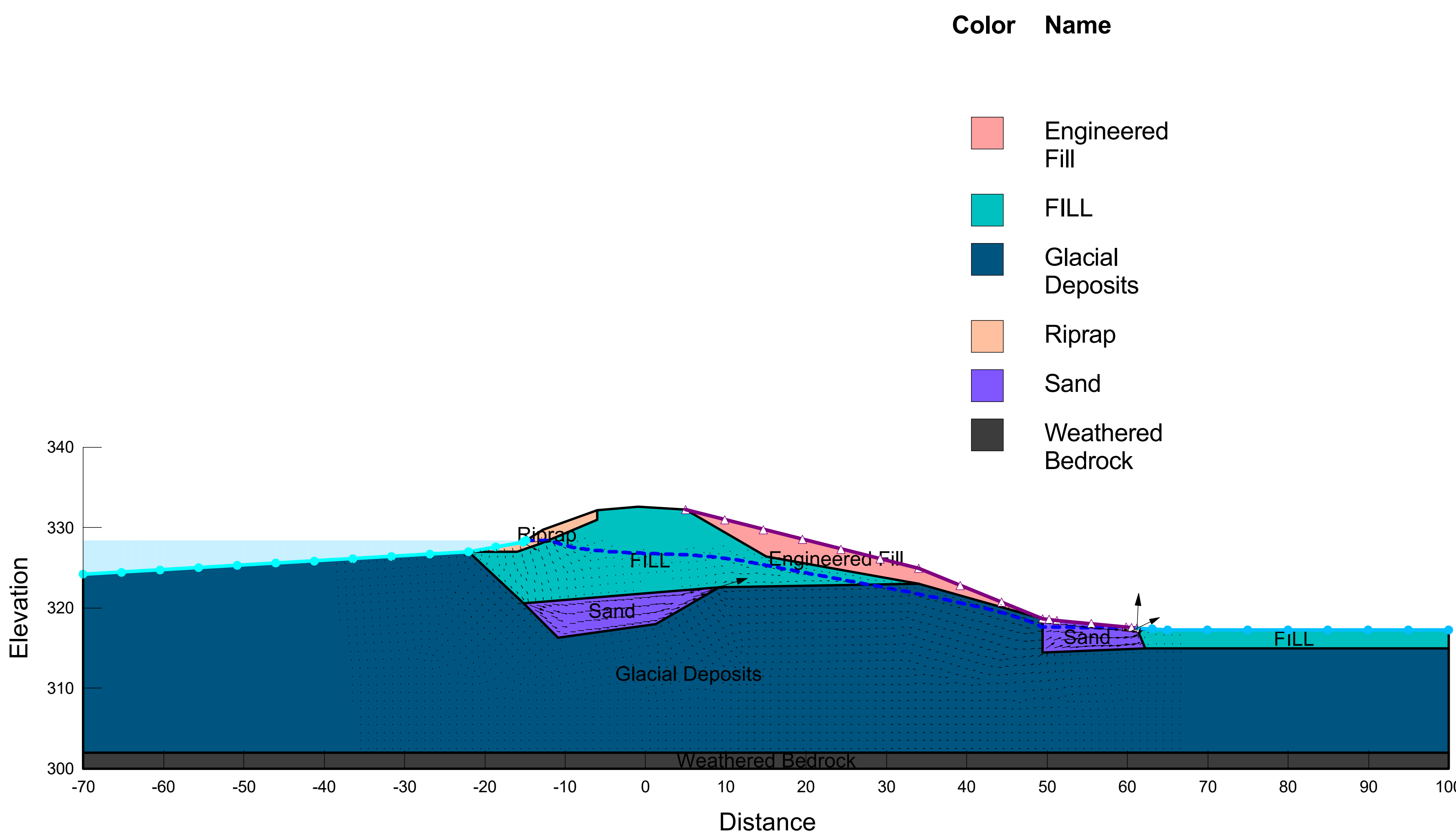
Proposed Rock Toe - Maximum Pool



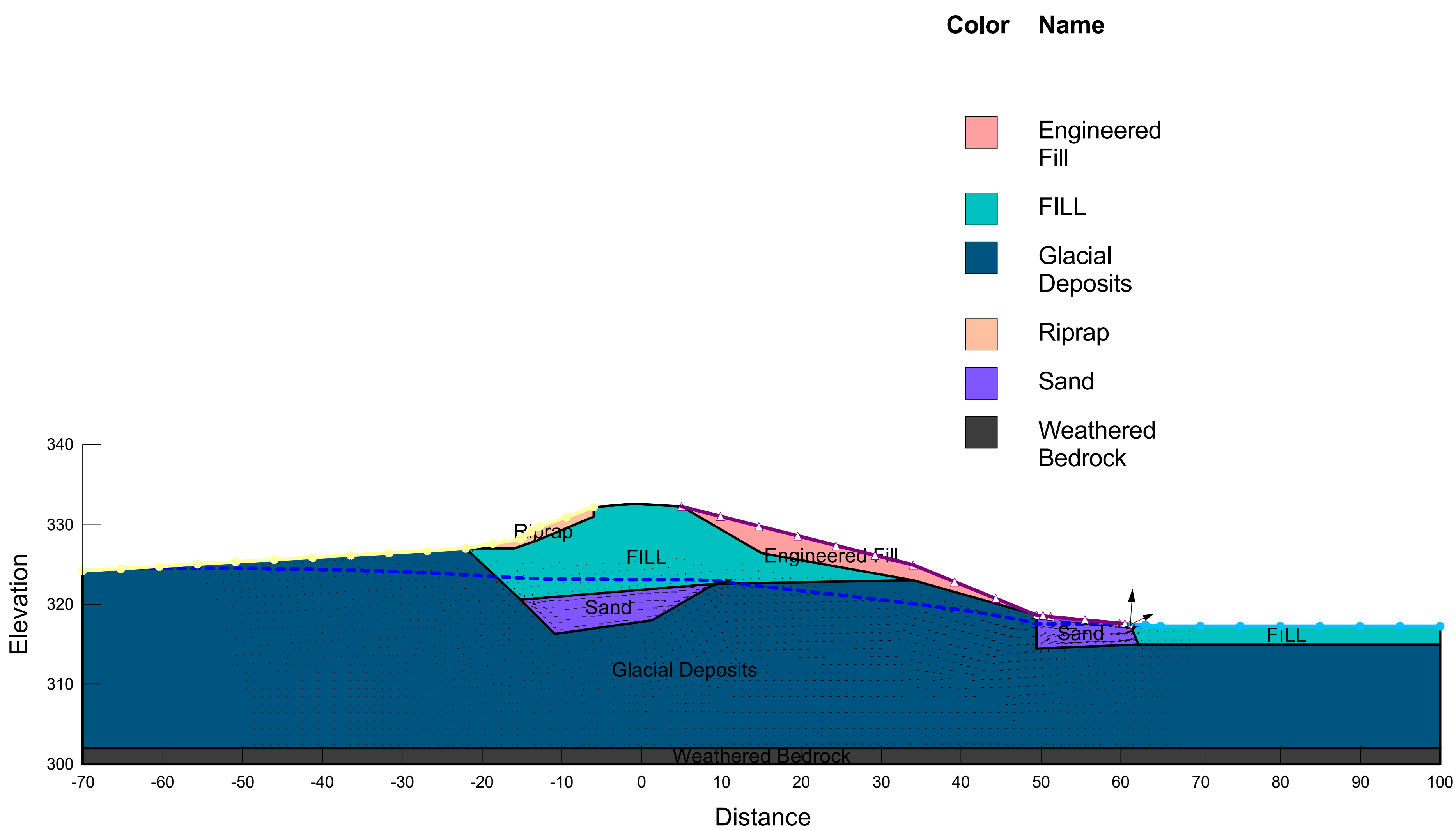
Proposed Rock Toe - Drawdown from Max



Proposed Rock Toe - Normal Pool

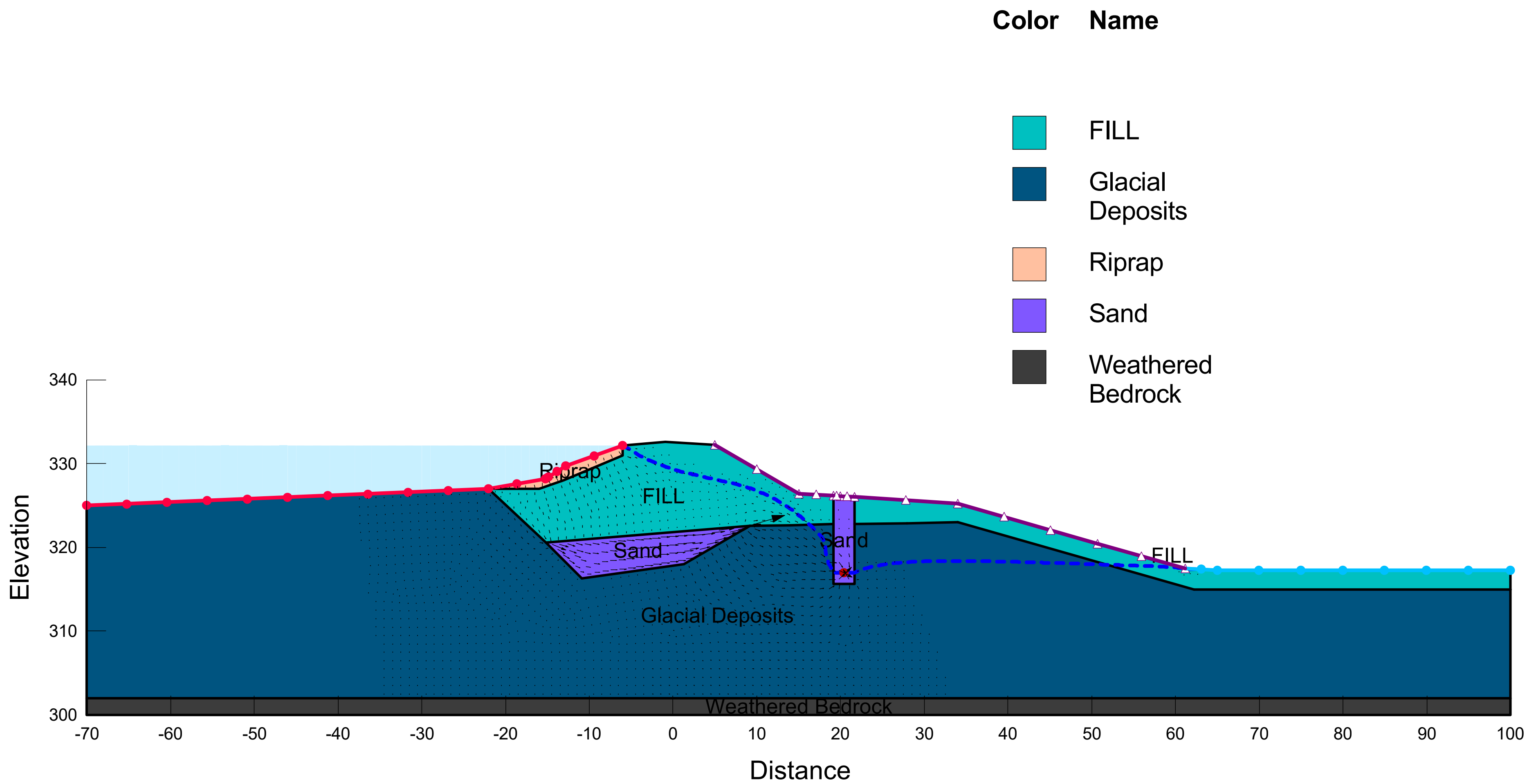


Proposed Rock Toe - Drawdown from Normal Pool

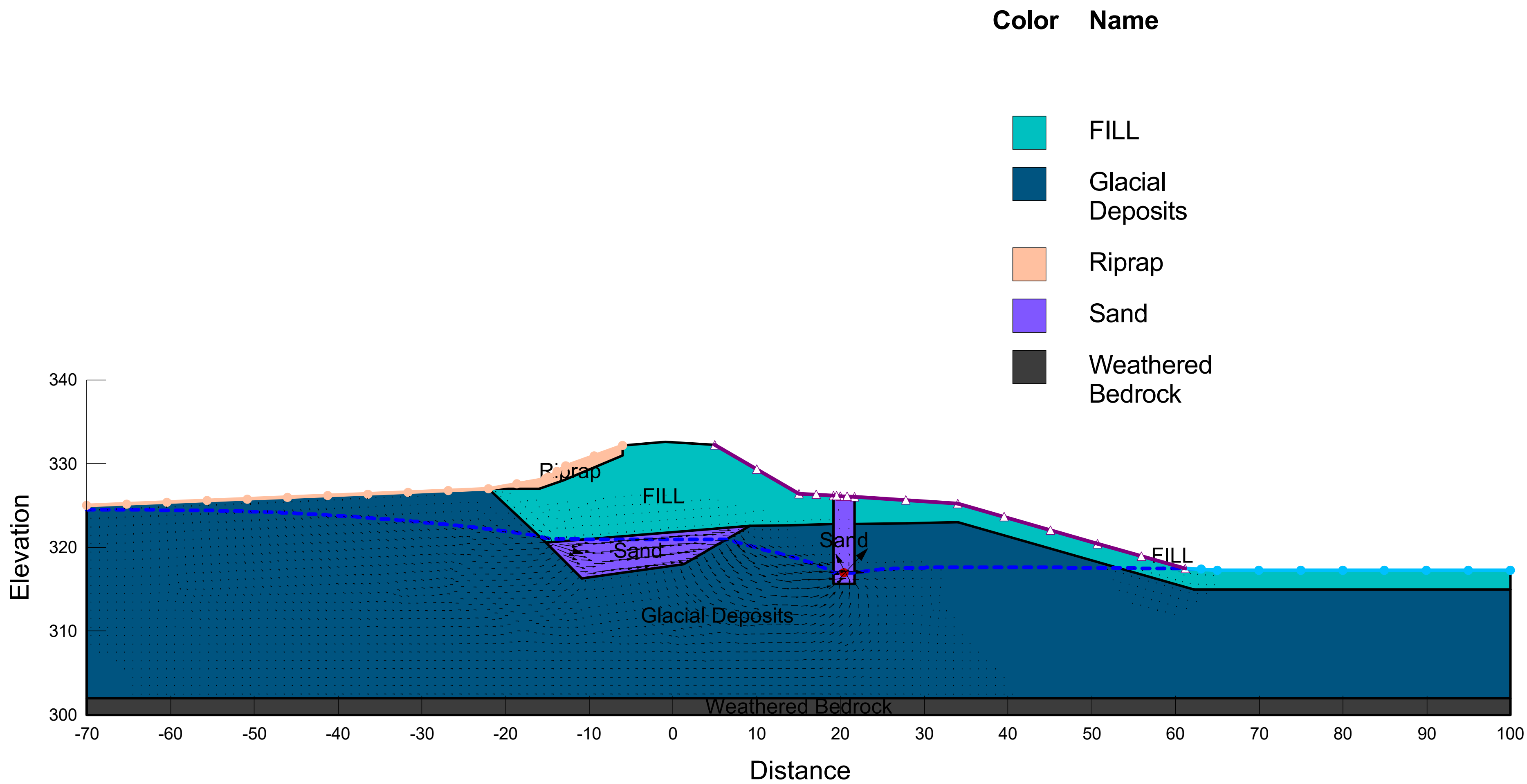


PROPOSED TRENCH DRAIN

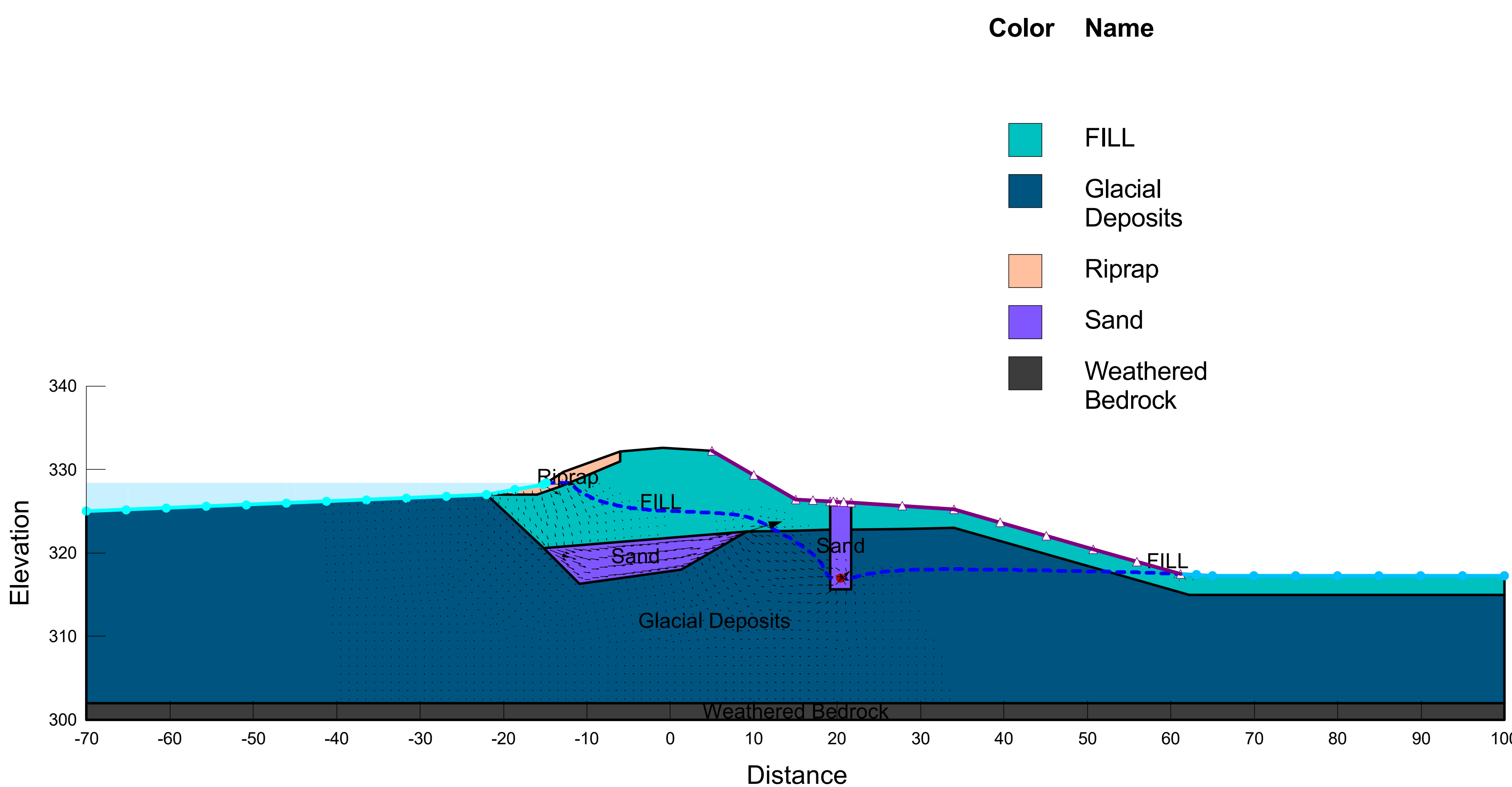
Trench Drain - Maximum Pool



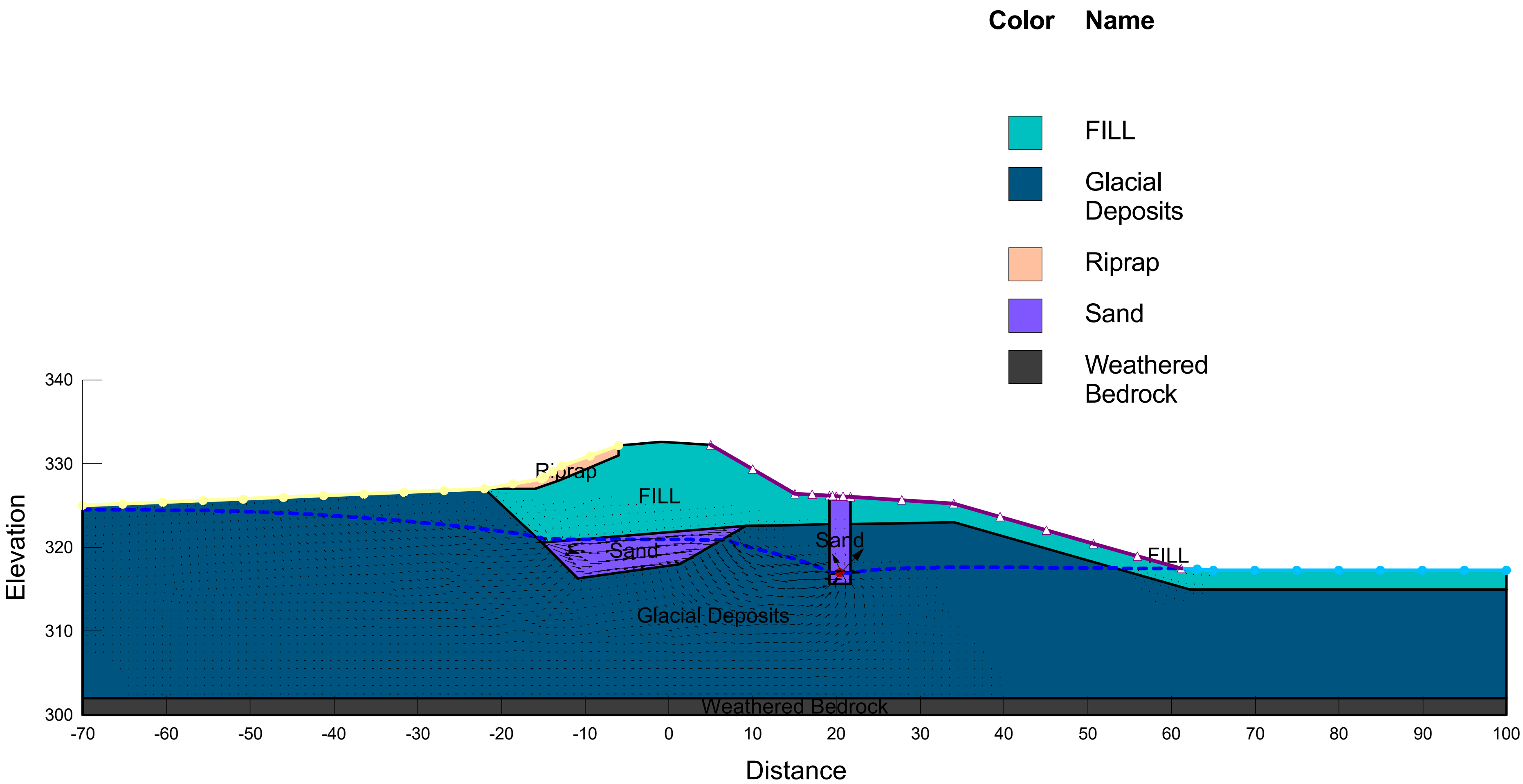
Trench Drain - Drawdown from Max



Trench Drain - Normal Pool

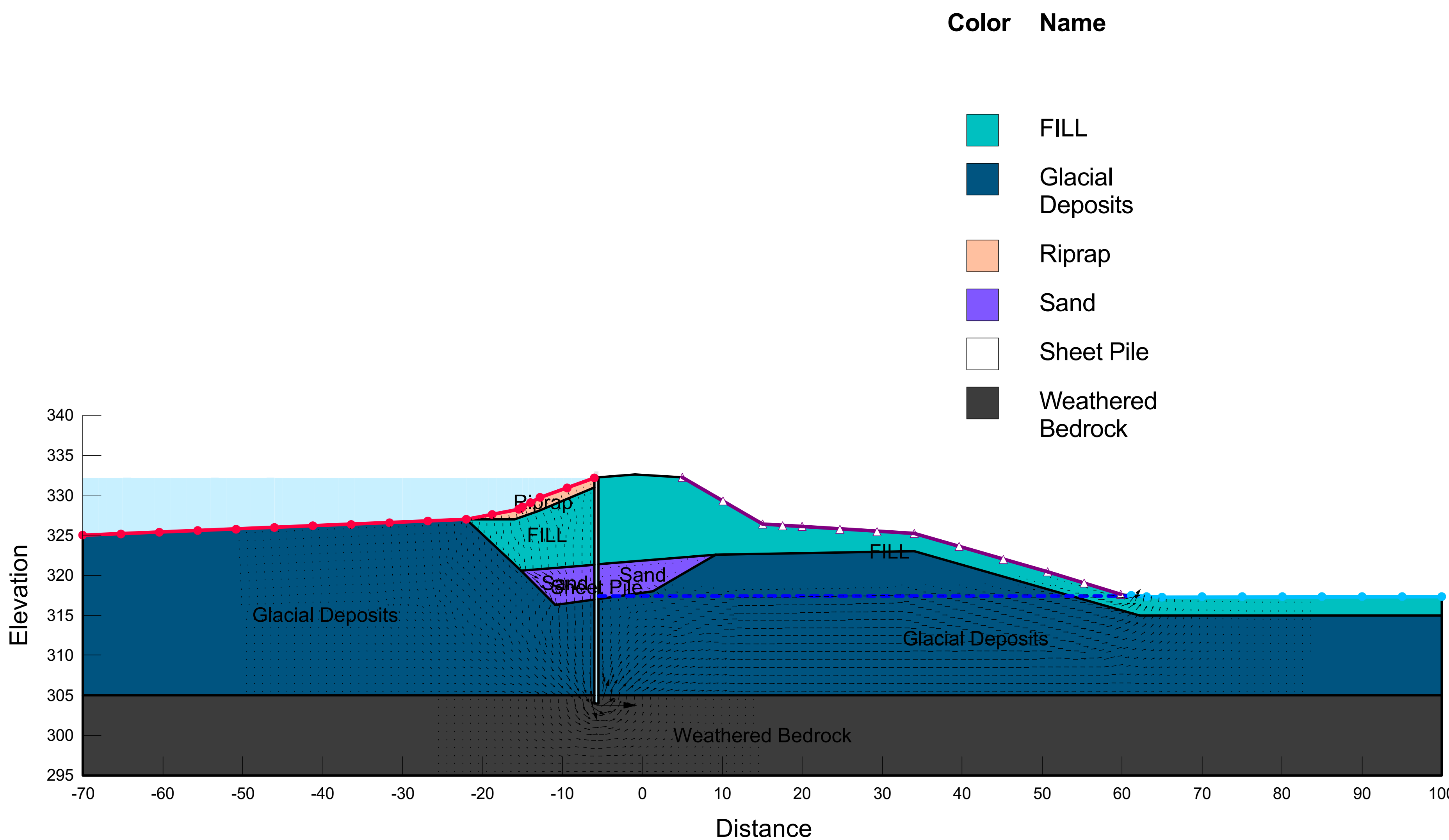


Trench Drain - Drawdown from Normal Pool

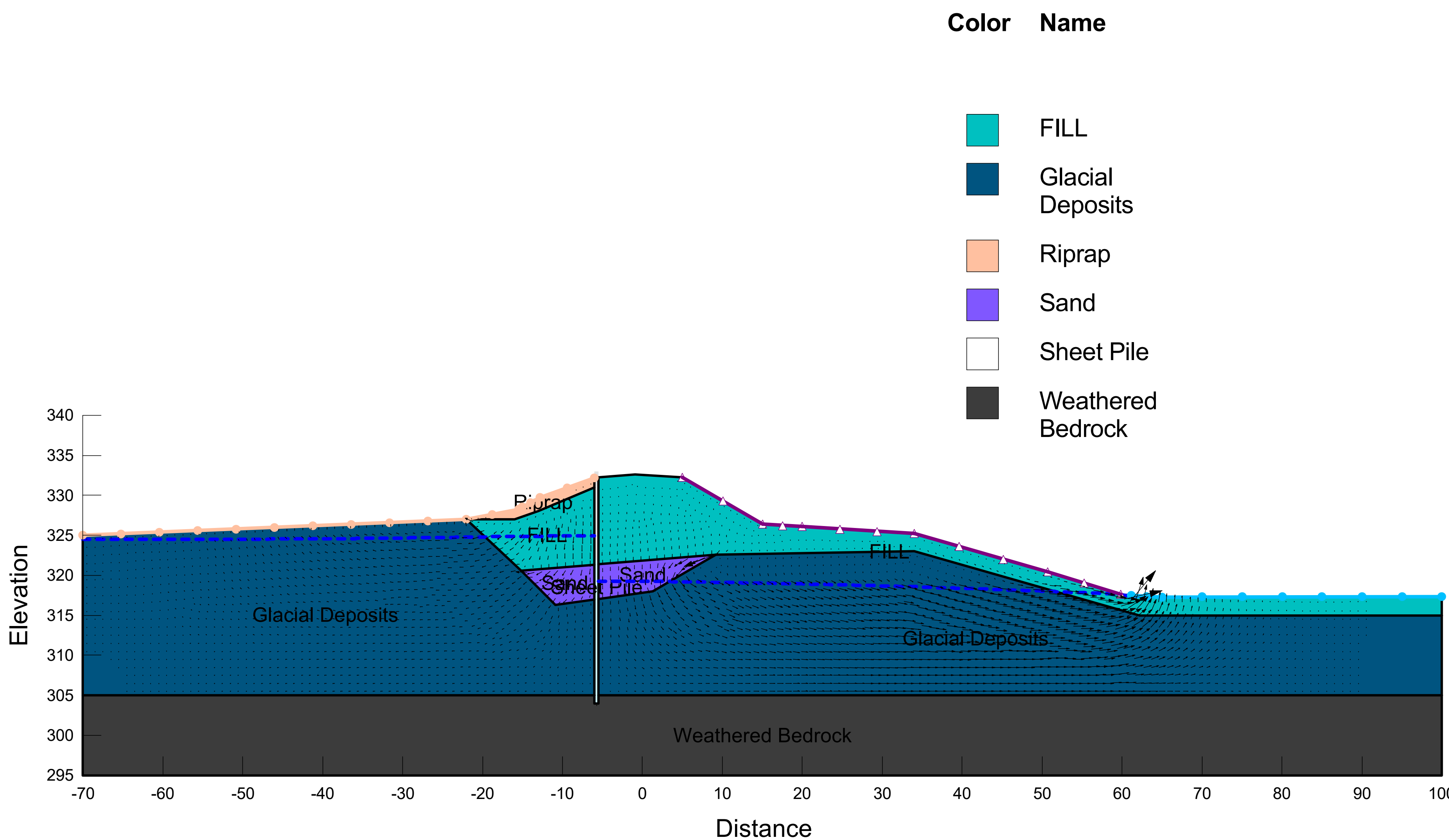


SHEET PILE CUTOFF WALL

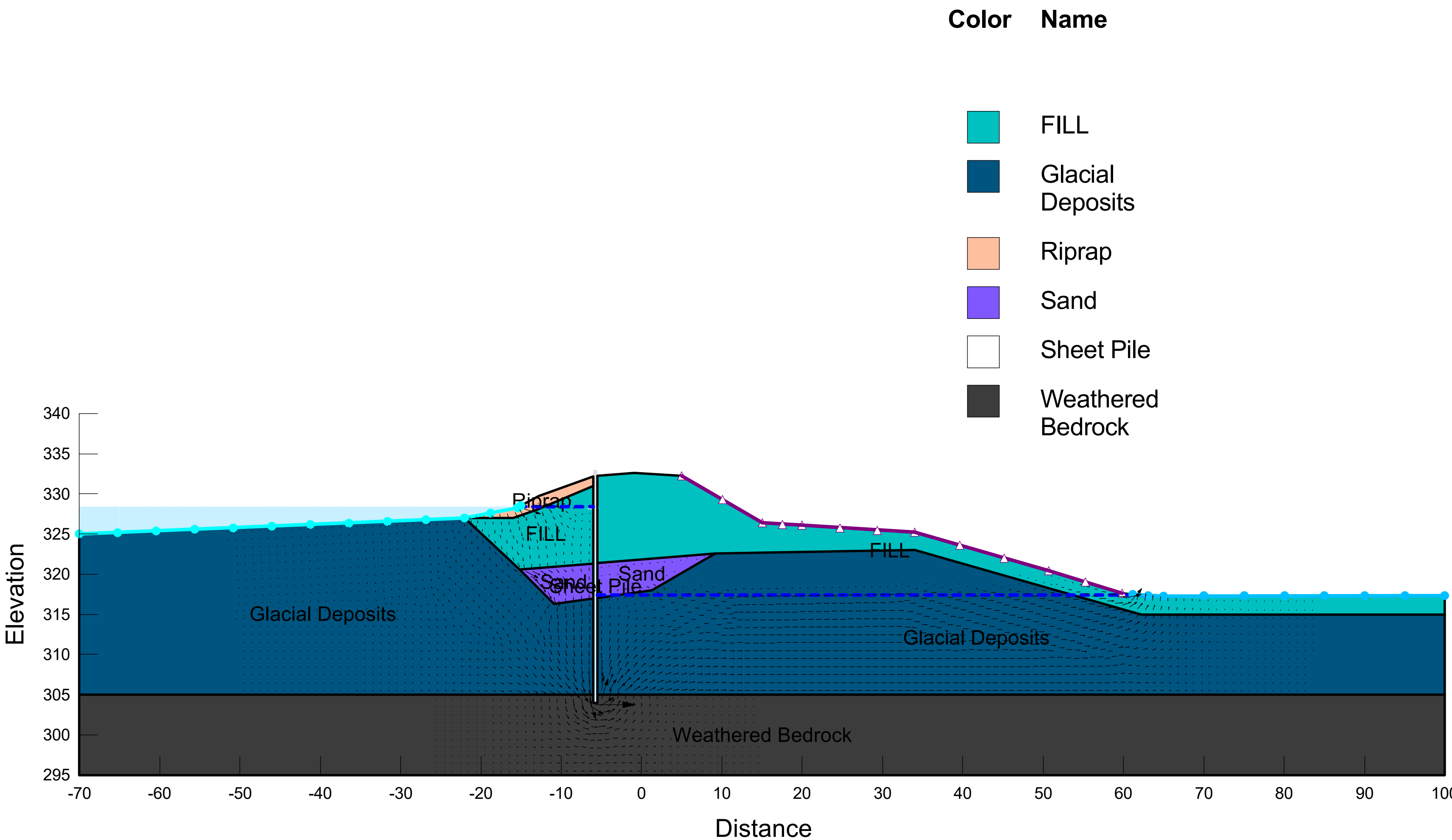
Sheet Pile Cutoff - Maximum Pool



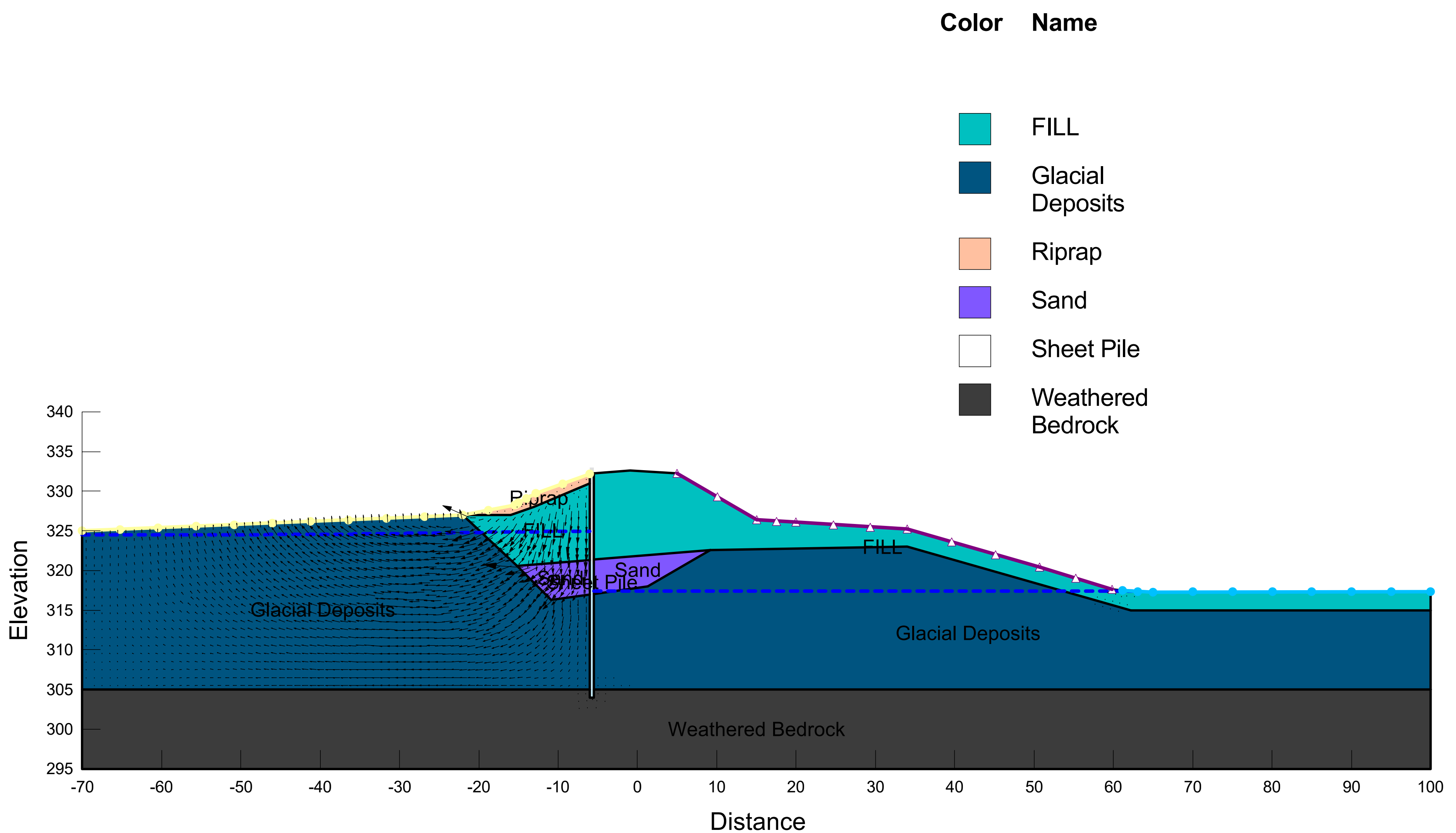
Sheet Pile cutoof - Drawdown from Max



Sheet Pile Cutoff - Normal Pool



Sheet Pile Cutoff - Drawdown from NP



APPENDIX D:
Visual Dam Inspection & Geotechnical Limitations
Waterman Lake Dam - Main Dam Study
Smithfield, RI

VISUAL DAM INSPECTION & GEOTECHNICAL LIMITATIONS

Visual Inspection

1. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigations and analyses involving topographic mapping, subsurface investigations, testing and detailed computational evaluations are beyond the scope of this report.

2. In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection, along with data available to the inspection team.

3. In cases where an impoundment is lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions, which might otherwise be detectable if inspected under the normal operating environment of the structure.

4. It is critical to note that the condition of the dam is evolutionary in nature and depends on numerous and constantly changing internal and external conditions. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Explorations

1. The analyses and recommendations submitted in this report are based in part upon the data obtained from subsurface explorations. The nature and extent of variations between these explorations may not become evident until construction. If variations then appear evident, Pare Corporation (Pare) should be asked to reevaluate the recommendations of this report.

2. The generalized soil profile described in the text is intended to convey trends in the subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretations of widely spaced explorations and samples; actual soil transitions are probably more erratic. For specific information, refer to the boring logs, test pit logs, and/or rock probe logs.

3. Water level readings have been made in the drill holes and or test pits at the times and under the conditions stated on the boring logs and/or test pit logs. These data have been reviewed and interpretations have been made in the text of this report. However, fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors occurring since the time the measurements were made.

Use of Report

1. This report has been prepared for the exclusive use of Citizens for the Preservation of Waterman Lake (CPWL) for specific application to the design basis report for the Waterman Lake Dam located in Smithfield, Rhode Island in accordance with generally accepted engineering practices. No other warranty, expressed or implied, is made.

2. This engineering report has been prepared for this project by Pare. This report is for design purposes only and is not necessarily sufficient to prepare an accurate bid. Contractors wishing a copy of this report may secure it with the understanding that its scope is limited to design considerations only.

APPENDIX E:
Previous Reports & References
Waterman Lake Dam - Main Dam Study
Smithfield, RI

PREVIOUS REPORTS AND REFERENCES

During the development of the report PARE also reviewed available information included within the following databases:

1. "Dam Inspection/ Evaluation Report", Pare Corporation, June 9, 11, and 12, 2020.
2. "Dam Inspection/ Evaluation Report", Pare Corporation, June 11, 2018.
3. Entries "Yearly Report of Commissioners of Dams and Reservoirs", 1883, 1885, 1908, 1913, 1916, 1919, 1921, 1927, 1929.
4. "Plan for Strengthening Retaining Wall at Draw Off Gate – Waterman Reservoir", 1884.
5. "Survey of State Dams" Division of Harbors and Rivers, July 15, 1940 (with plan)
6. "Special Inspection Report, Waterman's Reservoir, Dam No.111", Rhode Department of Public Works Division of Harbors and Rivers, November 1, 1946.
7. "Dam Inspection Report" Department of Natural Resources, December 15, 1977.
8. "Phase I Inspection Report – National Dam Inspection Program", ACOE, December 1977.
9. "Visual Inspection Checklist, Waterman", State of Rhode Island and Providence Plantations Department of Environmental Management, September 87, 1978.
10. "Dam Inspection Report, Waterman Lake Dam", State of Rhode Island and Providence Plantations Department of Environmental Management, May 29, 1985.
11. "Special Inspection Report, Waterman Lake Dam No.111," September 27, 1995.
12. "Special Inspection Report, Waterman Lake Dam No.111," May 21, 1996.
13. Letter – re: Main Gate Repairs (with gate drawings), CPWL, 1996.
14. "Dam Inspection Report", RIDEM, March 30, 2000.
15. "2010 Flood Status Report on Dams", RIDEM, March 31, 2010.
16. "Visual Inspection/Evaluation Report", Pare Corporation, August 2, 2012.

The following references were utilized during the preparation of this report and the development of the recommendations presented herein:

1. "Design of Small Dams", United States Department of the Interior Bureau of Reclamation, 1987
2. "ER 110-2-106 - Recommended Guidelines for Safety Inspection of Dams", Department of the Army, September 26, 1979.
3. "Guidelines for Reporting the Performance of Dams" National Performance of Dams Program, August 1994.

The following provides an abbreviated list of resources for dam owners to locate additional information pertaining to dam safety, regulations, maintenance, operations, and other information relevant to the ownership responsibilities associated with their dam.

1. RIDEM Office of Compliance and Inspection Website: http://www.dem.ri.gov/programs/benviron/compinsp/
2. "Dam Owner's Guide To Plant Impact On Earthen Dams" *FEMA L-263,September 2005*
3. "Technical Manual for Dam Owners: Impacts of Plants on Earthen Dams" *FEMA 534, September 2005*
4. "Dam Safety: An Owners Guidance Manual" *FEMA 145,December 1986*
5. Association of Dam Safety Officials – Website: www.asdso.org/
6. "Dam Ownership – Responsibility and Liability", ASDSO