Preliminary Geotechnical Engineering Report

Rush River Watershed – Amenia
Levee Alternative Sites Alt1 and Alt2
Cass County, North Dakota

Prepared for
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Appendix D-2

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Note: all geotechnical report appendices have been removed from the EA Appendix D version of this report, however they are available upon request to ND NRCS.

Certifications

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 North Dakota.

Eric Brandner, P.E.	May 17, 2019
PE #: PE-10374	Date

REGISTERED PROFESSIONAL ENGINEER
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NORTH DAKOTA

1 Introduction

Barr Engineering Co. (Barr), under authorization and contract with Moore Engineering, Inc. (Moore), completed a geotechnical investigation of two potential levee embankment alternatives to increase flood protection for the town of Amenia from the nearby Rush River. Barr understands that Moore is working in conjunction with the Cass County Joint Water Resource District to reduce the flood risk for the town of Amenia. At the time of this report, two proposed levee alternatives have been selected for preliminary geotechnical evaluation.

Barr performed a preliminary geotechnical investigation of both alternatives. This data was used for creating geotechnical models and performing analysis at representative locations across the project area. This report describes the preliminary geotechnical investigations and laboratory results and presents feasibility-level geotechnical evaluations, conclusions, and recommendations for levee embankment alternatives Alt1 and Alt2 for the town of Amenia in Cass County, North Dakota.

1.1 Site Location

The town of Amenia is located in north central Cass County, North Dakota (Figure 1). The two potential levee alternatives currently are located in fields consisting of agricultural farmland. Some trees and thicker vegetation is present near the existing river, and elevation generally decreases towards the river. The Rush River is located about 1/3 mile north of the town of Amenia, and many oxbows are present. The region is relatively flat, with occasional hills and valleys on the order of 10 feet of relief in a few locations, excluding the area near the Rush River. Figure 2 provides the location of the alternatives. Further discussion is provided in Section 1.3.

1.2 Geology

The following sections discuss the general geology of the project site.

1.2.1 Regional Physiography

The site is located in the Red River Valley region of North Dakota, within the Central Lowland physiographic province of the United States, and the soil is formed primarily from sediment that settled out of ancient glacial Lake Agassiz. Figure 3 shows the topography of the site. The surface elevation generally ranges between 940 to 970 feet.

1.2.2 Surficial Geology

Figure 4 indicates that the surficial geology consists primarily of the Oahe Formation, which was deposited during the Quaternary as a windblown silt, primarily along upland slopes. The thickness at the site is estimated to be between 0.2 to 1 meters (Clayton et al, 1976). Figure 5 indicates that the soil texture is mapped as various types of loam.

Sediment of glacial Lake Agassiz deposited in the Red River Valley has been recognized over an area of 200,000 square miles and is Quaternary in age (Harris et al, 1995). The area of the Red River Valley is a bedrock lowland with regional slope to the north. At least eight Pleistocene stratigraphic units underlie the Agassiz basin in North Dakota. The units vary significantly in texture and behavior. The town of Amenia is located on the outskirts and margins of the valley. The western edge of the valley is defined by beaches or shores of the ancient lake bed, primarily sand and gravel deposits, while the interior of the basin is comprised of Pleistocene lake-plain deposits (Arndt, 1977). The stratigraphy of the Red River Valley is well-known nearer the Red River where the deposits are thicker; however, towards the margins, the stratigraphy can become irregular and intermixed. Figure 6 indicates that the soil parent material at the site is mapped primarily as deposits of glaciolacustrine and alluvial deposits.

Near the town of Amenia, glacial sediment, the shallowest of which is anticipated to be derived from the Red Lake Falls Formation, underlies the Lake Agassiz sediment (Harris et al, 1995).

1.2.3 Bedrock Geology

Glacial deposits extend down to rock, which is anticipated to be on the order of 100 to 200 feet below the existing ground surface (Harris et al, 1995). According to the Geologic Bedrock Map of North Dakota (Bluemle, 1988), the shallowest bedrock is likely Cretaceous in age, consisting of the Belle Fourche, Mowry, Newcastle, and Skull Creek Formations. These formations are primarily observed as silty to sandy shale, with the exception of the Newcastle Formation, which is identified as a sandstone (Anderson, 2010). Bedrock is not anticipated to affect the design of either alternative.

1.2.4 Seismicity and Faults

No Quaternary faults are mapped at the site (USGS, 2019). Overall, seismic activity in the area is considered low.

1.3 Potential Alternative Levee Locations and Embankment Configuration

Based on communications with Moore, two potential alternatives are being evaluated to reduce the risk of flooding for the town of Amenia. The first, identified herein as Alt1, consists of a levee embankment around the town of Amenia (Figure 2). The crest elevation is 959.0 feet, with 3 feet of freeboard and side slopes of 4H:1V. The second alternative, identified herein as Alt2, consists of a levee embankment just south of the Rush River (north of the town of Amenia; Figure 2). The crest elevation ranges from 969.0 feet at the west portion and 959.0 feet at the east portion, with a general slope of 0.12 percent. The side slopes are 4H:1V and 3 feet of freeboard was assumed (Moore, 2018). Barr was provided preliminary plan sets for both alternatives dated January 24, 2019. It is possible that the final design elevations and embankment configurations may be different than the criteria used for this report.

The levee alternatives are not intended to impound a permanent pool but are only intended to temporarily retain water during flood conditions. Both levees will have storm water ponds to retain runoff during flood events. The proposed alignments of levee alternatives Alt1 and Alt2 at the time of this report are indicated on Figure 2.The ground surface elevation and geographic coordinates of the completed borings was taken from a survey performed by Moore upon completion of the geotechnical field work.

1.4 Previous Geotechnical Investigation

Barr was not aware of any previous geotechnical investigations conducted within the project boundaries. However, Barr has performed several geotechnical investigations within Cass County.

1.5 Geotechnical Investigation and Analysis

To support the design of the two proposed levee alternatives, a subsurface investigation, laboratory testing, and preliminary geotechnical engineering analysis were performed by Barr. The geotechnical components of the project are detailed below:

- Evaluation of soil stratigraphy based on field investigations
- Evaluation of soil parameters for seepage and stability modeling and analysis
- Preliminary modeling of seepage for the levee embankment system alternatives
- Underseepage mitigation evaluation
- Preliminary modeling of slope stability for the levee embankment system alternatives
- Evaluation of anticipated settlement for the levee embankment system alternatives
- Report discussing overall feasibility of the Amenia levee embankment alternatives Alt1 and Alt2

2 Preliminary Geotechnical Investigation Methods

2.1 Site Exploration

The preliminary geotechnical investigation consisted of soil borings, standard penetration testing (SPT), split-spoon soil sampling, undisturbed thin-wall tube sampling, and general soil laboratory testing. This program of geotechnical investigation was selected to accurately and efficiently evaluate the strength, compressibility, and density characteristics of the soils at the project site. The site investigation was conducted in August 2018, and laboratory testing was completed in October 2018. The following sections discuss the site work performed for the project.

2.1.1 Soil Borings

A total of nine soil borings were performed along the proposed alignments for Alt1 and Alt2. The soil boring locations are shown on Figure 7 and boring logs are included in Appendix A.

Soil borings along the proposed alignments were completed to a nominal depth of 40 feet below the existing ground surface. The boring locations were selected by Barr and approved by Moore to provide spatial coverage across the project area and to avoid unharvested crops. Figure 7 and Table 1-1 indicate the surveyed locations of the soil borings and elevations. Moore surveyed the borehole locations and provided the survey results to Barr.

Table 1-1 Summary of Soil Boring Locations

Boring ID	UTM Coordinates, Zone 14N NAD83 [meters]		Ground Surface Elevation [feet]	Total Depth of Boring [feet]
	Northing	Easting		
SB-01	5207455.7	635738.9	950.4	40.0
SB-02	5207424.4	634832.7	955.2	40.0
SB-03	5206970.0	635161.4	952.4	40.0
SB-04	5206941.5	635445.0	950.9	40.0
SB-05	5207394.6	633342.3	966.4	40.0
SB-06	5207591.0	633848.9	962.2	40.0
SB-07	5207831.4	634663.7	960.0	40.0
SB-08	5207883.3	635090.9	957.0	40.0
SB-09	5208169.9	635606.4	951.1	40.0

The soil borings were completed by Interstate Drilling Services, LLP, of Grand Forks, North Dakota, with a track-mounted drill rig using hollow-stem auger techniques. The augers used for the investigation were 4.25 inches in inner diameter, and the borehole was on the order of 9 inches in diameter. The soil borings were performed in general accordance with ASTM D1452, "Standard Practice for Soil Exploration and Sampling by Auger Borings." Standard penetration testing (SPT) and split-spoon sampling was performed in accordance with ASTM D1586, "Standard Test Method for Penetration Test and Split-Barrel Sampling of

Soils." Samples were collected continuously in order to determine the entire soil profile and evaluate for the presence of changing stratigraphy, sand or gravel seams, changing moisture content, and organic soils.

Three-inch-diameter Shelby tube samples were also collected at various depths for laboratory testing in accordance with ASTM D1587, "Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes." Where granular soils or stiff clays were encountered, Modified California sampling was performed in accordance with ASTM D3550, "Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils."

Based on the most recent autohammer calibration, which was performed in 2015, the minimum hammer efficiency was 68 percent. This indicates that the corrected *N*-values (N_{60}) are likely to be higher than the raw values if corrected to industry standards of 60-percent hammer efficiency. Hence, the raw *N*-values are reported on the boring logs.

The soil borings were observed and logged by Barr personnel in accordance with ASTM D2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)." Soil samples were delivered to Soil Engineering Testing, Inc. (SET) in Bloomington, Minnesota, for laboratory testing. The soil boring logs are provided in Appendix A.

2.1.2 Laboratory Testing

The following geotechnical laboratory analyses were completed by SET:

- Moisture content tests were performed in accordance with ASTM D2216, "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass"
- Dry unit weight tests were performed in accordance with ASTM D7263, "Standard Test Method for Laboratory Determination of Density (Unit Weight) of Soil Specimens"
- Grain Size and Hydrometer analysis in accordance with ASTM D422, "Standard Test Method for Particle-Size Analysis of Soils"
- Atterberg Limit determinations in accordance with ASTM D4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
- Unconfined compressive strength in accordance with ASTM D2166, "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil"
- Unconsolidated-Undrained (UU) Triaxial compressive strength in accordance with ASTM D2850, "Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils"
- Consolidated-Undrained (CU) Triaxial compressive strength in accordance with ASTM D4767, "Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils"

- Consolidation tests in accordance with ASTM D2435, “Standard Test Methods for One-Dimensional Consolidation Properties Using Incremental Loading”
- Shrink-swell testing in accordance with ASTM D4546, “Standard Test Methods for One-Dimensional Swell or Collapse of Cohesive Soils”
- Dispersion testing in accordance with ASTM D4221, “Standard Test Method for Dispersive Characteristics of Clay Soil by Double Hydrometer”
- Permeability testing in accordance with ASTM D5084, “Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter”
- Soil pH in accordance with ASTM D4972, “Standard Test Method for pH of Soils”
- Soil chemical analysis of soluble chloride and sulfate concentration in accordance with USEPA methods

Laboratory test reports and a summary of the laboratory tests completed are included in Appendix B.

3 Results

This section presents the data collected as part of the preliminary geotechnical investigation and provides further analysis of these results.

3.1 Subsurface Stratigraphy

Geologic information (Section 1.2), soil boring logs (Appendix A), laboratory test results Appendix B were reviewed to obtain an understanding of the project area subsurface stratigraphy.

The results of the soil borings indicated that the soils generally consisted of a thin layer of clayey topsoil overlying a layer of lean clay, which was underlain by softer clayey sediment of Lake Agassiz. Clayey glacial till soil deposits were encountered underneath the softer lake-plain deposits which extended to the termination depth of the soil borings. Silt soils were observed at three borings along the proposed alignment for Alternative 2 along the Rush River.

The soil types discussed in the following sections are the soil types used in seepage and stability modeling completed for the feasibility-level analysis.

3.1.1 Topsoil

Topsoil at the site consisted primarily of organic lean to fat clay with lesser amounts of sand. The topsoil was generally dark brown and the thickness ranged from 3 to 18 inches. The topsoil contained roots and other organic material consistent with planted agricultural fields.

3.1.2 Shallow Lean Clay

The results of the soil borings indicated the presence of shallow lean clay at all soil borings. The shallow lean clay extended to a depth ranging from 4 to 14.5 feet below the existing grade. The color of the shallow lean clay soils was observed to be primarily brown to dark brown with occasional gray zones. Lesser amounts of sand and gravel were observed throughout the shallow lean clay. The shallow lean clay deposits are anticipated to be deposits of the Oahe Formation.

Moisture content values for the shallow lean clay ranged from 11.9 to 29 0 percent, with an average of about 20 percent.

Atterberg limit testing on samples of the shallow lean clay indicated plastic limit values ranging from 19 to 24 percent, liquid limit values ranging from 37 to 40 percent, and plasticity index values ranging from 16 to 18 percent. According to the Plasticity Chart (Das, 2010), these soils plot as CL (lean clay).

Mechanical grain size testing in the shallow lean clay indicated no gravel content, the sand content was 29.4 percent, and the fines (silt and clay) content was 70.6 percent (dry weight).

Standard Penetration Test (SPT) *N*-values in the shallow lean clay ranged from 4 to 11 blows per foot, indicating that the shallow lean clay is in a soft to stiff condition. Hand penetrometer measurements on shallow lean clay indicated that the unconfined compressive strength ranged from 0.25 to 4.5 tons per square foot (tsf), with a typical range of about 1.5 to 2 tsf.

3.1.3 Sand

Sand was encountered at one of the soil borings (SB-08) between a depth of 3 to 4 feet. The shallow sand was observed to be tan. The sand was classified in the field as a silty to clayey sand. Sand is not widely anticipated at the project site, but may be encountered at isolated locations.

3.1.4 Fat Clay

The results of the soil borings indicated the presence of fat clay at all soil borings. The color of the fat clay was observed to be brown to tan towards the surface of the soil borings, and graded into gray to dark gray at depth, and contained very few sand inclusions. It is anticipated that the majority of the fat clay consists of various units of the Lake Agassiz sediment. Within the fat clay, occasional deposits and mineralization were observed which ranged in color from gray to white to orange.

Moisture content values for the fat clay ranged from 25.0 to 60.6 percent, with an average of about 44 percent.

Dry unit weights of the fat clay ranged from about 63.4 to 77.1 pounds per cubic foot (pcf). Moist unit weights for these soils were computed using the moisture content test results described above from the same samples tested for dry unit weight. The calculated moist unit weight ranged from 101.8 to 110.4 pcf, with an average of approximately 107 pcf.

Atterberg limit testing on samples of the fat clay indicated plastic limit values ranging from 25 to 29 percent, liquid limit values ranging from 71 to 107 percent, and plasticity index values ranging from 46 to 80 percent. According to the Plasticity Chart (Das, 2010), these soils plot as CH (fat clay).

Mechanical grain size testing in fat clay indicated no gravel content, the sand content was 2.1 percent, and the fines (silt and clay) content was 97.9 percent (dry weight). Hydrometer testing indicated that the silt content was 11.9 percent, and the clay content was 86.0 percent.

Seven laboratory unconsolidated-undrained (UU) triaxial compression tests were performed on samples of fat clay. Test results indicated that the maximum deviator stress ranged from 0.68 to 1.24 tsf. Hand penetrometer measurements on fat clay indicated that the unconfined compressive strength ranged from 0.25 to 4.5 tsf, with a typical range of about 0.5 to 2.0 tsf.

SPT *N*-values in the fat clay ranged from 2 to 16 blows per foot, with a typical range of 3 to 7 blows per foot, indicating that the fat clay is generally in a soft to medium stiff condition.

Three laboratory hydraulic conductivity tests were performed on intact samples of the fat clay. The results indicated that the hydraulic conductivity ranged from 1.10×10^{-8} to 4.30×10^{-8} cm/sec (3.61×10^{-10} to 1.41×10^{-9} ft/sec), with a geometric mean of 2.00×10^{-8} cm/sec (6.56×10^{-10} ft/sec).

Two laboratory direct shear tests were performed on intact samples of the fat clay, and the results indicated that the peak friction angle ranged from 5.9 degrees with an apparent cohesion of 0.341 tsf to 15.5 degrees with an apparent cohesion of 0.155 tsf.

One consolidated-undrained (CU) triaxial compressive strength test was performed on a sample of fat clay. Using the maximum deviator stress as the failure condition, the results indicated that the effective friction angle of the fat clay was 18.4 degrees, with an apparent cohesion of 0.16 tsf.

3.1.5 Silt

Native silt was observed at three soil borings (SB-05, SB-08, and SB-09), which were all observed along the alignment for Alt2 near the Rush River. The thickness of the silt layers ranged from about 5 to 10 feet. The silt was observed to be tan to orangish tan, with trace amounts of sand.

Moisture content values for the silt ranged from 34.3 to 41.0 percent, with an average of about 39 percent.

Atterberg limit testing on samples of the silt indicated plastic limit values ranging from 29 to 32 percent, liquid limit values ranging from 33 to 41 percent, and plasticity index values ranging from 4 to 9 percent. According to the Plasticity Chart (Das, 2010), these soils plot as ML (silt).

Mechanical grain size testing was performed on two samples of silt. Test results indicated no gravel content, the sand content ranged from 3.6 to 5.8 percent, and the fines (silt and clay) content ranged from 94.2 to 96.4 percent. Hydrometer analyses indicated that the silt content ranged from 69.4 to 86.8 percent, and the clay content ranged from 9.6 to 24.8 percent.

SPT *N*-values in the silt ranged from 2 to 7 blows per foot, indicating that the silt is in a very loose to loose condition.

3.1.6 Glacial Till

The results of the investigation indicated the presence of glacial till at all soil borings. The glacial till was observed to consist of low to moderate plasticity lean clay soil and was encountered at depths ranging from 11 to 31 feet below the existing ground surface. The color of the glacial till was observed to be brown to tan to gray. Lesser amounts of sand and gravel were observed throughout the glacial till. Occasional orange and gray mineralization were observed interbedded with the glacial till.

Moisture content values for the glacial till ranged from 18.7 to 25.1 percent, with an average of about 22 percent.

Dry unit weights of the glacial till ranged from 95.2 to 106.8 pcf. Moist unit weights for these soils were computed using the moisture content test results described above from the same samples tested for dry unit weight. The calculated moist unit weight ranged from 119.1 to 128.6 pcf, with an average of approximately 125 pcf.

Atterberg limit testing on one sample of the glacial till indicated a plastic limit value of 18 percent, a liquid limit value of 41 percent, and a plasticity index value of 23 percent. According to the Plasticity Chart (Das, 2010), this soil plot as CL (lean clay).

Mechanical grain size testing in glacial till indicated that the gravel content was 4.0 percent, the sand content was 35.3 percent, and the fines content was 60.7 percent (dry weight). Hydrometer testing indicated that the silt content was 39.4 percent, and the clay content was 21.3 percent.

Two laboratory unconfined compressive strength tests were performed, and the results indicated that the unconfined compressive strength ranged from 2.55 to 4.36 tsf. Hand penetrometer measurements performed on glacial till samples indicated that the unconfined compressive strength ranged from 1.25 to 4.5 tsf, with a typical range of 2.5 to 4.5 tsf.

SPT *N*-values in the glacial till ranged from 7 to 25 blows per foot, with a typical range of 12 to 20 blows per foot, indicating that the glacial till is generally in a stiff to very stiff condition.

One laboratory hydraulic conductivity test was performed on an intact sample of the glacial till. The results indicated that the hydraulic conductivity was 6.20×10^{-8} cm/sec (2.03×10^{-9} ft/sec).

3.1.7 Reworked Glacial Till

Visually, the glacial till appeared relatively consistent once encountered, but the results of SPT and hand penetrometer testing indicated that there was a thinner zone at the top of the glacial till unit at borings SB-01, SB-02, SB-03, SB-06, SB-07, and SB-09 that was much softer than the underlying glacial till. The weaker zones are anticipated to be a reworked zone of glacial till and are anticipated to behave in a slightly different way than the intact glacial till.

Moisture content values for the reworked glacial till ranged from 19.2 to 39.0 percent, with an average of about 32 percent.

Dry unit weights of the reworked glacial till ranged from 82.1 to 89.9 pcf. Moist unit weights for these soils were computed using the moisture content test results described above from the same samples tested for dry unit weight. The calculated moist unit weight ranged from 114.1 to 117.6 pcf, with an average of approximately 115 pcf.

Atterberg limit testing on one sample of the reworked glacial till indicated a plastic limit value of 38 percent, a liquid limit value of 22 percent, and a plasticity index value of 16 percent. According to the Plasticity Chart (Das, 2010), this soil plots as CL (lean clay).

Two laboratory unconsolidated-undrained (UU) triaxial compression tests were performed on samples of the reworked glacial till. Test results indicated that the maximum deviator stress ranged from 1.17 to 1.95 tsf, slightly weaker than the underlying material. Hand penetrometer measurements performed on samples of reworked glacial till indicated that the unconfined compressive strength ranged from 0.25 to 2.5 tsf, with a typical range of 0.5 to 1.5 tsf.

SPT *N*-values in the reworked glacial till ranged from 2 to 9 blows per foot.

One consolidated-undrained (CU) triaxial compressive strength test was performed on a sample of reworked glacial till. Using the maximum deviator stress as the failure condition, the results indicated that the effective friction angle of the clay was 36.8 degrees with no apparent cohesion.

3.2 Groundwater Conditions

Groundwater was encountered at all soil borings while drilling or immediately after drilling at depths ranging from 11.0 to 35.0 feet. Upon completion of drilling when the augers were removed from the borehole, the soils caved in to a depth ranging from 4.8 to 18.4 feet. Groundwater measurements from the field investigation are provided in Table 3-1. All depths are referenced from below the existing ground surface at the time of the investigation.

Table 3-1 Summary of Groundwater Levels from Soil Borings

Boring ID	Groundwater Measurement Depth [feet]		
	While Drilling	End of Drilling	Cave-in Depth
SB-01	NE	NE	NE
SB-02	27.5	NE	18.1
SB-03	19.0	NE	17.1
SB-04	35.0	NE	6.1
SB-05	15.0	NE	4.9
SB-06	17.0	NE	4.8
SB-07	25.0	NE	18.4
SB-08	35.0	NE	9.9
SB-09	11.0	13.2	14.1

NE – Not Encountered

Given the lower permeability of the primarily clayey soils encountered at the site, it is possible that the groundwater levels did not have time to stabilize in the short time the boreholes were open. As a result, the groundwater level may be shallower than observed. In general, the high moisture content of the fat clay tends to indicate that the soils are saturated, and the groundwater level is anticipated to be near the shallow lean clay/fat clay interface.

Many factors contribute to water level fluctuations, such as heavy rainfall events, dry periods, sand seams, etc. Based upon the observations made during drilling, the groundwater at both potential alternative alignments is anticipated to be in the upper 10 feet of soil.

3.3 General Soil Laboratory Testing

The laboratory test results from the soil borings are provided in Appendix B. Test results are summarized in Table B1 of Appendix B.

3.3.1 Moisture Content

A total of 51 moisture content tests were performed on samples collected from the soil borings. The soils tested included sands, clays, and silts. The native soil had moisture contents ranging from 11.9 to 60.6 percent, with an overall average of about 34 percent. The fat clay, reworked glacial till, and silt typically had higher moisture contents and the shallow lean clay and intact glacial till generally exhibited lower moisture contents.

3.3.2 Atterberg Limits

Atterberg Limits testing was performed on fine-grained soil samples and used to classify the material encountered in the borings. A total of 11 Atterberg Limits tests were conducted.

Atterberg Limits test results indicated that the liquid limit ranged from 33 to 107 percent, the plastic limit ranged from 18 to 32 percent, and the plasticity index ranged from 4 to 80 percent. According to the Plasticity Chart (Das, 2010), the soils tested are classified as CL (lean clay), CH (fat clay), and ML (silt).

3.3.3 Unit Weight

A total of 17 dry unit weight tests were performed on intact soil samples obtained during the investigation. Dry unit weight test results on all samples ranged from 63.4 to 106.8 pcf. Moist unit weight estimations using moisture contents from samples with dry unit weight results ranged from 101.8 to 128.6 pcf, with an average of about 111 pcf.

The fat clay exhibited the lowest dry unit weight, while the glacial till tended to exhibit the higher dry unit weight.

3.3.4 Mechanical Grain Size Analysis

Mechanical grain size testing was performed on five soil samples collected during the investigation. Test results indicated that the gravel content ranged from none to 4.0 percent, the sand content ranged from 2.1 to 35.3 percent, and the fines (silt and clay) content ranged from 60.7 to 97.9 percent (dry weight). Hydrometer analysis indicated that the silt content ranged from 11.9 to 86.8 percent and the clay content ranged from 9.6 to 86.0 percent.

3.3.5 Compressibility

Fine-grained soils (clay and silt) experience long-term consolidation if saturated and subjected to increased loading. Some of the fine-grained soils observed at the site were observed to be saturated, and are anticipated to experience long-term settlement as the increased stress from the proposed embankment squeezes out the water from the pore spaces.

Compressibility of the existing clayey soils was evaluated using laboratory one-dimensional consolidation testing. One sample of lean clay and one sample of fat clay were selected for laboratory testing. The consolidation test results are provided in Table 3-2.

Table 3-2 Summary of Laboratory Consolidation Test Results

Boring ID	Depth [ft]	Soil Type	P_c' [tsf]	OCR	C_c	C_r	e_0
SB-08	30-32	CL (glacial till)	5.00	4.4	0.18	0.02	0.624
SB-04	15-17	CH (fat clay)	3.70	5.5	0.68	0.16	1.597

Based on the test results, the clay soils appear to be overconsolidated (i.e., the current existing stress on the soil is less than the maximum stress that the soil has encountered throughout its history). Overconsolidated soils generally have a lower potential for settlement than normally consolidated soils. Glacial deposits are typically overconsolidated because the glaciers have previously compressed the material. Test results indicate that the clay soils have a relatively low to moderate compressibility.

3.3.6 Chemical Testing

Chemical testing was performed on two soil samples collected from the project site. Test results indicated that the soil pH ranged from 8.2 to 8.3, the soluble chloride concentration ranged from 64.0 to 576.0 mg/kg, and the soluble sulfate concentration ranged from 1,640.0 to 2,360.0 mg/kg. Soil chemical test results are summarized in Table 3-3.

Table 3-3 Summary of Chemical Tests on Soil Samples

Boring ID	Depth [ft]	Soil Type	pH	Soluble Chloride [mg/kg]	Soluble Sulfate [mg/kg]
SB-01	5-7.5	CH	8.3	64.0	1640.0
SB-04	2.5-4	CH	8.2	576.0	2360.0

3.3.7 Hydraulic Conductivity

Hydraulic conductivity tests were performed on selected samples to determine the permeability of the material for seepage analysis. The hydraulic conductivity tests were performed with the flexible-wall permeameter method according to ASTM D5084. Clay was the predominant soil type observed at the project site, and so four clay samples were tested. Impact sample test results indicated that the hydraulic conductivity of the clay soils ranged from 1.10×10^{-8} to 6.20×10^{-8} cm/sec (2.03×10^{-9} to 3.61×10^{-10} ft/sec), with a geometric mean of 2.66×10^{-8} cm/sec (8.72×10^{-10} ft/sec).

The hydraulic conductivity results from laboratory testing are considered a measure of vertical permeability as the water is forced to flow through the sample from the bottom face to the top face of the cylindrical specimen.

3.3.8 Double Hydrometer Testing

One double hydrometer test was performed on a sample of silt to evaluate the dispersion potential. Test results indicated that the dispersion potential was approximately 2 percent.

3.3.9 Soil Shrink-Swell

One laboratory shrink-swell test was performed on a sample of fat clay from the site. The specimen was inundated with distilled water in order to determine the free swell potential (swell percentage). The specimen was then incrementally loaded until the initial specimen height was achieved (swell was eliminated by loading). The results indicate that the sample tested free-swelled 5.4 percent, with no potential swell (defined as the vertical swell under a pressure equal to the overburden stress). The associated swell pressure (amount of pressure required to negate the swell) was 0.73 tsf.

3.4 Soil Shear Strength

The strength of the soils was determined from field and laboratory testing. Laboratory strength testing results are provided in Appendix B. The following sections discuss the soil strength in terms of friction angle (for the drained condition) and undrained shear strength (for the undrained condition).

3.4.1 Drained Shear Strength

Laboratory direct shear testing was performed on two samples of fat clay. The peak friction angles of drained shear strength ranged from 5.9 degrees with an apparent cohesion of 0.341 tsf to 15.5 degrees with an apparent cohesion of 0.155 tsf, respectively.

Two consolidated-undrained (CU) triaxial compressive strength tests were performed on samples of clay soil. Using the maximum deviator stress as the failure condition, the results indicated that the effective friction angle of the clay ranged from 18.4 to 36.8 degrees and the apparent cohesion ranged from 0.16 tsf to none, respectively.

3.4.2 Undrained Shear Strength

The undrained shear strength values for cohesive soils were derived from unconfined compressive strength testing and unconsolidated undrained (UU) triaxial strength tests on Shelby tube samples from the borings. Hand penetrometer measurements were also considered for the analysis. Undrained shear strength values are considered to be half of the unconfined compressive strength or maximum deviator stress of the soil at failure. SPT *N*-values are indicated on the boring logs in Appendix A.

The results from laboratory unconfined compressive strength testing, unconsolidated undrained triaxial compressive strength testing, and hand penetrometer testing indicated that the undrained shear strength ranged from approximately 680 to greater than 4,360 psf.

4 Preliminary Geotechnical Analysis

Geotechnical models were created for representative cross sections across the project area where varying conditions of subsurface stratigraphy were encountered. The primary goal of the preliminary analysis was to evaluate the slope and seepage stability across the project alignment for typical and worst-case conditions and, if necessary, provide a preliminary design to alleviate slope stability concerns.

4.1 Geometry and Design Considerations

The geometry of the cross sections is discussed in the following sections. For the preliminary analysis, two cross sections were evaluated: one for Alt1 and one for Alt2. The locations of these cross sections were selected to evaluate the varying conditions below the proposed embankment. The location of the modeled cross sections is shown on Figure 8.

The levee embankment configurations provided to Barr by Moore indicated that the Alt1 embankment height should be at an elevation of about 959 feet (mean sea level) and the Alt2 embankment height ranges from 959.0 to 969.0 feet (mean sea level) (Moore, 2018). The levee embankments are not planned to have an upstream pool under "normal conditions." Based on conversations with Moore, the levees are planned to be designed for 3 feet of freeboard (Moore, 2018). For the purposes of this report, the hydraulic loading condition of water at the freeboard height is referred to as "normal flood conditions" and the hydraulic loading condition of water at the crest is referred to as "maximum flood conditions."

The embankment fill was assumed to be clay from an on-site borrow pit. The location has not been identified at the time of this report.

The ground surface geometry used in the models was constructed based on available data from public sources and from measured elevations by Moore at the completed soil boring locations. As such, there is likely some variability between the modeled cross sections and the actual ground elevations. Barr recommends collecting additional survey information via traditional methods or light detecting and range (LiDAR) and bathymetric survey of the river for a more precise representation of the existing conditions for use in final analysis and construction depending on which alternative is selected.

4.1.1 Soil Profile Alignment

To assist in visualizing the soil stratigraphy along the proposed embankment, Barr prepared a profile drawing of the proposed alignments which took into account the stratigraphy of the recent soil borings. The apparent soil profile along the Alt1 and Alt2 alignments is provided in Appendix C1 and Appendix C2, respectively. Based on the results of the investigation, the main types of soil encountered were shallow lean clay, fat clay, silt, reworked glacial till, and glacial till. SPT *N*-values are indicated on the alignment. Because the seasonal water levels have not been studied (or provided to Barr), the assumed groundwater level is not provided on the profile alignment.

4.1.2 Cross Section 1 – Alt1

Soil stratigraphy for Alt1 was based on the results of the geotechnical investigations and represents Barr's interpretation of the existing soil conditions near the selected cross section. This cross section is labeled *CS1* for Alt1 on Figure 8, and stratigraphy was estimated primarily from soil boring SB-04 and other nearby information. The elevation of the existing ground surface is approximately 950.9 feet. This location was selected for analysis because of the thick fat clay deposits and relatively deep glacial till soil. To evaluate the factors of safety with respect to slope stability and seepage (including heave and erosion) of the cross section, a clay fill levee embankment with a crest width of 10 feet, upstream and downstream side slopes of 4H:1V, and crest elevation of 959.0 feet was analyzed per the provided embankment configuration. Barr examined no flood conditions, normal flood conditions, maximum flood conditions, and rapid drawdown scenarios.

4.1.3 Cross Section 2 – Alt2

Soil stratigraphy for Alt2 was based on the results of the geotechnical investigations and represents Barr's interpretation of the existing soil conditions near the selected cross section. This cross section is labeled *CS2* for Alt2 as shown on Figure 8, and stratigraphy was estimated primarily from soil boring SB-08 and other nearby information. The elevation of the existing ground surface at the cross section location is approximately 957.0 feet. The crest height varies for Alt2. Barr assumed that the crest height at Cross Section 2 was 969.0 feet, which is anticipated to be slightly higher than the actual crest elevation at that location. This location was selected for analysis because of the presence of the silt layer. To evaluate the factors of safety with respect to slope stability and seepage (including heave and erosion) of the cross section, a clay fill levee embankment with a crest width of 10 feet, upstream and downstream side slopes of 4H:1V, and crest elevation of 969.0 feet was analyzed per the provided embankment configuration. Barr examined no flood conditions, normal flood conditions, maximum flood conditions, and rapid drawdown scenarios.

4.2 Seepage Analysis

The main objective of the seepage analysis was to develop an understanding of the seepage flow through and under the levee embankment and its relationship to stability of the embankment slopes. Seepage through an embankment plays a major role in the stability and construction sequence of the embankment. Simulations were made to estimate seepage flow conditions for the proposed embankment.

The seepage simulations presented in this report modeled seepage flow through and under the levee embankment under steady-state conditions and rapid drawdown conditions. The seepage analyses for the hydraulic loading conditions were performed at each of the preliminary design sections identified in Section 4.1. In the analyses, each was evaluated for the final construction configuration (assuming no flood events during the construction process).

4.2.1 Seepage Analysis Background

The seepage analysis used for the levee embankment was conducted using SEEP/W, a computer modeling program developed by GEO-SLOPE International, Ltd. SEEP/W uses the finite-element analysis technique to model the water movement and pore-water pressure distribution within porous materials such as soils. This method was chosen because comprehensive formulation allows evaluation of highly complex seepage problems. SEEP/W can formulate saturated and unsaturated flow, steady-state and transient conditions, and a variety of boundary conditions. Model integration allows the use of seepage files in limit-equilibrium slope-stability analysis. SEEP/W generates an output file containing the heads at the nodes of the finite-element mesh. The integration of GEO-SLOPE products allows the use of the SEEP/W head file in the slope stability program (SLOPE/W) to compute the effective stress. Therefore, it allows evaluation of the seepage impact on stability. SLOPE/W also has an imbedded analysis method to conduct rapid drawdown evaluations.

4.2.2 SEEP/W Parameters

The following sections summarize the hydraulic conductivity parameters selected for seepage modeling (discussed in Section 3.3.7). The main parameter associated with soils relevant to the seepage analysis is the hydraulic conductivity, which is also referred to as permeability. The laboratory testing performed provided estimates of the vertical permeability, but that value was assumed for the horizontal permeability as well, which for well-graded soils is generally appropriate.

4.2.2.1 Shallow Lean Clay

The parameters for the shallow lean clay were assumed based on correlations to the soil type (Das, 2010). A value of 3.28×10^{-9} ft/s (1.00×10^{-7} cm/s) was selected.

4.2.2.2 Embankment Fill

Because the shallow lean clay may be used as the embankment fill, a permeability of 3.28×10^{-9} ft/s (1.00×10^{-7} cm/s) was used for the embankment fill.

4.2.2.3 Fat Clay

The parameters for the fat clay were taken from laboratory testing performed during the geotechnical investigation. The geometric mean of the data was selected for analysis, corresponding to a value of 6.56×10^{-10} ft/s (2.00×10^{-8} cm/s).

4.2.2.4 Silt

The permeability of the silt was evaluated using the Kozeny-Carman formula (outlined in Carrier, 2003), which is based on the grain-size distribution and void ratio. Based on the results of the analysis, the permeability of the silt was estimated to range from 3.51×10^{-5} to 2.26×10^{-5} ft/s (1.07×10^{-3} to 6.88×10^{-4} cm/s). A value of 2.26×10^{-5} ft/s (6.88×10^{-4} cm/s) was selected for silt, which generally agrees with published values for silts as identified in Freeze, et al. (1979).

4.2.2.5 Glacial Till and Reworked Glacial Till

The parameters for the glacial till and reworked glacial till were taken from laboratory testing performed during the geotechnical investigation. A value of 2.03×10^{-9} ft/s (6.20×10^{-8} cm/s) was selected.

4.2.2.6 Summary of Seepage Parameters

All soils were modeled using the "Saturated Only" model type, which assumes all soils used in the model are saturated. A summary of inputs used for seepage modeling is provided in Table 4-1.

Table 4-1 Recommended Seepage Parameters

Material Type	Model Type	Saturated Hydraulic Conductivity cm/s	Saturated Hydraulic Conductivity ft/s
Embankment Fill	Saturated Only	1.00E-07	3.28E-09
Shallow Lean Clay	Saturated Only	1.00E-07	3.28E-09
Fat Clay	Saturated Only	2.00E-08	6.56E-10
Silt	Saturated Only	6.88E-04	2.26E-05
Reworked Glacial Till	Saturated Only	6.20E-08	2.03E-09
Glacial Till	Saturated Only	6.20E-08	2.03E-09

*The anisotropy (Ky'/Kx' ratio) was assumed to be 1.0 for all materials.

4.2.3 Boundary Conditions and Assumptions

Boundary conditions and assumptions for the seepage simulations are as follows:

- Under normal flood conditions, the entire upstream portion of the embankment was modeled as constant total head of 956.0 feet for Alt1 and 966.0 feet for Alt2 (corresponding to the freeboard height, which is assumed to be 3 feet below the planned embankment height).
- Under maximum flood conditions, the upstream portion of the embankment was modeled as having groundwater up to the embankment crest elevation of 959.0 feet for Alt1 and 969.0 feet for Alt2.
- The proposed embankment was assumed to consist of recompacted on-site lean clay collected from the near-surface soil. A compaction level of 95 percent was assumed for the analysis.
- The embankment configurations were modeled as described in Sections 4.1.2 and 4.1.3.

4.2.4 Results of Seepage Analysis

The USACE provides specific guidance in regard to design of seepage control measures for levees in EM 1110-2-569 (2005), "Design Guidance for Levee Underseepage." The cross sections were modeled and analyzed for seepage, exit gradients, heave, and piping/erosion.

The calculated seepage flow through the proposed embankment was assessed to understand if additional seepage measures were required, such as underdrains or filters. The estimated water flux through the entire embankment was estimated based on the modeling results.

The recommended minimum required seepage factors of safety against piping/erosion and heave at the downstream toe of the levee are 1.6 for the normal flood water elevation (equal to the freeboard height) (ETL 1110-2-569, 2005), and a reduced factor of safety of 1.3 for the maximum flood water elevation (assumed to be the top of the embankment) (ETL 1110-2-575, 2011). The factor of safety for piping / erosion was estimated by dividing the critical gradient (buoyant soil unit weight divided by unit weight of water) by the exit gradient (change in total head divided by distance between measured total heads). The exit gradient was calculated between the toe of the embankment and approximately 2 feet below the toe when the embankment is founded on homogeneous materials. Alternatively, if the embankment is founded on low-permeability materials that are underlain by higher permeability materials, calculations were performed across the entire thickness of the uppermost low permeability clay layer.

The factor of safety for piping/erosion was only applied at cross sections where groundwater was passing through the ground surface at or near the downstream toe of the levee embankment. When groundwater was not passing through the ground surface at or near the downstream toe of the levee embankment, only the factor of safety for heave was calculated. The factor of safety for heave is determined by dividing total vertical stress by pore-water pressure at the interface between a high-permeability material overlain by a low-permeability material. Water above the ground surface was accounted for in the heave calculation by subtracting the pore-water pressure at the ground surface from the total vertical stress and pore-water pressure at the interface between the high and low permeability material.

The results from the analysis for piping/erosion, exit gradient, and heave without seepage mitigation are provided in Table 4-2.

Table 4-2 Summary of Factors of Safety for Heave and Erosion of Embankment

Cross Section	Analysis No.	Hydraulic Condition	Downstream Side Slope	Erosion FOS	Heave FOS	Target FOS	Estimated Water Flux Rate Under Embankment [ft^3/sec/ft of embankment]
CS1	2.0	Normal Flood	4H:1V	6.8	2.1	1.6	2.00E-09
	3.0	Maximum Flood	4H:1V	5.9	2.0	1.3	8.15E-09
CS2	2.0	Normal Flood	4H:1V	3.6	1.6	1.6	1.03E-07
	3.0	Maximum Flood	4H:1V	3.0	1.5	1.3	1.32E-07

The results of the piping/erosion and heave factors of safety indicated that both alternatives meet the required factor of safety for all hydraulic loading conditions, and additional considerations during final design to control seepage may not be required.

The results of the analysis at Cross Section 2 for Alt2 indicated that the factor of safety against erosion and heave at the downstream toe of the embankment would meet the required values. At that cross section, a layer of more permeable silt is present interbedded with the cohesive soil. This soil layering was one of the reasons this cross section was selected, as it was perceived to potentially be a risk for lower factors of safety. From a feasibility-level perspective, this cross section has a higher potential for seepage concerns since the factor of safety is near the recommended values.

4.3 Slope Stability Analysis

Two types of stability analyses are typically performed for slopes: the Undrained Strength Stability Analysis (USSA) and the Effective Stress Stability Analysis (ESSA). The USSA case is performed to analyze the case in which loading or unloading is applied rapidly, and excess pore-water pressures do not have sufficient time to dissipate during shearing. This scenario typically applies to loading from, for example, embankment construction where the loading takes place quickly relative to the permeability of the soils. Loading from flood waters also qualifies for USSA scenarios. This is often referred to as the "end-of-construction" case.

The ESSA case is performed to account for much slower loading or unloading, no external loading, or the case where excess pore pressures developed during rapid loading or unloading are fully dissipated, in which the drained shear strength of the materials is mobilized and no excess shear-induced pore pressures are present. Final design cases of embankments and excavated slopes also fall into this case. For this reason, the ESSA is often referred to as the "long term" case.

Both USSA and ESSA analyses were performed as part of the slope stability analysis for each of the hydraulic loadings on each cross section. This is because the initial construction case and flood water levels will cause excess pore-water pressures to develop and undrained shear strengths could be mobilized. Long-term design cases based on very slow or no fluctuation of water levels will generally allow for the possibility of drained shear strengths to be mobilized.

In addition to the USSA and ESSA analyses, Barr analyzed the embankment assuming that the water level dropped rapidly from the normal loading condition. This is considered a rapid drawdown condition, which occurs when the stabilizing pressure of the water on the upstream is lost, but the pore pressures within the levee embankment do not dissipate as quickly. This leads to potential instability of the embankments. It was considered unlikely that the embankment at the site will ever undergo a rapid drawdown from the maximum (crest height) hydraulic conditions to a water level which provides no support.

The stability of a slope is reported using a factor of safety value. The factor of safety is the ratio of the summation of forces and moments that are resisting slope movement to the summation of forces and moments that cause slope movement. These forces and moments could result from increased loading or decreased resistance, which may be caused by variation in pore-water pressure and the buttressing effect induced by changes in river levels. The point of "stability" is defined as a factor of safety equal to 1.0, where the driving forces equal the resisting forces, indicating theoretical failure.

4.3.1 SLOPE/W Parameters

Field and laboratory testing was conducted on native materials from the site to evaluate shear strength parameters under drained and undrained conditions. The following sections summarize the reasoning for the selected parameters.

4.3.1.1 Shallow Lean Clay

The undrained shear strength of the shallow lean clay were estimated from laboratory testing, hand penetrometer testing, and correlations to SPT testing. A summary of the laboratory testing was provided in Section 3.4. The shallow lean clay at the site typically has moderate SPT values and hand-penetrometer values. Hand-penetrometer testing indicated that the undrained shear strength ranged from 250 psf to greater than 4,500 psf, with most values exceeding 1,250 psf. An undrained shear strength of 1,250 psf was used for the shallow lean clay at the site.

The drained shear strength of the shallow lean clay was estimated based on correlations to the soil's plasticity index, provided by Terzaghi et al (1996). Laboratory testing results indicated that the plasticity index for the shallow lean clay ranged from 16 to 18 percent. This corresponds to a friction angle ranging from approximately 31 to 32 degrees. A drained shear strength (i.e., friction angle) of 31 degrees was used for the shallow lean clay.

Under rapid drawdown scenarios, the pore pressure is anticipated to remain elevated, while the buoyant force from the water is removed. Laboratory testing was not extensively performed on the shallow lean clay soil because it is relatively thin at the project site. For rapid drawdown scenarios, the effective stress parameters used for the shallow lean clay were a friction angle of 31 degrees and no apparent cohesion, and the total stress parameters were a friction angle of 30 degrees and an assumed cohesion of 100 psf.

The moist unit weight of the shallow lean clay was estimated to be approximately 110 pcf, and the saturated unit weight was estimated to be approximately 115 pcf.

4.3.1.2 Clay Embankment Fill

The shear strength of the embankment fill was estimated largely on the results of testing for the shallow lean clay soil, which is anticipated to be a suitable borrow source for the project, although a borrow pit location has not been identified at this time.

The drained shear strength of the embankment soils was estimated to be similar to the shallow lean clay, which was based on correlations to the soil's plasticity index, provided by Terzaghi et al (1996). Laboratory testing results indicated that the plasticity index for the shallow lean clay ranged from 16 to 18 percent. This corresponds to a friction angle ranging from approximately 31 to 32 degrees. A friction angle of 31 degrees was used for the clay embankment fill.

The undrained shear strength was assumed to be 1,000 psf for the clay embankment fill. This should be confirmed during final design for the project, once a borrow source has been identified.

The shear strength of the clay embankment fill during rapid drawdown conditions was assumed to be similar to the parameters used for the shallow lean clay. For rapid drawdown scenarios, the effective stress parameters used for the clay embankment fill were a friction angle of 31 degrees and no apparent cohesion, and the total stress parameters were a friction angle of 30 degrees and an assumed cohesion of 100 psf.

The recompacted moist unit weight was assumed to be 110 pcf (assuming that the backfill is compacted to approximately 95 percent of the maximum dry density according to standard Proctor), and the saturated unit weight was estimated to be approximately 115 pcf.

As part of the final design, additional laboratory testing should be performed to develop a more accurate determination of the shear strength of the recompacted clay used for the actual proposed embankment fill material.

4.3.1.3 Silt

For the purposes of this analysis, the silt was treated as a cohesionless drained material, since the clay content was relatively low. The shear strength of the silt was estimated from correlations to SPT testing (Das, 2007).

SPT test results in the silt layers ranged from 2 to 7 blows per foot. Based on the correlation, the friction angle was approximately 28 degrees. A friction angle of 28 degrees was used for both undrained and drained scenarios.

For rapid drawdown scenarios, the effective stress parameters used for the silt were a friction angle of 28 degrees and no apparent cohesion, and the total stress parameter was a friction angle of 27 degrees and a cohesion of 100 psf.

The moist unit weight of the silt was estimated to be 105 pcf, and the saturated unit weight was estimated to be approximately 110 pcf.

As part of the final design, additional laboratory testing should be performed to develop a more accurate determination of the silt shear strength.

4.3.1.4 Fat Clay

The undrained shear strength of the fat clay was estimated from laboratory testing. A laboratory testing summary was provided in Section 3.4. Based on the test results, the fat clay had an undrained shear strength ranging from 680 to 1,240 psf, with an average of 910 psf. The average value of 910 psf was used for analysis.

The drained shear strength of the fat clay was estimated based on laboratory direct shear testing and laboratory consolidated undrained triaxial compressive strength testing. Laboratory direct shear testing was also performed on two intact fat clay samples, which indicated that the peak friction angle ranged from 5.9 to 15.5 degrees. The samples also exhibited an apparent cohesion ranging from 310 to 682 psf.

The failure envelope selected for the fat clay from a laboratory triaxial compressive strength test indicated that the drained friction angle was approximately 24.5 degrees. Plotting all the test results indicated that the behavior was generally similar and matched closely with correlations to the fully softened shear strength developed by Stark and Hussain (2013) at low normal stresses, which depend on the clay fraction and liquid limit. As the triaxial test is considered a more refined test method, the drained shear strength used for analysis was a friction angle of 24.5 degrees.

Under rapid drawdown scenarios, the pore pressure is anticipated to remain elevated, while the buoyant force from the water is removed. A consolidated undrained triaxial compressive strength test was performed on one fat clay sample to simulate these conditions. Using the maximum deviator stress as the failure criteria, test results indicated that the effective friction angle was 24.5 degrees. The effective stress parameter used for the fat clay was a friction angle of 24.5 degrees, and the total stress parameters were a friction angle of 11.7 degrees and a cohesion of 360 psf based on laboratory testing using the maximum deviator stress as the failure condition.

The moist unit weight of the fat clay was estimated from laboratory testing to be approximately 104 pcf, and the saturated unit weight was estimated to be approximately 107 pcf.

4.3.1.5 Glacial Till

The undrained shear strength of the glacial till was estimated from laboratory testing. A laboratory test summary was provided in Section 3.4. Based on test results, the glacial till had an undrained shear strength ranging from 2,550 to 4,360 psf. An undrained shear strength of 2,550 psf was used for analysis.

The drained shear strength of the glacial till was estimated based on correlations to the soil's plasticity index, provided by Terzaghi et al (1996). Laboratory test results indicated that the plasticity index for the glacial till ranged from 16 to 23 percent. This corresponds to a friction angle ranging from approximately 30 to 32 degrees. A friction angle of 30 degrees was used for the glacial till.

Under rapid drawdown scenarios, the pore pressure is anticipated to remain elevated, while the buoyant force from the water is removed. No testing was performed in the intact glacial till, but one consolidated undrained triaxial compressive strength test was performed on a reworked glacial till sample to simulate these conditions, which are considered applicable to the stiffer glacial till. Using the maximum deviator stress as the failure criteria, test results indicated that the effective friction angle was 36.8 degrees, with no apparent cohesion. The effective stress parameters used for the glacial till were a friction angle of 30 degrees and no cohesion, and the total stress parameters were a friction angle of 29 degrees and a cohesion of 1,460 psf, using the maximum deviator stress as the failure condition.

The moist unit weight of the glacial till was estimated from laboratory testing to be approximately 120 pcf, and the saturated unit weight was estimated to be approximately 125 pcf.

4.3.1.6 Reworked Glacial Till

The undrained shear strength of the reworked glacial till was estimated from laboratory testing. A laboratory test summary was provided in Section 3.4. Based on test results, the reworked glacial till had an undrained shear strength ranging from 1,170 to 1,950 psf. An undrained shear strength of 1,170 psf was used for analysis.

The drained shear strength of the reworked glacial till was estimated based on correlations to the soil's plasticity index, provided by Terzaghi et al (1996). Laboratory test results indicated that the plasticity index for the reworked and stiffer glacial till (since the plasticity was similar) ranged from 16 to 23 percent. This corresponds to a friction angle ranging from approximately 30 to 32 degrees. A friction angle of 30 degrees was used for the reworked glacial till.

Under rapid drawdown scenarios, the pore pressure is anticipated to remain elevated, while the buoyant force from the water is removed. A consolidated undrained triaxial compressive strength test was performed on a reworked glacial till sample to simulate these conditions. Test results indicated that the effective friction angle was 36.8 degrees, with no apparent cohesion. The effective stress parameters used for the glacial till were a friction angle of 30 degrees and no cohesion, and the total stress parameters were a friction angle of 29 degrees and a cohesion of 1,460 psf based on laboratory testing.

The moist unit weight of the reworked glacial till was estimated from laboratory testing to be approximately 110 pcf, and the saturated unit weight was estimated to be approximately 115 pcf.

4.3.1.7 Sand

Because the sand at the project site was very isolated and limited in quantity, analysis was performed for the more typical cases for the project, and sand was not included in the analyses.

4.3.1.8 Summary of Shear Strength Parameters

The soils were treated as Mohr-Coulomb materials in the modeling program using the parameters in the table below:

Table 4-3 Shear Strength Parameters

			Drained Condition (ESSA)		Undrained Condition (USSA)		RDD Condition		
Material Type	Moist Unit Weight [pcf]	Saturated Unit Weight [pcf]	Friction Angle [deg]	Undrained Shear Strength [psf]	Friction Angle [deg]	Undrained Shear Strength [psf]	Total Stress Friction Angle [deg]	Total Stress Undrained Shear Strength [psf]	Effective Stress Friction Angle [degrees]
Embankment Fill	110	115	31	0	0	1000	30	100	31
Shallow Lean Clay	110	115	31	0	0	1250	30	100	31
Fat Clay	104	107	24.5	0	0	910	11.7	360	24.5
Silt	105	110	28	0	28	0	27	100	28
Reworked Glacial Till	110	115	30	0	0	1170	29	1460	30
Glacial Till	120	125	30	0	0	2550	29	1460	30

4.3.2 Stability Analysis

The slope stability analyses were conducted using SLOPE/W, a computer modeling program developed by GEO-Slope International. SLOPE/W uses limit equilibrium theory to compute the factor of safety (FOS) of earth and rock slopes. It is capable of using a variety of methods to compute the FOS of a slope while analyzing complex geometry, stratigraphy, and loading conditions. The pore-water pressure head file produced by SEEP/W during seepage analysis was imported into SLOPE/W to compute effective stress. As a result, this approach incorporates the effect of pore pressures when computing the FOS.

Pore-water pressures for the slope stability calculations are computed from the flow net during the SEEP/W analyses. Therefore, the integration of SEEP/W seepage pore-water pressures in a SLOPE/W analysis results in a more accurate calculation of factor of safety than traditional limit equilibrium software, which uses a phreatic line to simulate groundwater.

4.3.2.1 Factor of Safety Calculation and Requirements

Spencer's method was used to calculate the FOS of the slopes in this stability analysis. This method is typically used because it satisfies both the force and moment equilibrium in determining the factor of safety. For typical long-term conditions (ESSA) under steady seepage without seismic forces, Barr used the minimum recommended factor of safety of 1.5 based on requirements from the NRCS (NRCS, 2005). Barr used the minimum recommended end-of-construction (or short-term case, USSA) factor of safety of 1.3 (NRCS, 2005), where pore pressure within the soil has not dissipated when subjected to a shear force. This is recommended for both upstream and downstream slopes. For the hydraulic loading conditions where the water will reach the height of the embankment crest, a long-term factor of safety of 1.4 was used, since this is considered to be a less-likely loading condition (EM 1110-2-1902, 2003). For the rapid draw down case, where the water drains out quickly but the pore-water pressure remains in the slope, a factor of safety of 1.2 is recommended (NRCS, 2005), assuming that the water is at the freeboard height, which is considered more likely to occur than a significant draw down from the embankment crest height. Rapid draw down conditions from the full embankment height were assumed to not be considered routine for this site and proposed levee embankment alternatives.

Primarily circular potential failure surfaces were used in the analysis. Potential failure surfaces were defined using the entry and exit method. This allows the location of the trial slip surfaces to be chosen manually, or where it is anticipated to enter and exit the ground surface, with a selected number of entry and exit points.

4.4 Results of Slope Stability Modeling

Limit equilibrium stability modeling results are provided in this section. For these modeling scenarios, a minimum slip surface thickness of 2 feet was used, therefore small-scale surface sloughing was not considered in the analysis as surficial failures should not affect overall slope stability (commonly assumed to be the maintenance condition). This global stability case is identified in the summary tables.

The assumptions made for the two cross sections analyzed were provided at the beginning of this section.

4.4.1 Slope Stability Results at Cross Section 1 – Alt1

The analysis for Cross Section 1 at Alt1 was performed for an embankment configuration assuming downstream and upstream side slopes of 4H:1V, a crest width of 10 feet, and a crest height of 959.0 feet. Based on the analyses completed, the dam configuration meets the required factors of safety for all analyzed hydraulic loading scenarios, with the exception of the drained condition under maximum flooding conditions (Analysis 3.1).

The failure envelope used for the embankment fill in the analysis is likely conservative. Use of a modest amount of cohesion in the geotechnical model increased the factor of safety to meet the recommended value (Analysis 3.1a). Therefore, additional strength testing should be performed on the proposed embankment fill material from the final borrow pit location during the final design to verify that the shear strength of the material meets what was estimated in the model.

Table 4-4 summarizes the various analyses performed and corresponding factors of safety.

Table 4-4 Slope Stability Results for Cross Section 1

Analysis No.	Scenario	Upstream Side Slope	Down-stream Side Slope	Upstream Water Elevation	Embank-ment Height [feet]	Downstream FOS	Upstream FOS	Required FOS
1.1 / 1.2	ESSA; No Flood	4H:1V	4H:1V	--	8.1	2.37	2.37	1.50
1.3 / 1.4	USSA; No Flood			--		6.92	6.45	1.30
2.1 / 2.2	ESSA; Normal Flood			956.0		1.50	2.20	1.50
2.3 / 2.4	USSA; Normal Flood			956.0		6.75	10.04	1.30
3.1 / 3.2	ESSA; Maximum Flood			959.0		1.22 (1.45 with 100 psf cohesion)	2.61	1.40
3.3 / 3.4	USSA; Maximum Flood			959.0		6.46	14.44	1.30
4.1	Rapid Draw Down			956.0		--	2.35	1.20

The model outputs for Cross Section 1 are included in Appendix E1.

4.4.2 Slope Stability Results at Cross Section 2 – Alt2

The analysis for Cross Section 2 for Alt2 was performed for an embankment configuration assuming downstream and upstream side slopes of 4H:1V, a crest width of 10 feet, and a crest height of 969.0 feet. Based on the analyses completed, the dam configuration does not meet the required factor of safety for the ESSA case under normal and maximum flood conditions in the downstream direction (Analysis 2.1 and 3.1).

The failure envelope used for the embankment fill in the analysis is likely conservative. Use of a modest amount of cohesion (110 psf) in the geotechnical model increased the factor of safety to meet the recommended level (Analysis 2.1a and 3.1a). Therefore, additional strength testing should be performed on the proposed embankment fill material from the final borrow pit location during the final design to verify that the shear strength of the material meets what was estimated in the model.

Table 4-5 summarizes the various analyses performed and corresponding factors of safety.

Table 4-5 Slope Stability Results for Cross Section 2

Analysis No.	Scenario	Upstream Side Slopes	Down-stream Side Slopes	Upstream Water Elevation	Embank-ment Height [feet]	Downstream FOS	Upstream FOS	Required FOS
1.1 / 1.2	ESSA; No Flood	4H:1V	4H:1V	--	12.0	2.46	2.43	1.50
1.3 / 1.4	USSA; No Flood			--		4.62	4.26	1.30
2.1 / 2.2	ESSA; Normal Flood			966.0		1.23 (1.62 with 110 psf cohesion)	2.30	1.50
2.3 / 2.4	USSA; Normal Flood			966.0		3.52	7.54	1.30
3.1 / 3.2	ESSA; Maximum Flood			969.0		1.05 (1.40 with 110 psf cohesion)	2.79	1.40
3.3 / 3.4	USSA; Maximum Flood			969.0		3.28	9.02	1.30
4.1	Rapid Draw Down			966.0		--	2.43	1.20

The model outputs for Cross Section 2 are included in Appendix E2.

4.4.3 Commentary on Slope Stability Analysis Results

Based on Barr's experience, other slopes constructed on the high plasticity clay of the Red River Valley typically are designed at flatter slopes—on the order of 5H:1V or 6H:1V. To achieve the required factors of safety with more conventional side slopes, significant construction methods would be required to improve or properly construct the slopes and reinforcement may be needed. If Moore decides to pursue these options, Barr can provide guidance, but construction of flatter slopes may be more cost-effective and easier to implement during construction.

The analysis performed to date assumed that steady-state conditions are present, which is considered unlikely to be the case, as the water is anticipated to drain quickly. A transient analysis would more closely represent the anticipated conditions, but additional information would be required to perform the analysis. Transient analyses can also be difficult to calibrate.

The embankments could also be designed using a zoned embankment, or a filter blanket or drain could be used on the downstream slope to draw the phreatic surface away from the downstream face of the slope. Design and implementation of these methods are considered relatively minor with regard to the cost of the entire structure and are considered appropriate options during final design depending on which alternative is selected for development.

4.5 Settlement of Existing Soils Due to New Embankment

The construction of an embankment on native soil will increase stress on the soils. As mentioned in Section 3.2, the clay soils are likely saturated at shallow depths due to the anticipated shallow water table and high moisture contents. As such, the clay soils are anticipated to experience long-term consolidation settlement, as well as immediate, elastic settlement due to the weight of the fill used to construct the embankment. Barr performed settlement estimations based on the anticipated embankment configurations. Settlement was estimated at the center of the embankment, where the impact of the increased load is greatest. The total settlement of a levee or embankment is not necessarily limited by existing codes, but the total settlement of the embankment should be considered during final design to ensure that the required height of the embankment does not fall to below the anticipated hydraulic conditions (i.e., maximum groundwater height and required freeboard).

4.5.1 Long-Term Settlement from Consolidation Test Results

The subsurface conditions encountered during the field work indicated that the material encountered at the locations of Alt1 and Alt2 generally consists of layers clay and silt. The groundwater, as observed during the soil borings, was as shallow as 11 feet below the existing grade, but may be shallower based on the presence of primarily clayey soils, which due to the low permeability, likely did not allow seepage into the borehole in the short amount of time the borehole was open for measurement of an accurate long-term groundwater level.

The long-term settlement of clay soils supporting the embankment can be computed using consolidation characteristics and the following equation:

$$S = \frac{C_r}{1+e_o} \cdot L \cdot \log\left(\frac{\sigma'_p}{\sigma'_{vo}}\right) + \frac{C_c}{1+e_o} \cdot L \cdot \log\left(\frac{\sigma'_f}{\sigma'_p}\right) \qquad \text{(Das, 2007)}$$

where:

C_r = recompression index

C_c = compression index

e_o = initial void ratio

L = height of soil layer

σ'_p = maximum past effective stress where soil transitions from overconsolidated to normally consolidated

σ'_{vo} = existing effective stress at the midpoint of the clay layer below embankment

σ'_f = final effective stress equal to $\sigma'_{vo} + \Delta\sigma'$, where $\Delta\sigma'$ = average pressure increase to the clay layer caused by the added load

Using this formula, the long-term settlement of an embankment can be calculated. To calculate the consolidation settlement, the soil was split into multiple layers, with the effective stress recalculated at the midpoint of each layer. The stress dissipates at greater depth in the ground according to the Poulos and Davis method (FHWA, 1974). The total depth of calculation was taken as twice the approximate base width of the embankment.

Based on consolidation test results as discussed in Section 3.3.5 and an assumed loading consistent with the embankment design assumed for this report, settlement was estimated for the two cross sections evaluated. Therefore, the analysis consisted of layers of soils with variable compressibility, which closely estimates the in-situ conditions. The parameters selected represent the anticipated properties of the clay soils at the site based on a review of all available laboratory consolidation data.

The results of the consolidation analysis indicated that the estimated total long-term settlement ranged from 5.0 to 6.3 inches, as indicated in Table 4-6.

Table 4-6 Summary of Settlement Analysis

Cross Section ID	Estimated Long Term Settlement at Center of Crest [inches]
CS1	5.0
CS2	6.3

The actual total long-term settlement will likely be slightly higher. The immediate, or elastic settlement, was not considered for this analysis, but will likely be realized during construction. Therefore, the total long-term settlement is estimated to be a maximum of 7 to 9 inches at the center of the embankment, depending on location and embankment height. A minimum 9-inch overbuild would be recommended for settlement concerns (not including freeboard, superiority, etc.).

4.6 Additional Geotechnical Considerations

The following sections describe some additional considerations for further design of the levee embankment alternatives.

4.6.1 Slope Protection

It is recommended that slope protection be utilized for the constructed embankment. The slope protection should be selected to avoid erosion of the newly constructed embankment, particularly along any slope that will be exposed to moving water during flood events. Slope protection could consist of vegetation, rip-rap, or turf reinforcement. Barr recommends use of a more resilient method (i.e., rip-rap or turf reinforcement) on the upstream slope due to the rural location, potential for erosion due to contact with flood waters, and limited inspection anticipated for the project once constructed.

4.6.2 Seismic Site Requirements

The following seismic design criteria are recommended for the design of structures at this site based on the 2012/2015 International Building Code (IBC) (USGS, 2018). The seismic values below are recommended for both Alt1 and Alt2 at the site.

- S_s = 0.050 *g* (Site Class B)
- S_1 = 0.020 *g* (Site Class B)
- Recommended Site Classification: Site Class D

A Site Class D is recommended for foundation design at the site. The above seismic values need to be adjusted accordingly for Site Class D for structural design (if required). However, seismicity in this area is generally low and likely will not control the design.

4.6.3 Cement Type

The results from the tests indicate that the soluble sulfate content ranged from 1,640.0 to 2,360.0 mg/kg, which indicates severe sulfate exposure (ACI, 2014). If concrete structures are used for the project, cement with an exposure class of S2 is recommended.

4.6.4 In-Situ Shrink/Swell Potential

The shrink/swell potential of a soil is related to its liquid limit and plasticity index. Soils with liquid limit values less than 50 and plasticity index values less than 25 are considered to have low shrink-swell potential. Soils with liquid limit values of 50 to 60 and plasticity index values of 25 to 35 are considered to have moderate shrink-swell potential. Soils with liquid limit values greater than 60 and plasticity index values greater than 35 are considered to have high shrink-swell potential (Das, 2007).

Based on laboratory test results, the measured range of liquid limit values was 33 to 107 percent, the measured range of plasticity index values was 18 to 32 percent, and the plasticity index ranged from 4 to 80 percent. Therefore, some soils at the site are considered to have a high shrink-swell potential, particularly those identified as fat clay in the boring logs, which was encountered at some locations as shallow as 2 feet and extended to a maximum depth of approximately 31 feet.

One laboratory swell test was performed on a sample of fat clay and indicated that the maximum free swell was 5.4 percent and the corresponding swell pressure was 0.73 tsf. This corresponds to no potential swell (swell under a loading equal to the overburden stress). As a result, the embankment will provide a higher bearing pressure than simply the overburden stress, and shrink/swell of the subgrade should not affect the embankment design. Care should be taken by the contractor to prevent significant moisture content change during construction to avoid drying and cracking of the soil.

Discussion of shrink/swell potential of the potential fill material is discussed in Section 4.6.7.

4.6.5 Earthwork Shrink-Swell Factor

The soils will have an earthwork shrink-swell factor and this should be considered during determination of final design quantities. A typical preliminary estimate of 15 to 25 percent shrinkage can be used for the feasibility cost analysis.

4.6.6 Frost Depth

The frost penetration depth for the proposed alignment is a depth of 72 inches (US Army, 1992). The frost depth is not anticipated to affect the proposed embankment, but structures or infrastructure associated with the proposed embankment should be protected from frost to a depth of at least 6 feet. As a general recommendation, fill should not be placed on frozen subgrades, and frozen materials should not be used as fill.

4.6.7 Dispersion Potential

Dispersive soils have their parcels disassociate with some amount of particles going into suspension when immersed in relatively still water. Silt and clay particles exhibit dispersion when the repulsive forces between the particles exceed the attractive forces when saturated. These particles are then carried away with flowing water, weakening the soils and creating seepage paths. For embankments and other water-retention structures, the dispersion potential of the foundation and embankment soils should be addressed, as saturation of the soils may lead to dispersion and internal erosion (Maharaj, 2013). Silt and clay soils were observed in the project site soil borings. Silt soils often have a lower fraction of clay particles, lack capillary forces within the soil structure, and are at greater risk for dispersion. In general, the dispersion potential of clayey glaciolacustrine and glacial till is considered to be low due to the higher clay content.

The dispersion potential of the silt was measured through laboratory double hydrometer testing to be approximately 2 percent. According to Elges (1985), dispersion ratios less than 15 percent are considered non-dispersive. Therefore, the silt can be considered non-dispersive based on the available data, although additional testing should be performed during the final design.

In general, the hydraulic loading conditions on the proposed embankment are anticipated to be relatively short, and steady-state conditions may not develop during the short loading periods, which is not considered likely to lead to an internal erosion failure. In addition, the silt layers were observed at depths of 11 to 12 feet below the existing grade and not near the surface. Therefore, the velocity gradient of groundwater at those depths is likely to be very low. Accounting for the available information, the risk of dispersion of the silt is considered low for the project site for the perceived function with no normal upstream pool. Using a properly filtered drainage blanket on the lower portion of the downstream slope would further reduce the potential for piping and internal erosion.

As part of the final design, dispersion potential of the proposed embankment material should be performed.

4.6.8 Selection of Embankment Fill Material

The results of the borings indicate that there is a thin mantle of shallow lean clay underlain by high plasticity fat clay. The shallow lean clay nearer to the surface has lower moisture content and very occasional sand seams, but appears suitable for use as borrow material. However, the limited thickness may not provide enough volume to construct the embankment. It is recommended to construct the embankment out of homogeneous material to avoid differences in soil behavior and performance. Additional borings should be performed to evaluate the potential borrow source and material volume.

The fat clay at the site may be used as fill, but would likely need to be moisture conditioned to dry the material to an acceptable level to be placed. Consideration could be given to placing the material at slightly above the optimum moisture content in order to allow the material to shrink in the event that it dries out. If designed and placed properly, the embankment may not shrink below the desired crest elevation and require a re-build. Conversely, under hydraulic loading events, which are anticipated to be of short durations (although the duration has not been provided to Barr), the embankment should have a low enough permeability such that swelling of the soil should not lead to swelling and cracking. If the fat clay is used, it would be recommended to monitor the crest height of the embankment after completion of construction to ensure that the embankment is behaving as planned.

Use of the fat clay as fill may also require flatter slopes given the high plasticity and lower shear strength of the material. This may lead to a need for more volume to construct the embankments.

5 Summary

5.1 Summary

Barr was retained by Moore to complete a preliminary geotechnical investigation and feasibility-level geotechnical evaluation of the Amenia Levee Embankment Alternatives Alt1 and Alt2. Upon the completion of the field investigation and subsequent laboratory testing, Barr performed geotechnical modeling and evaluation of representative cross sections for each alternative. In addition, Barr analyzed the long-term settlement along the proposed levee embankment alternative.

The results of the analysis indicate that the long-term settlement is estimated to range between about 5 to 7 inches. Actual total settlement will likely be on the order of 7 to 9 inches taking into account immediate elastic settlement.

Seepage and slope stability modeling results indicate that an embankment configuration using 4H:1V downstream side slopes and 4H:1V upstream side slopes for Alt1 and Alt2 is generally suitable pending further laboratory testing to confirm the drained shear strength of the embankment and foundation soil. The computed factors of safety indicate that slope stability and seepage are expected to meet recommended values.

The shallow lean clay should be further evaluated for shear strength and permeability during final design, but this material should be suitable for use as embankment fill. Additional investigation should be performed to identify locations where this material exists based on proximity to whichever alternative is considered. Use of the fat clay will likely result in a need for flatter slopes, which will lead to higher construction costs. The condition of using the fat clay for embankment material was not evaluated for this report.

5.2 Future Geotechnical Investigation and Analysis

As part of the design phase geotechnical investigation, Barr recommends the following program to further evaluate the potential alternatives:

- CPT soundings in between the previously investigated soil borings along the final alignment to a depth of 40 feet.
- Flat plate dilatometer testing (DMT) soundings at locations along the proposed final alignment to a depth of 40 feet to determine settlement estimations.
- Pore pressure dissipation testing (PPD) at locations along the proposed final alignment to a depth of 40 feet to estimate in-situ permeability.
- Soil borings coinciding with select CPT soundings to verify lithology and to collect additional samples for laboratory testing. Conversely the CPT soundings could be performed near the location of the soil borings completed for this investigation.

- Evaluation of hydraulic conductivity of the silt soil along Alt2.
- Soil borings and laboratory testing to evaluate potential borrow sources.
- Installation of standpipe or vibrating wire piezometers along the alignment to determine the long-term groundwater level and monitoring of pore-water pressure during construction.
- Shear and compression wave velocity testing to determine elastic soil parameters.
- Additional seepage and slope stability modeling to verify that the assumptions in this report were correct and to evaluate additional critical cross sections (if necessary).
- Identification and further testing of potentially dispersive soil if identified.
- Evaluation of shrink/swell potential if fat clay is considered for use as embankment fill.
- Evaluation of groundwater control during construction via test pits.

6 Limitations of Analysis

This report is for the exclusive use of Moore Engineering, Inc. Without written approval by Barr, no responsibility to other parties regarding this report is assumed. Barr's evaluation, analysis, and recommendations may not be appropriate for other parties or projects. The proposed designs and analysis provided herein should be considered for preliminary use only and will need to be verified prior to implementation.

No established national standards exist for data retrieval and geotechnical evaluations. Barr has used the methods and procedures described in this report, which generally comply with NRCS recommendations (NRCS, 2005). In performing its services, Barr used the degree of care, skill, and generally accepted engineering methods and practices ordinarily exercised under similar circumstances and under similar budget and time restraints by reputable members of its profession currently practicing in the same locality. Reasonable effort was made to characterize the project site based on the site-specific field work, however, the analyses represent a large area, and variations in stratigraphy, strength, and groundwater conditions from any of the locations at which testing was performed may occur. It is important that engineering and operations personnel regularly observe the pond slopes and embankments and note any changes in strata or water conditions as these may require modification of the mine operation requirements to maintain slope stability. No warranty of the investigation, analysis, or design presented herein, expressed or implied, is made.

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