Innovative approaches
Practical results
Outstanding service

SHAWNEE CITY LAKE NO. 1 DAM PRELIMINARY ENGINEERING REPORT

Prepared for:

City of Shawnee

January 2025

Prepared by:

FREESE AND NICHOLS, INC.
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Tulsa, Oklahoma 74135
(539) 444-8677

SWN24427

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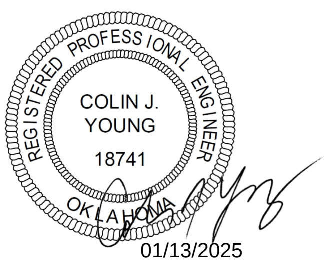

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EXECUTIVE SUMMARY

City of Shawnee, Oklahoma (City) retained Freese and Nichols, Inc. (FNI) in June 2024 to assist the City in response to certain actions required by a November 2023 consent order issued by Oklahoma Water Resources Board (OWRB). The scope of the work included the following major tasks:

1. Visual site assessment,
2. Limited topographic survey to assist in geotechnical and hydrologic/hydraulic (H&H) evaluations,
3. Historical document review,
4. H&H studies to determine spillway capacity with respect to OWRB requirements,
5. Independent seepage and slope stability analysis of the embankment using existing data from a 2024 report prepared by Terracon Consultants,
6. Preparation of this Preliminary Engineering Report (PER) which includes results of above tasks, recommendations for further analysis, monitoring, and/or repairs as applicable, and peer review of the 2024 Terracon report.

FNI performed a visual assessment on July 3, 2024 and found the spillway and embankment to be in Fair condition based on the OWRB condition rating terminology.

The Terracon investigations and the Consent Order were promulgated by the 2022 annual dam inspection that indicated a slope failure was occurring along the downstream slope near the spillway. FNI examined this area visually and did not conclude that a slope failure had occurred. FNI concluded that the repaired area reflects a localized pavement surface course and subgrade failure. The pavement had been repaired before FNI's site visit.

In the opinion of FNI, the geotechnical strength parameters used in the 2024 Terracon report for slope stability analysis are conservative and are not consistent with values of similar soils on other dams in the state. The Terracon results are also inconsistent with a dam that has performed satisfactorily without a slope stability incident since it was constructed in 1936. Seepage, slope stability, and H&H analyses were conducted by FNI, and the results indicate that the dam is in compliance with the Title 785, Chapter 25 requirements for spillways and embankment dams.

Through the visual assessment and the engineering evaluations, FNI did not identify any dam safety issues that need to be addressed immediately. There were no adverse seepage observations, nor any visual indication of past or current slope instability other than historically noted riprap sloughs in the upper portion of the upstream slope.

FNI recommends an updated breach analysis with modern 2D modeling techniques to better document hazard rating and quantify downstream impacts. To assess the post seismic performance of the dam, a liquefaction and loss of strength analyses/study may be performed. If existing soils are determined to liquefy or lose strength through seismic loading, a post-seismic slope stability and deformation analysis is recommended.

To improve the physical condition of the dam and limit further deterioration, FNI recommends riprap repair/replacement to the upstream slope, clearing and grubbing of the lower downstream embankment slope, along with some other minor items listed in Section 7. The full scope of recommended repairs are estimated to cost on the order of $3 to $3.5 million, based on 2024 construction costs for similar work in the region. The cost estimate range includes replacing upstream slope riprap along nearly the full length of dam, clearing and grubbing of the lower downstream slope, installing seepage monitoring devices, and adding riprap scour protection upstream of the spillway drop structure. Engineering design and permitting services are not included in these costs.

1.0 INTRODUCTION

In June 2024, the City of Shawnee (City) contracted Freese and Nichols, Inc., (FNI) to perform certain engineering services for Shawnee City Lake No. 1 Dam in response to a Consent Order issued by the Oklahoma Water Resources Board (OWRB) on November 30, 2023. This Order was in response to an October 7, 2022 inspection report citing observations of a slippage crack, longitudinal crack, and depressions in the pavement on the dam crest (Terracon, 2022). OWRB considered the dam to be in poor condition based on that inspection. Prior to FNI's involvement, the City contracted with Terracon Consultants, Inc. to conduct a geotechnical and geophysical investigation of the embankment. During a July 20, 2023 virtual meeting between the City, OWRB representatives, and consultants from Terracon, the dam was found to be in an unsafe condition, and it was recommended that the water level be lowered until additional analysis could be conducted (OWRB, 2023). The City was given until December 31, 2024, to begin repair construction measures on the dam.

FNI's scope of services under this contract include limited topographic surveys, evaluation of hydraulic spillway capacity, visual assessment of the dam, independent seepage and slope stability analysis using existing data, peer review of the Terracon geotechnical engineering report, and dam safety repair/rehabilitation recommendations (Terracon, 2024).

2.0 SHAWNEE CITY LAKE NO. 1 DAM

Shawnee City Lake No. 1 Dam, which forms Shawnee City Lake No. 1, is owned and maintained by the City of Shawnee (City) and is located in Potawatomie County, west of Shawnee, Oklahoma. Construction on the earthen embankment was completed in 1936. The dam is 2,570 feet long and 55 feet high above the stream bed of the south branch of South Deer Creek. It has a storage capacity of 36,500 acre-feet (ac-ft) according to the National Inventory of Dams (USACE, 2024). The dam is equipped with a 320-foot-wide uncontrolled concrete spillway which was originally set to elevation 1,074.2 feet (NAVD88) but raised with a 1-foot ogee weir to 1,075.2 feet when Shawnee City Lake No. 2 Dam was constructed in 1960. The two dams are connected by an excavated "equalization" channel, to the left of the spillway. The concrete ogee weir spillway serves both dams. An earthen spillway on the northeast end of Shawnee City Lake No. 2 Dam serves as the auxiliary spillway for both dams.

Design drawings, provided by the City, show an upstream slope of 3H:1V and a downstream slope of 2.5H:1V and a 15-foot top width; however, those design drawings were modified to incorporate a two-lane road across the top of the dam. The two-lane road modification yielded a top width of 33 feet and side slopes at the uppermost portion of the dam at 1.5H:1V on both the upstream and downstream slopes. This steepened slope section eventually ties into the respective slope of the dam. From the design drawings (Figure 1), FNI concluded that either the decision was made to widen the crest late in the design process, or the concept was adopted as a money saving effort on embankment fill quantity. This oversteepening of the uppermost portion of the embankment has contributed to some of the localized sloughing of the riprap along the upstream slope.

The design drawings also indicate a rock-fill downstream toe; however, the geotechnical/geophysical investigations reflect soil materials rather than rock fill (Figure 1) (Terracon, 2024). It is assumed that the dam designers planned this as a waste area for excessive rock excavation during construction. Visual observations and soil probing to a depth of 3 to 4 feet by FNI during the July 2024 site inspection did not detect the presence of a rock-fill toe.

Available station references for the dam consist of a profile view of the dam embankment in the historical design drawings, consisting of stations -3 to +26. Using negative stationing and attempting to reference accurate locations in reference to the dam embankment is therefore difficult. To aid with this, FNI created a new dam alignment and associated station references, which are used throughout this report. See Figure 2 for alignment and stationing.

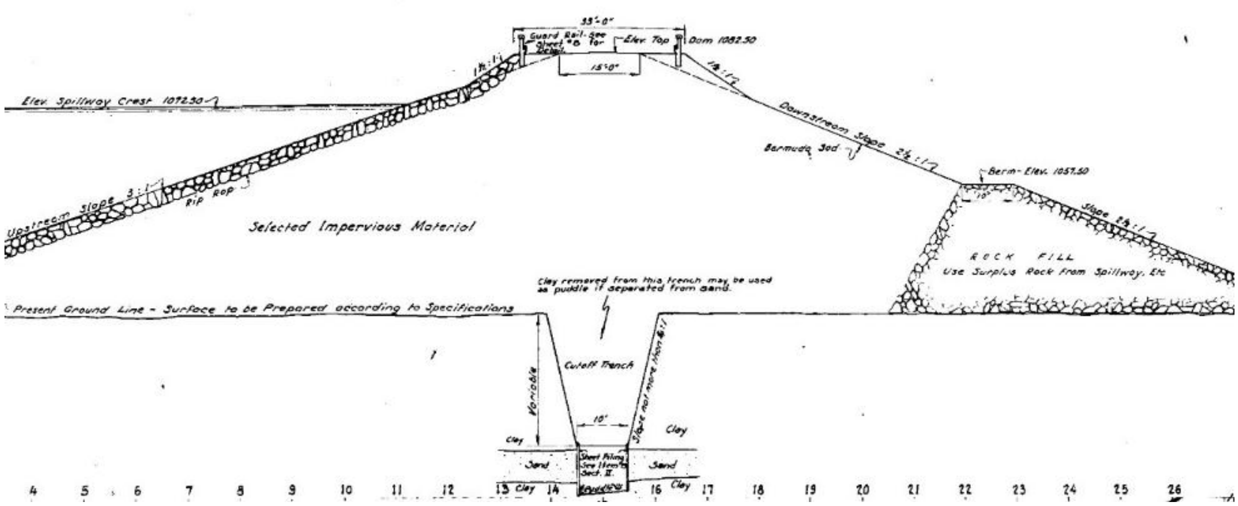


Figure 1. Dam 1 Design Drawing Section (Geocon, 1978)

Available dam configuration information is summarized in Table 1.

Table 1. Dam Construction Information

Main Embankment[1]		
Type		Earth fill/Rockfill[2]
Length		2,570 feet
Maximum Height		55 feet
Crest Width		33 feet
Top Elevation		1,084 feet (nominal)
Upstream Slope		3H:1V / 1.5H:1V[3]
Downstream Slope		2.5H:1V / 1.5H:1V[3]
Service Spillway		
Type		Uncontrolled concrete channel-lined with a concrete flip bucket stilling basin to a natural channel
Location		Left Abutment
Width		320 feet
Crest Elevation		1,075.2 feet (est.)
Channel Side Slopes		Vertical
Service Outlet/ Plant Intake		
Type		Concrete Intake Tower
Location		Sta 23+00[4] (approx.)
Conduit		48-inch diameter welded steel
Emergency Spillway		
Type		Earthen
Location		Left Abutment of Dam #2
Width		500 feet
Crest Elevation		1,079.5 (est. from survey)
Channel Side Slopes		Natural, varying from 22:1 on the right to 48:1 on the left
Reservoir		
Normal Pool Elevation		1,075.2 feet (est.)
Storage at Normal Pool		22,600 ac-ft
Storage at Top of Dam		36,500 ac-ft
General		
OWRB Size Classification		Intermediate
OWRB Hazard Classification		High

1. Data from National Inventory of Dams, 2024 topographic survey, and site visits.
2. Rockfill indicated on the design drawings, but no rockfill has been identified.
3. Normal Constructed Slope / Slope at top to facilitate the roadway.
4. Stationing by FNI based on

Figure 2. Shawnee City Lake No. 1 Dam Alignment and Stationing

In Oklahoma, the OWRB is responsible for the administration of the state dam safety laws. Dams are classified according to size, and the potential for loss of human life and/or property damage within the area downstream of the dam. The small, intermediate, or large size classifications are identified by the reservoir storage capacity and the embankment dam height. Intermediate size dams are those which have a maximum storage of the dam between 10,000 ac-ft and 50,000 ac-ft and a maximum height between 50 feet and 100 feet. Shawnee City Lake No. 1 Dam is classified as an intermediate size dam meeting both the height and storage criteria.

Dams are classified by the potential loss of human life and/or property damage within the area downstream of the dam. The dam hazard classification is identified as low, significant or high depending on the potential downstream impacts from a dam failure. Shawnee City Lake No. 1 Dam is currently classified as a high hazard potential structure by OWRB.

2.1 OWRB DESIGN CRITERIA

Shawnee City Lake dams are regulated by OWRB. As such, the embankment and spillway must follow design requirements set forth in OAC Title 785, Chapter 25, Section 785:25-3-11 which requires the following safety factors to be met for earthen embankments:

- Steady State Seepage at Emergency Spillway Crest (Surcharge Pool): 1.5
- Rapid Drawdown (RDD) from Principal Spillway: 1.2
- Earthquake/Seismic: 1.0

For the seismic stability case, the upstream slope should consider the reservoir at the service spillway elevation; while the downstream slope should consider steady state seepage when the reservoir is maintained at the emergency spillway crest.

OWRB rules require intermediate size dams constructed prior to June 13, 1973, to safely pass or contain the 50% Probable Maximum Flood (PMF) with no minimum freeboard. Shawnee City Lake No. 1 Dam was constructed in 1936 and, thus, must adhere to these criteria. Changes were made to the service spillway, including the installation of the flip bucket stilling basin and raising the crest 1 foot, in the early 1960s, pre-dating the June 13, 1973, rule.

2.2 SPILLWAYS AND SERVICE INTAKES

Shawnee City Lake Nos. 1 and 2 Dams share both service and emergency spillways to pass storm flows. The service spillway is a 320-foot-wide concrete ogee weir spillway on the left abutment of Shawnee City Lake No. 1 Dam. The emergency spillway is a 720-foot-wide earthen channel near the left abutment of Shawnee City Lake No. 2 Dam. The crest of the emergency spillway is approximately 4.3 feet above the crest of the ogee weir on the service spillway. Downstream of the ogee weir is a concrete and native rock-lined section ranging in length (in direction of flow) from about 100 feet on the right to 160 feet on the left leading to a 268-foot-wide concrete flip bucket stilling basin. This feature was added in the early 1960s. Below this structure is an earthen and native rock channel.

Shawnee City Lake Nos. 1 and 2 Dams provide water to the City of Shawnee from a 10-foot inner diameter (ID) reinforced concrete tower equipped with upper, middle, and lower 24-inch intakes. A 48-inch (ID) welded steel-lined concrete conduit transmits water from the tower to the water treatment plant. Original design drawings show the presence of five concrete seepage collars around the conduit on 50-foot spacings.

2.3 SURVEYS

In 2024, FNI contracted Civil & Environmental Consultants, Inc. (CEC) to survey select areas of Shawnee City Lake No. 1 Dam, including the service spillway, the emergency spillway on the north end of Shawnee City Lake No. 2 Dam, and the highly vegetated area at the downstream toe of the dam embankment. Existing surveys were obtained from the City, including some bathymetry in Lake No. 1. Existing LiDAR and terrestrial LiDAR gathered by CEC were also used in these analyses. The surveys were performed in Oklahoma State Plane Coordinate System, North Zone. The horizontal datum is North American Datum of 1983 (NAD83 - 2011), and the vertical datum is North American Vertical Datum of 1988 (NAVD88 - Geoid12B). The purpose of the survey was to obtain accurate elevation data for use in hydraulic modeling for Shawnee City Lake No. 1 Dam and to support geotechnical analysis and eventual design. There are some gaps between the CEC survey and the City-provided data, but they do not materially affect the analyses in this report.

3.0 VISUAL SITE ASSESSMENT

FNI performed a visual inspection of Shawnee City Lake No. 1 Dam on July 3, 2024. Colin Young, P.E.; Chris Stoner, P.E.; Wade Anderson, P.E.; Taylor Green, P.E.; Christian Capehart; Isaac Davis, E.I; Taylor

DiGiacinto, E.I.; and Victoria Regits of FNI were joined by Zach Hollandsworth, P.E.; Natalie Orbesen, P.E.; and Luis Peralta, E.I. from OWRB. The visual inspection included the embankment and spillway areas. These areas were also videoed with a small unmanned aerial vehicle (drone). The lake level was estimated to be 1,072 feet, approximately 3 feet below the spillway crest. Weather conditions were clear, with temperatures in the 90s (degrees Fahrenheit), and were suitable for inspection. General conditions and observations from this inspection are outlined in the following sections of this report and photos from the inspection are in Appendix A. This visual site assessment is not meant to replace the OWRB-required annual inspection. For consistency of terms, however, FNI utilized the FEMA/OWRB condition assessment terminology below in the assessment of the embankment and spillway.

Satisfactory - No existing or potential dam safety deficiencies are recognized. Acceptable performance is expected under all loading conditions (static, hydrologic, seismic) in accordance with the applicable regulatory criteria or tolerable risk guidelines.

Fair - No existing dam safety deficiencies are recognized for normal loading conditions. Rare or extreme hydrologic and/or seismic events may result in a dam safety deficiency. Risk may be in the range to take further action.

Poor - A dam safety deficiency is recognized for loading conditions which may realistically occur. Remedial action is necessary. Poor may also be used when uncertainties exist as to critical analysis parameters which identify a potential dam safety deficiency. Further investigations and studies are necessary.

Unsatisfactory - A dam safety deficiency is recognized that requires immediate or emergency remedial action for problem resolution.

3.1 EMBANKMENT OBSERVATIONS

The embankment appeared to be in overall fair condition. No significant erosion, cracks, or slides were observed along either side of the embankment or crest.

3.1.1 Crest

The crest is in overall satisfactory condition. The crest is covered with a two-lane asphalt road. There is some cracking due to age/weathering and traffic loading in wheel lanes, and patching was present in several areas. Guard rails exist on each side of the road and appear to have sustained damage from previous impacts. Power line poles along the downstream side of the crest appear to be leaning downstream. This is common on the crest of embankment dams due to slight downhill creep of the near surface soils over time and the shallow burial depth of the poles. This in and of itself is not a dam safety issue. Review of historical annual inspection reports indicate poles have been leaning for many years. Depressions or low areas of concern were not observed along the crest of the dam.

3.1.2 Upstream Slope

The upstream slope is in overall fair condition. The slope is covered in riprap; however, there are areas along the slope with missing and mis-matched (type and size) riprap and minor vegetation. Some large pieces of concrete are also present among the riprap. Because of the exaggerated mismatched sizes, there are several gaps in the riprap that allow embankment soil to be eroded (see Figure 3). There was no visual evidence of proper granular bedding material under the riprap. Slope steepening exists in localized areas along the upstream slope caused by sloughing. As previously mentioned in Section 2.0, the original over steepened design of the upper portion of the slope has contributed to the localized sloughing noted in previous dam inspections.

Figure 3. Mis-matched riprap and voids

3.1.3 Downstream Slope

The downstream slope is in overall fair condition. Adequate grass coverage exists above the downstream berm. Below the berm, overgrown woody vegetation and trees prevented a thorough visual inspection of the lower slope, and the thick canopy has prevented adequate grass coverage. See Figure 4 for a photo of the downstream slope.

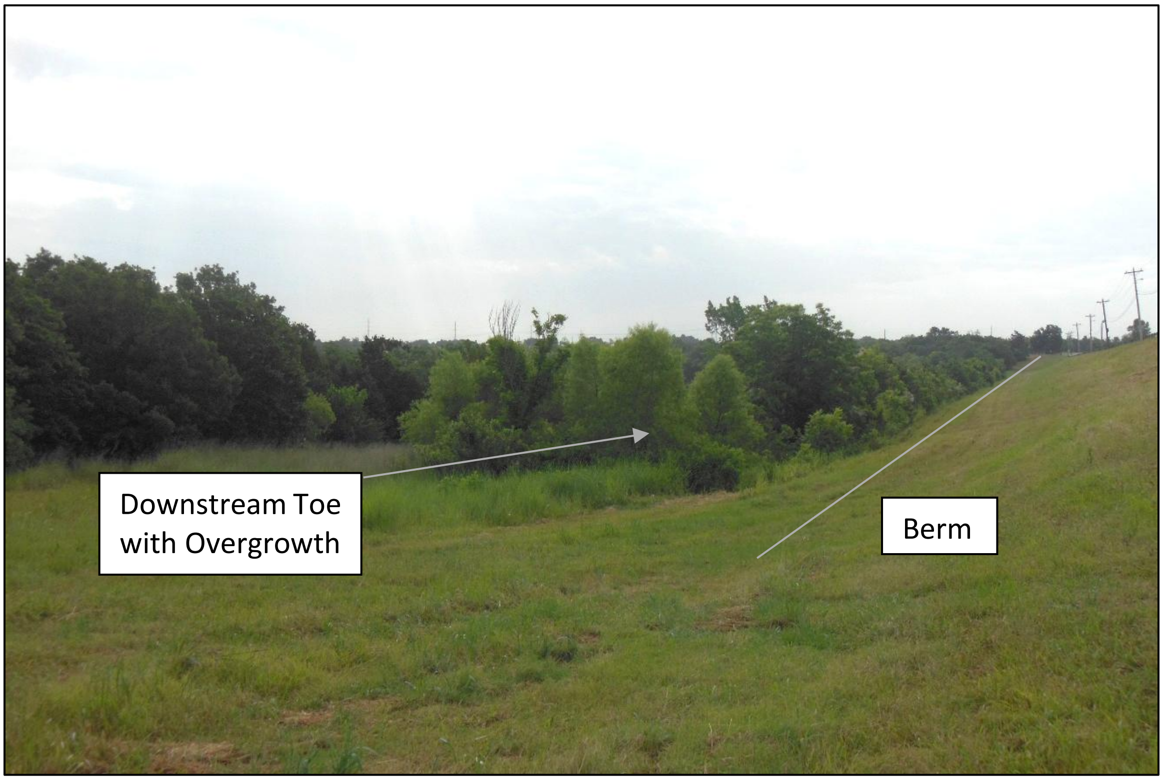


Figure 4. Left abutment looking downstream

Occasional small animal burrows were seen along the downstream slope above the berm. Some wet areas and possible seepage were observed along the downstream toe. The seepage water was mostly clear and not carrying soil materials. It should be noted that during a brief site visit by FNI in March 2024, the entire toe area was inundated by standing water. During the July inspection, however, standing water was not present along the toe in the mid-section of the dam, only near the abutments. This observation indicates that much of the water along the toe is from standing seasonal rainwater and not persistent seepage.

3.1.4 Spillway Observations

The spillway was observed visually and included manual hammer soundings of the concrete surfaces. The ogee crest section appeared in satisfactory condition with only minor cracking and spalling. The ogee consists of considerably newer concrete than the adjoining apron which was part of the original construction. Drawings indicate that the apron is at least 2 feet thick, and possibly thicker in some areas. That concrete exhibited typical surface erosion (loss of paste) expected of exposed concrete of that age.

There is some delamination and several previous thin patches that have also delaminated. This delamination and weathering are not severe enough to warrant an extensive repair project at this time, and thin patches will not be successful due to constant weather exposure. Based on the visual observations, the apron will eventually need to be replaced, but it is currently in serviceable condition.

The service spillway training walls appeared to be in satisfactory condition with no observations of abnormal cracking, displacement, or concrete deterioration. The spillway channel between the end of the concrete apron and the flip bucket stilling basin has some localized scour just upstream of the back wall of the flip bucket, some of which have been filled with riprap. Although not an immediate dam safety issue, this erosion should be addressed before it worsens with future spillway flows. Severe scour in this area could eventually impact the integrity of the structure.

4.0 GEOTECHNICAL EVALUATION

The subsurface conditions were explored within 16 geotechnical borings along the dam crest and the bench along the downstream slope at Shawnee City Lake No. 1 Dam in 2023 and 2024. The boring logs and laboratory tests associated with these borings are contained in the Shawnee City Lake Dam No. 1 Investigation Geotechnical Engineering Report prepared by Terracon (Terracon, 2024). These borings were designated as B-1 through B-17 (B-4 was not performed in the program) and the exploration depths in feet below ground surface (ft bgs) are summarized within Table 2.

Table 2. Boring Schedule

Boring	Location	Terminal Depth (ft bgs)
B - 1	Crest	12.0
B - 2	Crest	12.0
B - 3	Slope	29.0
B - 5	Slope	24.0
B - 6	Crest	94.0
B - 7	Crest	95.0
B - 8	Crest	99.0
B - 9	Crest	99.0
B - 10	Crest	64.0
B - 11	Crest	39.0
B - 12	Slope	70.0
B - 13	Slope	69.0
B - 14	Slope	69.0
B - 15	Slope	74.0

Boring	Location	Terminal Depth (ft bgs)
B - 16	Slope	59.0
B - 17	Slope	29.0

The drilling investigation activities were performed by Terracon in two phases between April 2023 and January 2024. Phase I focused on the west end of the dam near the spillway bridge and included four borings (B-1, B-2, B-3, and B-5) ranging in depths from 12 to 29 feet. Phase II included the rest of the dam and the additional 12 borings. Each boring was drilled using solid-stem flight augers and casing to maintain the hole. Once groundwater was encountered, mud rotary wash was used to maintain the hole. Samples were retrieved using a split-spoon sampler in conjunction with Standard Penetration Tests (SPT), or thin-walled steel tube samplers.

4.1 GEOPHYSICAL EXPLORATION

A geophysical survey was conducted for the embankment and subsurface lithology as part of Terracon's Phase I and Phase II embankment studies. Cone Penetrometer Tests (CPTs) soundings and geophysical evaluations using Multichannel Analysis of Surface Waves (MASW) cross sections were performed. Geophysical testing and CPTs are typically calibrated with traditional geotechnical investigations and laboratory testing. The 2024 Terracon report does not describe calibration of geophysical tests data with the results of the conventional borings (Terracon, 2024).

4.1.1 Generalized Subsurface Conditions

The Shawnee City Lake No. 1 Dam embankment subsurface stratigraphy is interpreted as fill overlying a residual foundation material derived from the underlying shale and sandstone. The fill material encountered in the crest borings generally consisted of clayey sand, silty sand, and lean clay having various amounts of silt and sand. Borings along the berm encountered primarily sandy materials having various amounts of silt and clay. Groundwater was observed within borings along the embankment crest and varied between depths of approximately 25 to 31.5 ft bgs at the time of drilling.

Foundation materials encountered within the soil test borings consisted primarily of residual lean clays with varying amounts of sand and silt (CL) and layers of silty sand (SM). Underlying bedrock material was described as poorly cemented to well cemented, weathered sandstone or soft to moderately hard, highly weathered to weathered shale.

4.1.2 Geology

Shawnee City Lake No. 1 Dam is located just northwest of the City of Shawnee, Oklahoma. According to geologic maps developed by the Oklahoma Geological Survey, the dam is located within the Wellington Formation which consists of interbedded sandstone, claystone, and concretionary clay-shale, with minor siltstone and sandstone breccias locally. (Oklahoma Geological Survey, 2021) . The geologic maps also indicate the stream channel along South Deere Creek contains unconsolidated alluvial deposits from the Holocene period consisting of locally derived clay, silt, sand, and rarely gravel sized sedimentary material (Oklahoma Geological Survey, 2021).

4.1.3 Groundwater

Depth to groundwater was measured within each soil test boring. Groundwater levels were observed during auger drilling, immediately after, and 24 hours after completion of drilling. Groundwater observations are listed in the 2024 Terracon report and were compared to the phreatic water surface generated by the seepage and slope stability model software in this study (Terracon, 2024).

4.2 SEISMICITY

4.2.1 Faults

To identify potential faults near the site, FNI reviewed both the Oklahoma Fault Database and the Comprehensive Fault Database (Oklahoma Geological Survey, 2015). Based on these sources, Shawnee City Lake No. 1 Dam is situated west of faults that trend southwest to northeast from Pauls Valley, OK to Cleveland, OK. The closest unnamed fault runs east to west through the City of Shawnee. Based on available records, the USGS Quaternary Fault Map suggests that the fault is not considered active (U.S. Geological Survey, 2022).

4.2.2 Design Earthquake Criteria

Dams and other large structures are often designed to resist the ground motion that corresponds to a seismic event with return period of between 2,475 years and 10,000 years based on consequences of failure during a seismic event, as defined in Technical Release TR 210-60 Earth Dams and Reservoirs (U.S. Department of Agriculture, March 2019).

- **Low Consequence**.—Dams in rural or agricultural areas where seismic failure of the dam or appurtenance may result in damage to farm buildings, agricultural land, or township and country roads.
- **Significant Consequence**.—Dams in predominantly rural or agricultural areas where seismic failure of the dam or appurtenance may result in damage to isolated homes, main highways or minor railroads, and interrupt service of relatively important public utilities. For high hazard potential and significant hazard potential dams, significant consequence of a seismic failure of the dam or appurtenance may also include the loss of stored water from water supply dams where there is no other water supply source. For high hazard potential and significant hazard potential dams, significant consequence also includes the loss of the permanent pool where a seismic failure of the dam or appurtenance would have significant consequences such as the loss of important recreational facilities or environment damage.
- **High Consequence**.—Dams where seismic failure of the dam or appurtenance may cause loss of life or serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

Figure 5. Level of Consequence as defined by TR 210-60

FNI considered the Shawnee City Lake No. 1 Dam to be a "Significant Consequence" dam (Figure 5), because the dam is in a rural area, and there would likely be isolated damage to homes and infrastructure if the dam were to seismically breach at normal pool. Shawnee City Lake No. 1 Dam is a high hazard potential dam, and the lake is a water supply reservoir for the City of Shawnee, which further supports selection as a "Significant Consequence" dam, if breached during a seismic event. For a 'Significant Consequence" structure, a 5,000-year return period is recommended by TR 210-60. The 2013 breach analysis does not provide enough detail to definitively assign a consequence category, therefore, FNI elected to evaluate seismic stability of the dam with the ground motions associated with a 1% chance in 100 years event (10,000-yr return period), which is a stronger ground motion event.

4.2.3 Response Spectrum and Site Class

FNI queried in December 2024 the United States Geological Survey (USGS) Deaggregation tool, as the ASCE Hazard Tool recommended by ASCE 7-22 was limited to 2,500-year ground motion at the time of this report. Consistent with ASCE 7-16, the selected Peak Ground Acceleration (PGA) and Uniform Hazard Response Spectrum (UHRS) for the 10,000-year return period was selected from the Dynamic: Conterminous 2014 Data Set (Latitude: 35.34832, Longitude: -97.06440). Figure 6 presents the UHRS for the hypothetical rock outcrop (termed the B/C boundary) for the site, which indicates that PGA and spectral acceleration at one second (S_1) are 0.3162g and 0.670g, respectively.

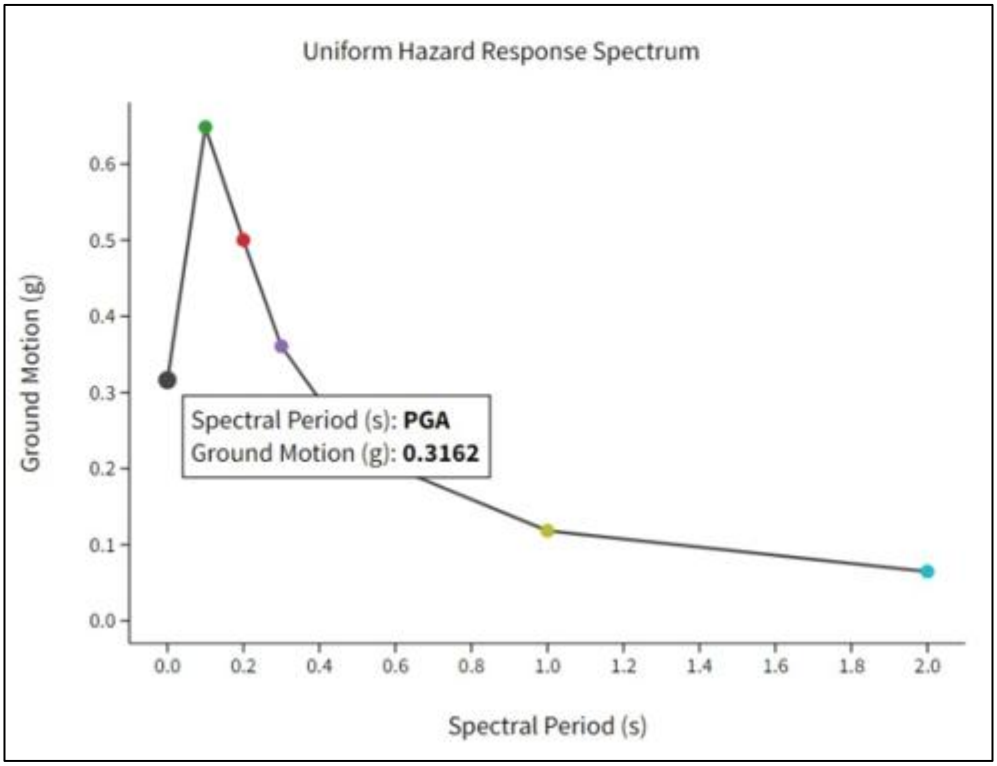


Figure 6. Uniform Hazard Response Spectrum for 10,000-year Return Period Ground Motions

Within the crest borings, bedrock was encountered approximately 40 feet beneath the embankment. Layers of lean clay and silty sand were identified between the embankment and bedrock. ASCE 7-16 defines Seismic Site Class D as stiff soils with a range of velocity of 600 ft/sec to 1200 ft/sec with blow counts of between 15 and 50. Based on the MASW data from B-8 and B-14 and the boring logs in the 2024 Terracon report, the average shear wave velocity in the foundation of the dam is in the range of 353.5 ft/sec and 1234 ft/sec. Blow counts for the foundation soils below the dam were between 3 and 88 with an average of approximately 19. Most SPT values for the site fall between the 15 to 50 range. Based on these qualifications, FNI assigned a Seismic Site Class D to the site.

4.2.4 Engineering Parameter Interpretation and Selection

FNI reviewed in-situ and laboratory data collected during the geotechnical investigation performed by Terracon in 2023 and 2024 to develop engineering parameters for the existing soils. Hydraulic conductivity parameters for existing strata were selected based on the USCS soil classification, depositional or construction history, engineering correlations, and laboratory tests. FNI did not utilize the geophysical test data when developing material parameters.

Seepage analyses were conducted using partially saturated soil mechanics as modeled by the soil-water characteristic curve (SWCC). Functions recommended by Fredlund, Xing, and Huang were applied to estimate the SWCC based on grain size data, liquid limit, the estimated saturated water content, and the horizontal hydraulic conductivity at saturation (Fredlund, Xing, & Huang, 1994).Appendix B.

Table 3 presents the selected horizontal hydraulic conductivity (permeability) for each layer. The selection of parameters is discussed within Appendix B.

Table 3. Selected Hydraulic Seepage Parameter Summary

Soil Zone	Material	Permeability		K_v/K_h Ratio
		K_h (cm/sec)	K_h (ft/sec)	
Embankment Fill	Lean Clay (CL)	5.0E-08	1.6E-08	0.25
Embankment Fill	Clayey Sand (SC)	4.0E-05	1.3E-06	0.25
Embankment Fill	Clayey Sand, Silty Sand (SC/SM)	2.0E-05	6.0E-07	0.25
Embankment Fill	Silty Sand (SM)	3.0E-04	9.8E-06	0.33
Foundation Soils	Lean Clay (CL)	2.0E-06	6.6E-08	0.25
Foundation Soils	Clayey Sand (SC)	4.0E-05	1.3E-06	0.25
Foundation Soils	Clayey Sand, Silty Sand (SC/SM)	2.0E-05	6.0E-07	0.25
Foundation Soils	Silty Sand (SM)	3.0E-04	9.8E-06	0.33
Bedrock	Shale	1.01E-07	3.3E-09	0.1
Bedrock	Sandstone	9.14E-07	3.0E-08	1

For cohesionless soils (i.e., sands and gravels), correlations based on SPT results were used to select representative effective stress shear strength parameters. For cohesive soils, index parameters from laboratory tests and fully softened shear strength curves developed by Castellanos et al. were applied to select representative effective stress shear strength parameters from established correlations and ranges (Castellanos, Brandon, & VandenBerge, 2016). Total (CU) strength values for cohesive soils were selected assuming the total cohesion to be twice the drained cohesion and assuming the total friction angle is about two-thirds the drained friction angle. The selected parameters are summarized within Table 4, while data and methodologies applied to assign material parameters are provided in Appendix B.

Table 4. Selected Shear Strength Parameters

Soil Zone	Material	Unit Weight (pcf)	Shear Strength Parameters		
			Parameter	Effective Strength	Total Strength
Embankment Fill	Lean Clay (CL)	125	Friction angle, deg	25	17
			Cohesion, psf	150	300
Embankment Fill	Clayey Sand (SC)	120	Friction angle, deg	27	18
			Cohesion, psf	50	100
Embankment Fill	Clayey Sand/Silty Sand (SC/SM)	120	Friction angle, deg	27	-
			Cohesion, psf	-	-
Embankment Fill	Silty Sand (SM)	120	Friction angle, deg	30	-
			Cohesion, psf	-	
Foundation Soils	Lean Clay (CL)	125	Friction angle, deg	27	18
			Cohesion, psf	100	200
Foundation Soils	Clayey Sand (SC)	130	Friction angle, deg	32	-
			Cohesion, psf	50	-
Foundation Soils	Clayey Sand/Silty Sand (SC/SM)	125	Friction angle, deg	26	-
			Cohesion, psf	-	-
Foundation Soils	Silty Sand (SM)	130	Friction angle, deg	33	-
			Cohesion, psf	-	-
Bedrock	Shale	130	Friction angle, deg	16	-
			Cohesion, psf	400	-
Bedrock	Sandstone	134	Friction angle, deg	24	-
			Cohesion, psf	950	-

4.3 CROSS SECTION GEOMETRY

FNI developed two representative cross sections of the embankment dam from the 2024 topographic survey, historical survey and bathymetric data, and field observations. Cross sections were selected based on previously identified areas of suspected displacement and the tallest cross section. The following sections were analyzed for both existing conditions and are labeled based on dam stationing FNI developed for this report (and proceeds from right to left):

- STA 5+50: Left section near spillway
- STA 17+00: Center section

The external geometry of the upstream slope to the downstream toe was developed from a topographic survey prepared by CEC in 2024 and a bathymetric survey provided by the City of Shawnee within AutoCAD Civil 3D. The downstream slopes were typically 2.5H:1V and the upstream slopes were typically 3H:1V and were steepened to between 1.5 and 2H:1V near the top of the slope at an approximate elevation of 1,071 ft.

The subsurface stratigraphy was developed from soil test borings and geophysical investigation results, which delineated the transition from embankment to residual foundation soils. For borings in the tallest embankment sections, bedrock was encountered within each boring at an approximately 40-foot depth beneath the embankment dam. Bedrock consisted of shale or sandstone both originating from the Wellington Formation. The soil test borings indicated that the residual soils consist of a conglomerate of lean clays and silty sands derived from the weathering of the shale and sandstone found within the Wellington Formation.

The soils test borings and laboratory testing indicated layers of sandy lean clay to lean clay within the center of the embankment based on minor differences in fines content within the samples. Due to similarities in index properties, the center embankment section material was combined into a single representative region for analysis resembling the impervious material zone shown in the original as-built drawings. The left section of the embankment included layers of lean clay and clayey sand and were modeled as individual stratums. The borings and laboratory test data for soil comprising the downstream embankment berm which is described on the as-builts as rock-fill material indicated mostly clayey silty sands with layers of lean clay. This zone of material was also combined into a single representative region for analysis. The foundation materials were comprised of alternating layers of lean clays, clayey sands, and silty sands. Due to similarities in index properties, the sandy lean clays and lean clays were combined to form a single region while the silty sand was modeled as its own layer in the seepage and stability model.

4.4 SEEPAGE ANALYSIS

FNI performed seepage analyses to estimate the phreatic surface in support of slope stability analysis and to estimate seepage pressures to analyze whether the embankment is susceptible to internal erosion mechanisms. The analyses were performed within the SEEP/W software program developed by

GeoStudio, Inc., and the following subsections describe the boundary conditions and results for each case. Phreatic surfaces estimated from the seepage analysis generally aligned with the groundwater observations made during Terracon's investigation. Therefore, the phreatic surfaces developed from the seepage analysis were used for the stability models. .

4.4.1 Hydraulic Boundary Conditions

Hydraulic boundary conditions were assigned to model seepage within the embankment to develop a phreatic surface and to estimate the head/pressure or the flux/flow at specific locations. FNI assigned boundary conditions for the upstream slope (reservoir) based on the noted reservoir elevation provided in the survey data for the spillway crest. A boundary condition of a potential seepage face was assigned along the downstream slope and was modeled with a water flow rate of zero cubic feet per second (cfs) and seepage face review. Seepage was analyzed without consideration of existing tailwater conditions Table 5 summarizes the boundary conditions for the embankment cross section.

Table 5. Hydraulic Boundary Conditions

Location	Boundary Condition	Value
Upstream Slope and Ground Surface	Normal Pool Elevation	Head = 1,075.2 ft
Downstream Slope and Ground Surface	Potential Seepage Face	Water Rate = 0 cfs (w/ Seepage Face Review)
Bottom of Model and Downstream Vertical Surface	Zero Flux Boundary	Water flux = 0 cfs/sf

4.4.2 Results

The cross-section geometry developed within Section 4.2 was used in SEEP/W and the material properties and hydraulic boundary conditions provided in Table 3 and Table 5, respectively, were assigned as applicable. The computed phreatic surface was compared to water levels from the boring logs found in the 2024 Terracon report (Terracon, 2024). The predicted phreatic surface was similar to water levels listed in the boring log data and was utilized for all further analyses. Flow nets and pressures were developed along the cross section which were utilized for slope stability analyses and to evaluate internal erosion susceptibility.

4.4.3 Internal Erosion

The long-term earthen embankment stability is influenced by pore pressure distribution through the embankment. The computed pressures are evaluated to assess the potential for internal erosion.

Heave is a process where pressures develop directly below the embankment in a cohesionless foundation layer. During heave, vertical seepage forces reduce the effective stress in the cohesionless layer just downstream of the embankment. This creates a sudden volume and permeability increase and can cause displacement of soils due to heave pressure (such as sand boils).

Backward erosion, or simply piping, is a concentrated erosion process due to seepage. Once the erosion occurs, the seepage is concentrated, and erosion continues in the upstream direction such that a small pipe or tunnel is formed. For this type of erosion to occur, the following must happen: (1) there must be an essentially continuous, nearly horizontal layer of material, (2) it must possess enough cohesion or structure that it can form a roof, and (3) it must have an unfiltered exit point. An industry-wide accepted method is not available for this process, but one method used for evaluating the potential for piping is Seepage Severity, which was utilized herein (Duncan, O'Neil, Brandon, & Vanden Berge, 2011).

FNI analyzed the associated safety factors for these potential failure modes for existing conditions. The U.S. Bureau of Reclamation recommends safety factors for potential seepage failure modes in *Design Standard No. 13: Embankment Dams* (U.S. Bureau of Reclamation, 2014). For heave, a safety factor of 4.0 (new dams) to 3 (existing dams) is considered reasonable. A flow threshold of less than 2.2 x 10^{-5} cfs/ft of head/foot of embankment was established by the Center for Geotechnical Practice and Research (CGPR) Report No. 64 (Duncan, O'Neil, Brandon, & Vanden Berge, 2011). Below this flow rate, seepage erosion risk is considered negligible even for high exit gradients. FNI evaluated the internal erosion potential at the downstream toe for heave. FNI calculated the vertical exit gradient and factor of safety against heave at three depths and evaluated the flux through the entire cross section at the downstream toe and 100 feet away from the downstream toe. The seepage analysis results for the selected cross section are summarized in Table 6.

Table 6. Existing Condition Seepage Analysis

Section	Depth, ft	Reservoir Level, ft	Vertical Exit Gradient	Safety Factor for Heave	Seepage Severity, cfs/ft-head/ft embankment	Seepage Severity Category
West Section-Toe	1	1,075.2	0.03	Stable[1]	7.39E-12	Negligible
	3		0.02	Stable		
	5		0.01	Stable		
West Section– 100 ft	1		0.00	Stable	8.00E-14	Negligible
	3		0.00	Stable		
	5		0.00	Stable		
Center Section - Toe	1	1,075.2	0.34	3.16	6.79E-09	Negligible
	3		0.22	4.98		
	5		0.17	6.50		
Center Section – 100 ft	1		0.00	Stable	4.05E-10	Negligible
	3		0.00	Stable		
	5		0.00	Stable		

1. Stable indicates that the computed factors of safety are above 20.

The seepage analysis results indicate that the target factor of safety for heave near the downstream toe is met under existing conditions. Field observations indicated sparse and limited amounts of discharge at isolated areas near the abutments of the downstream toe.

4.5 SLOPE STABILITY ANALYSIS

Slope stability analyses were performed using the SLOPE/W software package developed by GeoStudio, Inc. which implements limit equilibrium analyses. Spencer's method, a method of slices approach which satisfies both force and moment equilibrium, was implemented to evaluate potential slip surfaces and identify if the computed safety factors for the existing embankment dam meet or exceed OWRB criteria.

4.5.1 Evaluation Cases

For the representative cross sections, slope stability analyses were evaluated considering the steady-state seepage results when the reservoir is maintained at an elevation of 1,075.2 feet. The conditions evaluated, shear strengths, and phreatic conditions for each scenario are described as follows:

- Steady State Seepage (SSS) of Downstream Slope: Effective stress shear strengths, which represent long term loading, were assigned to evaluate the embankment dam for steady state seepage while the reservoir is maintained at normal pool.

- Seismic Slope Stability: Seismic slope stability was evaluated for the upstream and downstream slopes and considered total stress (undrained) shear strengths assuming that steady state seepage and consolidation of subsurface soils exists. The seismic load was modeled using a pseudostatic approach, which applies a horizontal force to each slice equivalent to the weight of the soil slice times a pseudostatic coefficient. The method for calculating the pseudostatic coefficient is outlined in the following section.
- Rapid Drawdown (RDD): Stability during rapid drawdown was evaluated for the upstream slope using the three-stage approach developed by Duncan, et al. (Duncan, Wright, & Wong, 1990). This staged approach uses the stability analysis before drawdown at normal pool and after drawdown. In this method, effective and total shear strength parameters are assigned for cohesive soils. Only effective parameters are used for noncohesive soils.

FNI notes that total stress shear strength parameters are commonly reduced by 20% to account for strength loss during extended seismic shaking. Duncan, et al. notes that the strength reduction can be ignored because the peak ground acceleration for which the pseudostatic coefficient (k_h) is developed from occurs during the ground motion well before strength degradation of soils occurs and compounding the two effects is unnecessary (Duncan, Wright, & Brandon, 2014).

4.5.2 Pseudostatic Stability and Seismic Coefficient

Pseudostatic slope stability was performed considering both effective stress envelope shear strengths and undrained shear strengths, as defined in Section 4.4. For materials that freely drain, effective stress shear strengths were applied in both undrained and drained analyses. The method to select the pseudostatic coefficient is described below; while results are discussed in Section 4.5.4.

FNI utilized the displacement-based method proposed by FHWA/NCHRP 12-70 (2008) to select the horizontal pseudostatic coefficient (k_h) to evaluate seismic slope stability (NCHRP , 2008). The method considers the amplified peak ground acceleration within the embankment as the reference acceleration (a_{ref}) and applies an acceleration multiplier (a/a_{ref}) between 0.2 to 0.5 based on the acceptable displacement for the structure to compute the pseudostatic coefficient (k_h). For this analysis, FNI assumed that displacements up to 5 centimeters were considered acceptable.

To compute the reference acceleration (a_{ref}), the PGA at the hypothetical rock outcrop ($PGA_{B/C}$= 0.3162g) was amplified to predict the horizontal acceleration at the ground surface (termed "free-field") using guidance provided by the American Society of Civil Engineers (American Society of Civil Engineers, 2016). The $PGA_{B/C}$ for the 10,000-year seismic event was amplified by a factor (F_{PGA}) selected based on the site class. Based on the selected Site Class D, the interpolated F_{PGA} of 1.28 was utilized to compute a free-field PGA of 0.405g.

To compute the average maximum horizontal acceleration within the embankment, the PGA at the dam crest was estimated using empirical curves developed by Harder et al (1990). The maximum horizontal acceleration for a critical slip surface is estimated from Makdisi and Seed (1978) based on the slope height, and the critical slip surface sliding depth, which is selected termed the reference acceleration (a_{ref}) for analysis. FNI notes that the critical slip surface depth may be different based on the analysis approach (i.e., local vs. global stability) and may require nominal iterations. At the Site, the upstream (US) and downstream (DS) slopes were evaluated separately with separate reference accelerations. As suggested in FHWA (2011), the acceleration multiplier is selected based on allowable displacement and earthquake magnitude. For an earthquake magnitude of 5.7 and 5 centimeters of displacement, an acceleration multiplier of 0.5 was used to select the reference acceleration to a pseudostatic coefficient. For each analysis and slope, the selected coefficient is provided in Table 7.

Table 7. Horizontal Acceleration for 10,000-year Seismic Event

	PGA (B/C Boundary)	PGA (Dam Crest)	Maximum Horizontal Acceleration[1] (for Critical Surface)	k_h[2]
Center Section DS Slope	0.3162	0.670	0.228	0.114
Center Section US Slope			0.268	0.134
Left Section DS Slope			0.302	0.151
Left Section US Slope			0.262	0.131

1. MHA computed at the centroid of critical slip surface for the geometry.
2. Pseudostatic coefficient (K_H) computed as ½ of the reference acceleration (i.e. MHA) assuming 5 cm of displacement is acceptable.

4.5.3 Liquefaction or Cyclic Softening Screening

Although post-seismic stability analysis is not required by OWRB, foundation soils beneath the dam were evaluated to assess whether the strata are susceptible to liquefaction and loss of strength during a strong seismic event.

Soil types most susceptible to liquefaction include saturated or partially saturated, loose to moderately dense granular soils, such as silty sands or sands and gravels generally deposited in the Holocene Era. Fine-grained soils may also be susceptible to loss of strength during strong cyclic loading with sufficient load cycles – often termed cyclic softening.

Soils encountered within the dam footprint consisted primarily of silty sand and lean clays of varying sand content (Terracon, 2022). Several collected soil samples classified as silty sand (SM), clayey sand (SC), clayey silty sand (SC-SM), and poorly graded sand (SP) of varying clay content, which may be susceptible to liquefaction. Based on the boring logs, groundwater levels and associated SPT n-vales, several samples may be susceptible to liquefaction or cyclic strength loss during strong seismic shaking (Terracon, 2022). An evaluation of soils susceptible to liquefaction or loss of strength are further presented in Appendix B.

To assess the post-earthquake dam performance, a liquefaction triggering and strength loss evaluation may be performed. If existing residual soils are identified to potentially liquefy or lose strength through seismic loading, a post-seismic slope stability and/or deformation analysis is recommended.

4.5.4 Results

Slope stability analyses were performed for the evaluation cases described herein and results are provided within Table 8 and Table 9 for existing conditions.

Table 8. Existing Condition Slope Stability Analysis Results

Cross Section	Hydraulic Condition	Analysis Condition	Slope	Minimum Required FS	Calculated FS
Left Section STA 5+50	Normal Pool	Steady-State	D/S	1.5	1.67
	Normal Pool	Steady-State	U/S	1.5	1.64
	Normal Pool	RDD	U/S	1.2	1.70
Central Section STA 17+00	Normal Pool	Steady-State	D/S	1.5	1.53
	Normal Pool	Steady-State	U/S	1.5	2.07
	Normal Pool	RDD	U/S	1.2	1.30

Table 9. Existing Condition Pseudostatic Stability Analysis Results*

Cross Section	Strength Parameters	Hydraulic Condition	Analysis Condition	Slope	K_H	Minimum Required FS	Calculated FS
Left Section STA 5+50	Effective	Normal Pool	Pseudostatic	D/S	0.151	1.0	1.26
		Normal Pool	Pseudostatic	U/S	0.131	1.0	1.26
	Undrained	Normal Pool	Pseudostatic	D/S	0.151	1.0	1.26
		Normal Pool	Pseudostatic	U/S	0.131	1.0	1.26
Central Section STA 17+00	Effective	Normal Pool	Pseudostatic	D/S	0.114	1.0	1.13
		Normal Pool	Pseudostatic	U/S	0.134	1.0	1.59
	Undrained	Normal Pool	Pseudostatic	D/S	0.114	1.0	1.10
		Normal Pool	Pseudostatic	U/S	0.134	1.0	1.59

*All pseudostatic analyses were performed considering Site Class D and return periods of 2,500-yr, 5,000-yr and 10,000-yr. Results using the 10,000-yr return period are shown in the table. Factors of safety from the 2,500-yr and 5,000-yr return period exceed these values.

The calculated factors of safety satisfy the minimum requirements established by OWRB for the static steady-state seepage (1.5), rapid drawdown (1.2), and pseudostatic seismic analysis conditions (1.0). SEEP/W and SLOPE/W output files, which depict the flow lines or total head contours and critical slip surfaces, respectively, are provided within Appendix B.

5.0 H&H EVALUATION

The project included an updated hydrology and hydraulics analysis to confirm OWRB compliance since the last evaluation was conducted as part of the Phase I inspection (Geocon, 1978). The evaluation included both the Shawnee City Lake No. 1 Dam and Shawnee City Lake No. 2 Dam subbasins. Shawnee City Lake Nos. 1 and 2 Dams are hydraulically connected via an excavated channel that extends between the left abutment of Shawnee City Lake No. 1 Dam and the right abutment of Shawnee City Lake No. 2 Dam. The existing spillway system includes a 320-foot ogee spillway located on the left side of Shawnee City Lake No. 1 Dam, as well as an earthen emergency spillway located in the left abutment of Shawnee City Lake No. 2 Dam. In order to incorporate the channel interconnecting the two reservoirs, FNI developed a hydraulic model of the reservoir, dam, and spillway system within HEC-RAS.

5.1 HYDROLOGIC ANALYSIS

The U.S. Army Corps of Engineers' (USACE) Hydrologic Engineering Center Hydrologic Modeling Software (HEC-HMS) was utilized to develop runoff hydrographs for the ½ PMF, as well as other fractions of the PMF to use in the evaluation of the capacity of the existing dam and spillway (U.S. Army Corps of Engineers, 2023). The following sections describe the input parameters utilized within HEC-HMS to estimate the various flood discharges within the Shawnee City Lake Nos. 1 and 2 Dam subbasins.

5.1.1 Watershed Delineation

The watershed, or area of land that contributes runoff to the reservoir areas, was obtained from the U.S. Geological Survey (USGS) online application Stream Stats. The web-based tool utilizes geographic information system (GIS) routines to delineate the watershed boundary based on USGS quadrangle maps, which cover the contiguous United States (U.S.) at a 10-meter resolution. This boundary was then manually refined to a greater detail using National Resource Conservation Service (NRCS) 2-meter LiDAR collected in 2020. The contributing areas are shown in Table 10.

Table 10. Shawnee City Lake Dams' Contributing Areas

Drainage Basin	Short Identifier	Basin Area (mi^2)
Shawnee City Lake No. 1	SWN1	21.3
Shawnee City Lake No. 2	SWN2	11.5

An aerial map depicting the watershed boundaries is shown below in Figure 7.

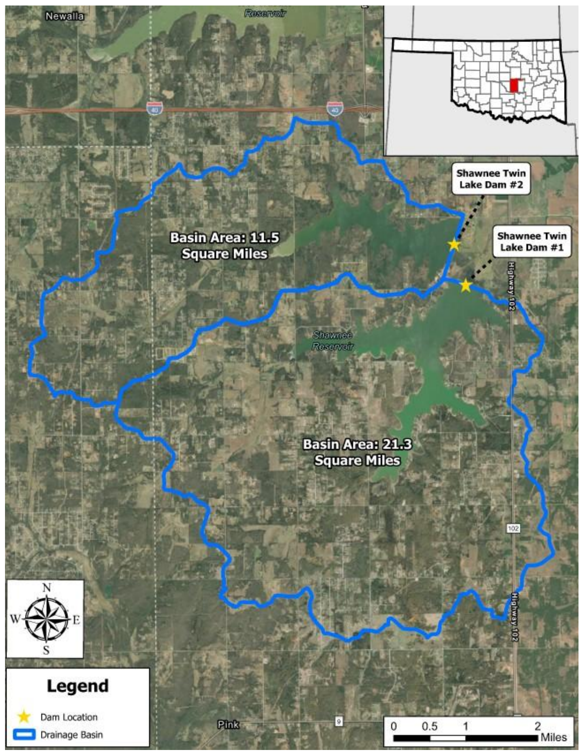


Figure 7. Shawnee Drainage Basin Map

5.1.2 Runoff Curve Number

Runoff volumes were estimated based on guidance provided in the NRCS Technical Release 55 (TR-55) (NRCS, 1986). The composite curve number (CN) was calculated for the watershed based on both the hydrologic soil group and land use types delineated within the watershed. Soil information was obtained from the USGS Soil Survey Geographic (SSURGO) database. Existing land use conditions were derived from the Multi-Resolution Land Characteristics (MRLC) Consortium's National Land Cover Database (NLCD, 2021). The NLCD is a publicly available database that utilizes satellite imagery to categorize land use characteristics of the contiguous U.S. at a 200-foot resolution. A composite CN of 72 was calculated for both watersheds utilized in this analysis under average antecedent runoff conditions (ARC II). A composite CN of 86 was calculated for both watersheds in this analysis under saturated runoff conditions (ARC III).

Maps that depict the hydrologic soil groups and land use classifications within the watershed, as well as detailed calculations, are included in Appendix C.

5.1.3 Lag Time

The lag time was calculated according to the National Engineering Handbook (NEH) Chapter 15 watershed lag method (U.S. Department of Agriculture, 2010). Parameters used as a part of this calculation include the longest flowpath, maximum potential retention (function of Curve Number), and the average watershed land slope (percent rise). Calculated lag times for each subbasin are depicted in Table 11.

Table 11. Lag Time Results

Drainage Basin	Short Identifier	Lag Time (min)
Shawnee City Lake No. 1	SWN1	94.5
Shawnee City Lake No. 2	SWN2	121

5.1.4 Precipitation

The Probable Maximum Precipitation (PMP) is the theoretically greatest depth of rainfall for a given duration that is physically possible over a given size storm area at a particular geographic location. Precipitation depths for the PMP evaluation were obtained from a regional PMP study, *Regional Probable Maximum Precipitation Study for Oklahoma* (Applied Weather Associates, 2019). OWRB hosts an online tool in which the centroid coordinates of the contributing area were input, and PMP depths for the requested area were provided. PMP depths for three storm types were included from the PMP tool output: general, local, and tropical. The general and tropical storm events include 6-hour, 12-hour, 24-hour, 48-hour, and 72-hour depths, and the local storm events include the 6-hour, 12-hour, and 24-hour depths. PMP depths are shown in Table 12.

Table 12. PMP Depths

Storm Type	PMP Duration				
	6-hour (in)	12-hour (in)	24-hour (in)	48-hour (in)	72-hour (in)
General Storm	20.4	23.98	24.87	32.88	33.4
Local Storm	23.2	27.77	32.54		
Tropical Storm	23.7	32.33	36.87	39.1	39.23

The tropical storm results in the highest 72-hour depth and was, therefore, selected for the PMF analysis. While the OWRB web tool provides temporal distributions for the storms, OWRB requirements specify the use of the Hydrometeorological Report 52 (HMR-52).

To estimate the temporal distribution, the integrated HMR-52 storm model was utilized within HEC-HMS. (U.S. Department of Commerce, 1982). HMR-52 calculated isohyets, or ellipses of equal rainfall depths, and oriented them over the drainage basins in such a way as to maximize the total amount of precipitation the basin would receive during the design storm. The HMR-52 User's Manual, developed by the U.S. Army Corps of Engineers, was used to optimize the PMP for storm orientation and area necessary to run the HMR-52 storm (U.S. Army Corps of Engineers, 1984).

Optimizing of the HMR-52 storm yields the 72-hour storm temporal distribution on the Shawnee drainage basins. This output hydrograph was then transformed into a 24-hour hydrograph using critical stacking of the 72-hour storm, and the general 24-hour storm depth was applied, creating the 24-hour PMP for use in the hydrologic model.

5.1.5 Design Storm Model Results

The PMF is defined as the greatest flood to be expected assuming complete coincidence of all factors that would produce the heaviest rainfall and maximum runoff. To evaluate the PMF inflows to Shawnee City Lake Nos. 1 and 2 reservoirs, and to obtain initial results of hydraulic adequacy prior to hydraulic modeling in HEC-RAS, the design storm event was modelled in HEC-HMS by applying the 24-hour storm described in Section 5.1.4 as inflow. Criteria for dam requirements in Oklahoma are contained in OWRB *Hydrologic and Hydraulic Guidelines for Dams in Oklahoma* (Oklahoma Water Resources Board, 2011). The standard for an intermediate dam of high hazard potential category is a minimum design of 50% of the PMF. A ratio of 0.5 was applied to the PMF evaluation in the HEC-HMS model. Table 13 contains the 50% PMF model results for Shawnee City Lake Nos. 1 and 2 Dams. Detailed hydrologic modeling results can be found in Appendix C.

Table 13: Shawnee City Lake Nos. 1 and 2 Dam HEC-HMS Model Results

Dam Name	Guideline Storm	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Elevation (ft)
Shawnee City Lake No. 1 Dam	1/2 PMF	46,382.1	18,534.1	1,082.3
Shawnee City Lake No. 2 Dam	1/2 PMF	21,847.9	11,939.3	1,080.5

These inflow hydrographs were used in hydraulic modeling of Shawnee City Lake No. 1 Dam in HEC-RAS, described in Section 5.2.

5.2 HYDRAULIC MODELING

To assess the hydraulic adequacy of the Shawnee City Lake spillway and emergency spillway, a 2-dimensional (2D) HEC-RAS hydraulic model was created. This model was used to analyze the existing dam and spillway system based on OWRB guidelines, which require a design storm event equal to the ½ PMF. The hydraulic model was also utilized to assess the maximum capacity of the dam and spillway system.

5.2.1 Hydraulic Model Development

The hydraulic analysis was performed using HEC-RAS Version 6.6 hydraulic modeling software (U.S. Army Corps of Engineers, 2024). 1D modeling utilizes representative cross sections to characterize the terrain through which flood discharges travel. Flow is conveyed from cross section to cross section, interpolating cross-section geometry based on distance between cross sections. 2D modeling utilizes a digital elevation model (DEM) to characterize the terrain based on the user input computational mesh. The individual cells created by the DEM and mesh are the basis for a finite element analysis that calculates time-dependent solutions to the momentum and continuity equations at each cell, allowing water to pass from cell to cell as opposed to cross section to cross section in 1D modeling. Due to the unique terrain and conveyance channels represented in the study area, 2D modeling was deemed the appropriate modeling method.

Shawnee City Lake No. 1 Dam is represented by a 2D area connection with inflows developed from the HEC-HMS model input within the reservoir areas. The HEC-RAS model used a 2D flow mesh to represent the reservoirs and area immediately downstream of the two dams and spillways.

5.2.2 Discharge Rating Curve

The spillway located near the left abutment of Shawnee City Lake No. 1 Dam is a 320-foot-long ogee crested weir, which flows onto a flat concrete apron. Downstream from the concrete spillway apron is a natural sandstone channel which flows to a concrete flip bucket drop structure. The ogee weir is located under a bridge with seven sets of approximately 3.3-foot-wide piers, creating eight bays and an effective weir length of 296.9 feet. The spillway crest is located at elevation 1,075.2 feet and the low-chord of the bridge is at elevation 1,082.55 feet, resulting in an opening height of max head of 7.35 feet. This spillway primarily serves Shawnee City Lake No. 1 Dam, but has some effect on the water levels within Shawnee City Lake No. 2 as well due to the channel connecting the two reservoirs.

The emergency spillway, located in the left abutment of Shawnee City Lake No. 2 Dam, is a natural channel that conveys discharges from the reservoir, through the left abutment of the dam, and into the downstream drainageway. The emergency spillway was modeled directly in HEC-RAS as a 2D area connection with a broad-crested weir.

The discharge rating curve for the existing ogee spillway was calculated by using methodology from the USBR Design of Small Dams for calculating flow over an ogee spillway (U.S. Bureau of Reclamation, 1987). This method factors in head, downstream apron elevation, upstream face slope, crest length, pier contraction coefficient, abutment contraction coefficient, and discharge coefficient as a function of head. Above an elevation of 1,082.55 feet, flow transitions to orifice flow as that is the elevation of the low-chord of the bridge above the spillway. The combined elevation-discharge relationship is shown in Table 14. Spillway discharge calculations can be seen in Appendix A.

Table 14. Service Spillway Discharge Rating Curve

Elevation (ft)	Discharge (cfs)
1,075.2[1]	0
1,076	658
1,077	2,319
1,078	4,511
1,079	7,127
1,080	10,129
1,081	13,483
1,082	17,178
1,083	20,979
1,084[2]	23,384

1. Denotes Normal Pool Elevation
2. Denotes Top of Embankment Elevation Based on Survey

5.2.3 Two-Dimensional Mesh Development

The HEC-RAS model consists of a two-dimensional mesh overlain on a digital elevation model representing the Shawnee City Lake reservoirs, embankments, overflow spillways, and downstream floodways, as well as the excavated channel connecting the reservoirs. A cell size of 100 feet was used throughout the model, as a balance of accuracy and computation time. A refinement region was created to cover the excavated channel connecting the two reservoirs. This refinement region utilized 25-foot cells in order to capture flow through this area in greater detail. Break lines were placed along major topographical changes and pertinent features, such as ridges, channels, and roadway embankments, to provide additional definition to the natural direction of flow through the system. The mesh area was computed using the 2020 NRCS 2-meter LiDAR topographic data with 2024 survey data (CEC) overlain in the emergency spillway and Shawnee City Lake No. 1 Dam spillway and embankment areas. The two-dimensional mesh is shown in Figure 8.

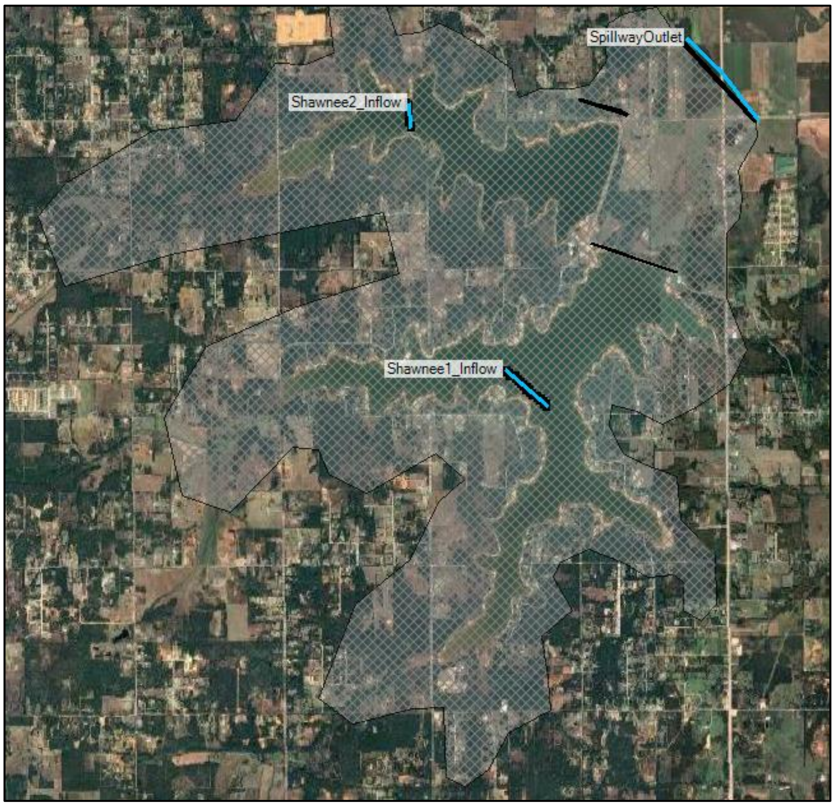


Figure 8. HEC-RAS Two-Dimensional Mesh

Manning's roughness coefficients (n-values) were determined from the National Land Cover Dataset for 2021 and were used in the calculation of energy losses of the water as it travels through the system.

Table 15 defines Manning's n-values associated with the National Land Cover Dataset. The selection of n-values was guided by recommendations given by the HEC-RAS 2D Modeling User's Manual (U.S. Army Corps of Engineers, 2024).

Table 15. Manning's n Values for Two-Dimensional Flow

LAND COVER	MANNING'S N-VALUE
BARREN LAND ROCK/SAND/CLAY	0.027
CULTIVATED CROPS	0.035
DECIDUOUS FOREST	0.15
DEVELOPED, LOW INTENSITY	0.09
DEVELOPED, MEDIUM INTENSITY	0.12
DEVELOPED, HIGH INTENSITY	0.16
DEVELOPED, OPEN SPACE	0.04
EMERGENT HERBACEOUS WETLANDS	0.068
EVERGREEN FOREST	0.12
GRASSLAND/HERBACEOUS	0.038
MIXED FOREST	0.14
OPEN WATER	0.02
PASTURE/HAY	0.038
SHRUB/SCRUB	0.115
WOODY WETLANDS	0.098

5.2.4 Inflow Hydrographs

Runoff hydrographs developed within HEC-HMS, as described in Section 5.1.5, were utilized as inflow hydrographs for Shawnee City Lake Nos. 1 and 2 Dams. These inflow hydrographs represent runoff from the contributing basins during the respective storm events. Initial condition points were utilized to set the initial reservoir elevation within the 2D mesh at the normal pool elevation. Internal boundary conditions were located within the approximate center of each reservoir, where the inflow hydrographs were applied to the 2D mesh. The inflow hydrograph locations are denoted by blue lines and labeled as "Shawnee1_Inflow" and "Shawnee2_Inflow" in Figure 8.

5.2.5 Downstream Boundary Conditions

Downstream boundary conditions were utilized for the model and allow flow to exit the 2D mesh. A single downstream boundary condition was located along the downstream edge of the mesh, approximately 7,000 feet downstream of Shawnee City Lake No. 1 Dam. Normal depth was selected for the downstream boundary condition to represent an energy grade approximately equal to the slope of the discharge channel. For Shawnee City Lake Nos. 1 and 2 Dams, a slope of 0.0025 ft/ft, as measured along the tributary channel, was utilized to represent the normal depth condition. The downstream boundary condition location can be seen in Figure 8.

5.3 ROUTING RESULTS

To evaluate compliance of Shawnee City Lake No. 1 Dam with OWRB guidelines, the ½ PMF was routed through the 2D HEC-RAS model. Additionally, various fractions of the PMF were also routed through the 2D model in order to evaluate the maximum capacity of the existing dam and spillway system, based on a dam crest elevation of 1,084 feet. The results of the hydraulic analysis are summarized in Table 16 below. Based on these results, the existing dam and spillway system does meet current OWRB design criteria for existing high hazard structures with approximately 1.4 feet of freeboard during the ½ PMF. The results of the hydraulic analysis also indicate that the existing dam and spillway system can safely pass up to approximately 65% of the PMF, which is less than the capacity estimated in the Phase 1 report of 81% PMF (Geocon, 1978).

Table 16. Hydraulic Modeling Results

Storm	Stage (ft)[1]	Peak Inflow Shawnee No. 1 (cfs)	Peak Inflow Shawnee No. 2 (cfs)	Peak Outflow Shawnee No. 1 (cfs)	Peak Outflow Shawnee No.2 (cfs)
50% PMF	1,082.6	46,400	20,700	17,400	9,830
Capacity (65% PMF)	1084	60,300	26,900	22,700	16,800

[1]Stage taken from center of the dam (approximate sta. 18+00)

Hydraulic mapping is presented in Appendix C

6.0 PEER REVIEW

FNI completed a technical peer review of the Shawnee City Lake No. 1 Dam Investigation, Geotechnical Engineering Report (Terracon, 2024).

The report includes documentation of geotechnical and geophysical testing at the dam, description of site conditions including assumed stratigraphy and approximated shear strength parameters, stability analyses, and recommendations. In summary, the report suggests the embankment consists of zones of low strength materials, potential voids, and does not meet the required stability factors of safety. The report identifies slope instability and movement near the west end (left) abutment, just right of the spillway, with stability analyses indicating both the upstream and downstream slopes present stability factors of safety of less than 1.

Although not noted in the report, email communication by Terracon described the drilling process as solid-stem flight augers to advance the borings with casing used to maintain the hole. Once groundwater was encountered, the advancement utilized a mud rotary with drilling mud to maintain the hole. As noted in Engineering Regulation 1110-1-1807 (U.S. Army Corps of Engineers, 2014), there have been reports of hydraulic fracturing of earth embankments and foundation materials when drilling with mud as a circulating medium. Use of hollow stem augers or steel casing is typically advanced with sampling through the auger or casing to protect the embankment and foundation materials. Several borings in the natural valley section (B-6 thru B-16) encountered low SPT blow counts near the elevation of groundwater while still drilling by advancing steel casing. These drilling techniques, while desirable for protection of the embankment dam and foundation, often loosen foundation soils at the base of the casing due to hydrostatic pressures, which heaves the soils in the zone of sampling and results in lower SPT blow counts.

During visual observations of the dam, FNI observed leaning power poles and misaligned guard rail but did not observe tension cracks on the slope or evidence of a scarp indicating slope instability. Movement of the guard rail and depressions in the wheel path of the crest roadway pavement were observed on the west end of the dam in the reach identified as unstable by the Terracon report. Historic photos from the 1978 Phase 1 Report identified similar pavement distress (rutting and alligator cracking), which suggest the depressions in the roadway are likely from pavement subgrade failures caused by traffic loading (Geocon, 1978). See Figure 9.

The crest of the dam indicates some pavement distress due to heavy vehicle traffic. Several areas of previous road maintenance were evident. Road construction and maintenance have altered the crest profile to preclude detailed informed comments regarding embankment settlement. Visual observations of the crest, and upstream slope did not reveal any cracks, slumps, or depressions indicative of embankment instability.

10. Pavement deterioration and subsequent repair.

Figure 9. Historical Pavement Repairs (Geocon, 1978)

The interpretation of data, associated stratigraphy and shear strength parameters, stability analyses, and overall conclusions and recommendations appear to strongly consider the geophysical test results. Secondly, the cone penetration tests (CPTs) results appear to be preferred over the conventional geotechnical borings, samples, and laboratory tests. Geophysical testing and CPTs are typically calibrated with traditional geotechnical investigations and laboratory testing. However, the Terracon report does not describe calibration of geophysical test data with the results of the actual borings. The following Terracon report content and data with resultant FNI observations are noted:

- Material descriptions and laboratory test data on the boring logs is limited. Additional description detail and laboratory testing would allow more accurate characterization of the subsurface stratigraphy. Foundation rock core samples were not obtained for strength testing, rock quality designation, or visual observation of the rock structure or bedding (e.g., blocky, laminated, slickensided, thickly bedded). Such information is typically used in the development of rock mass strength parameters.
- The geophysical data indicates very soft to soft materials for the upper 10 feet of most of the embankment crest and the top of the berm (referred to as the toe) while traditional soil sampling indicates stiff to very stiff sandy clay or sandy clay.

- The geophysical data indicates a consistent zone of soft materials at depths of 15 to 20 feet below the downstream berm through the center reach of the dam. While these soft or loose materials were not consistently encountered in borings along the downstream berm, The conventional borings did encounter limited thickness of loose silty sands or soft clay materials near these elevations in borings B-12, B-14, B-15, and B16.
- On the west end, conventional borings B-1 and B-2 appear to correlate well with adjacent CPT B-1.CPT and B-1.CPT; however, B-1 and B-2 were advanced to much shallower depths than B-1.CPT and B-2.CPT and B-1 and B-2 did not reach the depths of any softer soils identified in the adjacent CPTs. B-5.CPT indicated two zones of softer fined grained soils that were identified as medium dense silty sand in conventional boring B-5, one with a Standard Penetration Test (SPT) value of 25.
- CPT B-6 and CPT B-6D were drilled adjacent to each other. While both generally indicate similar soil materials and consistencies, CPT B-6 indicates organic or softer soils in zones from elevation 1,051 to 1,060 feet while CPT B-6D identifies a very thin organic lense near elevation 1,052 feet, only.
- CPT B-18 showed an approximate 6-foot-thick layer of very soft material, possibly organic, from about elevation 1,048 to 1,042 feet. The geophysical results along the berm (identified as along toe) did not indicate an anomaly at this depth and location. Additional traditional borings and CPTs in this area did not encounter this same very soft, organic layer. Localized soft zones of clay embankment fill and foundation materials were noted on multiple boring logs. The soft zones identified by the geophysical testing were isolated and consistently described as medium stiff to stiff on boring logs. The soft zones noted below the groundwater, and therefore drill casing depths, could reflect localized drilling disturbance of embankment and foundation materials. A specific location where this appears likely is boring B-8 at a depth of 73.5 feet. While the SPT resulted in a blow count value of 3, the SPT value at a depth of 68.5 feet resulted in a SPT value of 10. However, the moisture contents were 18.9% at 73.5 feet and 21.4% at 68.5 feet. For similar materials (as described in the boring log), a higher moisture content typically coincides with a lower SPT value rather than a lower moisture content combined with a lower SPT value.
- Several reported moisture contents appear anomalous, not correlating well with the reported material classification or blow count values. For example, the sample at a depth of 43.5 feet in boring B-15 and the sample at a depth of 33.5 feet in boring B-12 are described as lean clay (CL) but have reported moisture contents of 50.4 and 36.4%, respectively. Moisture contents of this magnitude are typically associated with fat clay (CH) or with organic materials. Similar to the previous comment, higher moisture contents for similar materials are typically associated with lower SPT values or softer materials, particularly when the moisture content is well above the plastic limit.

- The shear strength values for the lean clay embankment fill are representative of fully softened or even lower than typical residual shear strengths. Shear strength values near residual shear strength, such as these, are appropriate for soil masses that have experienced significant movement or potential for shrink/swell leading to fissured or slickensided surfaces within the soil mass. The lean clay with sand, as described in the boring logs, is typically not subject to this type of behavior and is not common on embankment dams in Oklahoma constructed of similar materials.
- Shear strength values for the weathered shale and sandstone materials appear to be conservative. The SPT values in the weathered sandstone and weathered shale indicate a very competent material with a friction angle much greater than the 30 degrees assumed in the report.
- Shear strength of the embankment on the west end is based on one laboratory shear strength test from boring B-1. Sample 1 of that shear test indicates an initial water content and saturation of 19.9% and 99%, respectively. During the test, the water content is reported as 15.2% with a saturation of 75.7%. It appears the sample was allowed to dry, potentially leading to an incorrect test result. Typically, a dryer sample could provide a greater strength. However, a higher strength for the sample tested at the lower cell pressure could result in a greater cohesion value but a lower estimate for phi angle value.
- Two other triaxial shear tests of similar embankment fill material type resulted in much greater shear strength, especially phi angles.
- Some samples, such as samples 2 and 3 from B-1 (6-8 feet), were tested to a low strain while the stress was still increasing at the final strain. If the samples had been tested to a higher strain, the failure stress would likely have been higher. As a result, the estimated shear strength and slope stability factors of safety estimates would likely be higher.
- Although the three triaxial shear strength tests only took the portion of the test with greatest cell pressure to any significant strain, the results did not indicate much reduction in strength with strain indicating the soil strengths do not reduce to a significantly lower fully softened or residual strength with movement or shrink/swell as described above.
- With a factor of safety (FS) of 0.87 shown in Table 10 and illustrated in Figure 7 using conservative shear strengths, slope instability on the downstream slope of the west end should be visually apparent under existing load conditions. In other words, a failure should have already occurred since mathematically failure occurs when the factor of safety reaches unity.
- Similarly, the FS shown in Table 10 and Figure 8 for the west end upstream slope should present obvious slope instability under current loading conditions. Upstream slope instability was not noted in the report nor visually observed by FNI.

- Shear wave results from the geophysical testing, CPTs indicating soft zones or voids, and isolated low blow counts from Standard Penetration Tests are suggests as reasons to infer potential of voids within the embankment. Actual borings (in the same vicinity of a CPT boring suggesting voids do not indicate voids. This was noted in the Terracon report for boring B-6.
- Groundwater levels in borings B-12 through B-15 along the downstream toe of the dam indicated a phreatic surface as high as approximate elevation 1,035 feet. The phreatic profile in the stability cross sections conservatively show a straight-line phreatic surface from the upstream water line to the embankment toe. However, the accuracy of the groundwater readings within hours or a few days of completing the borings is questionable given the above-mentioned drilling methods.
- As a standard practice for embankment dam analysis, a seepage analysis was not conducted in combination with the slope stability analyses.
- Slope stability analyses due to seismic loading were performed using site-class-adjusted PGA values from USGS. The results suggest an FS of less than 1.0. If the estimated seismic stability FS is less than 1.0, permanent displacements and deformations using documented semi-empirical procedures is recommended. Terracon reported slope instability, voids, and the need for remediation and repair of the embankment. The recommended repair options of compaction grouting or chemical injection are not typically used on embankment dams or their foundations. One of the reasons these techniques are not utilized on embankment dams is the potential to cause hydraulic fracturing. Hydraulic fracturing of the embankment or foundation can negatively impact the dam performance and cause internal damage that could result in undesirable seepage that could lead to concentrated leak erosion or backward erosion piping. Any repair technique that could result in such is typically avoided or critically evaluated and controlled in dam engineering practice.
- Several Seismic Cone Penetration Tests with pore pressure measurements (SCPTu) were conducted on the central to eastern embankment. Conventional borings were drilled adjacent to some of these SCPTu locations as shown in the Exploration Plan of the Terracon Report. Table 17 summarizes locations of organic/soft soils noted at SCPTu locations where conventional boring sample descriptions were collected and described.

Table 17. Soft Areas identified by CPT with adjacent Sample Descriptions

Boring	Elevation, ft	Sample Description/Blow Count
B-6	1,059	Stiff, N=13
B-6	1,054	Very Stiff, N=16
B-7	1,040	Very Stiff, N=21
B-8	1,042	Very Stiff
B-8	1,019	Stiff, N=12
B-10	1,054.5	Stiff, N=8

The slope angles of the embankment are typical of other earth dams in the state with the exception of the upper steepened portion. FNI's observations indicate that neither the upstream nor downstream slope appears to be currently experiencing movement of concern. FNI is not aware of historical evidence of slope instability except for localized shallow sloughing of the top portion of the upstream slope. Geophysical tests results and CPT tests do not appear to consistently correlate with conventional drilling and sampling results. In general, the strength estimates and stability analyses do not appear to reflect the observed performance and therefore are questionable.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 CURRENT CONDITIONS

The data, interpretations, and analyses, used to determine the current condition of the dam and resulting in the OWRB consent order, do not appear to reflect actual conditions and performance of the dam, as illustrated in Figure 10. The dam, in its current configuration, meets current hydraulic OWRB Dam Safety Criteria. The dam is capable of passing 65% of the current Probable Maximum Flood (PMF) without overtopping. Slope stability factors of safety meet OWRB criteria with no visible signs of instability, currently or from previous inspections.

Figure 10. Downstream Slope in vicinity of the repaired roadway section (July 2024)

The concerns raised in recent inspections (Terracon, 2022) regarding the pavement distress were noted in the 1978 Phase 1 Report (Geocon, 1978). That investigation indicated distress from heavy vehicle traffic and noted several past repairs in this same area. FNI noted similar conditions during our investigation as shown in Figure 11 .

Figure 11. Road Repairs noted during the July 2024 inspection by FNI

Seepage is present along both downstream abutments. Such seepage has been noted as far back as 1948 in an investigation conducted by C. H. Guernsey Consultants (C.H. Guernsey Consultants, 1948). Similar seepage was noted in the 1978 Phase 1 Report (Geocon, 1978). Neither of these reports noted this seepage as a dam safety concern.

Geotechnical instrumentation, such as piezometers or inclinometers, is not recommended at this time. While the data obtained from each can be useful and interesting, the cost and logistics of management of such instruments currently outweigh their value.

7.2 RECOMMENDATIONS

While the conditions described above do not constitute urgent or imminent dam safety concerns, FNI recommends the following additional studies and improvements to the dam.

7.2.1 Additional Studies

As described in Section 4, there are some foundation soils under the embankment that may be susceptible to liquefaction or strength loss due to seismic shaking. Susceptibility is based on material properties but the effect of those soils on stability of the dam following seismic loading is based on the degree of seismic shaking. The additional analyses recommended in Section 4 regarding these soils require either a semi-quantitative risk assessment (SQRA) or at least a determination of relative consequences (see Figure 5 in Section 4.2.2). Both of those efforts require at least a semi-quantitative assessment of the downstream consequences which cannot be performed satisfactorily with the current 2013 dam breach analysis and inundation mapping. An updated breach analysis using two-dimensional flow modeling techniques (HEC-RAS 2D) is required to determine depths and velocities in the inundation zone. In addition to providing specific information on impacts to structures downstream, this updated model and associated mapping will be beneficial for EAP planning and execution by emergency management personnel.

To assess the post seismic performance of the dam, a liquefaction and loss of strength analyses/study may be performed. If existing soils are determined to liquefy or lose strength through seismic loading, a post-seismic slope stability and deformation analysis is recommended. Should a SQRA suggest the post-seismic stability and deformation present risk above tolerable guidelines, a site-specific seismic study may be recommended.

7.2.2 Downstream Slope Repair

The lower slope of the downstream embankment, below the berm, is severely overgrown with large trees and woody vegetation. Tree roots can cause seepage paths, and trees can uproot during storm events, causing removal of portions of the dam, leading to a dam safety concern. Per OWRB requirements, trees and other woody vegetation in this area should be cleared and grubbed with fill material appropriately compacted into the voids left by the grubbing activity. The entire area should then be seeded and established with a sod-forming grass variety.

7.2.3 Upstream Slope Repair

The upstream slope of the embankment is covered with rock and concrete riprap; however, some areas are in need of repair, as shown in Figure 12. Several areas along the slope show signs of previous repairs with a wide variety of armoring materials being used including broken concrete, masonry, and various sizes of rock and gravel. There was no visual evidence of proper granular bedding material under the riprap. Properly sized and graded rock riprap relies on the jagged nature of the stones to provide an interlock network of protection. Mis-matched size and materials are less effective at preventing wave erosion. Left untreated, erosion along the front slope can lead to slope instability, sliding or sloughing which could impact the integrity of the dam and lead to a potential catastrophic failure.

Removal and replacement of the majority of the length of the slope (approximately 2,300 linear feet) from the dam crest down to approximate elevation 1,070 feet is recommended. Spot treatment of the more severely deteriorated areas is an option should funding be limited. Establishment of a gentler upper slope is also recommended to improve stability of the upper slope riprap and prevent future oversteepening due to sloughing. FNI recommends the most cost-effective solution is to push the existing materials to just below normal reservoir levels where the upstream slope breaks to a flatter 3H:1V slope. This technique would create a stabilizing buttress for the new bedding and riprap providing a wave berm to dissipate wave energy prior to impacting the dam upstream slope. Properly sized bedding material and rock riprap could then be placed at a 2H:1V slope creating low-maintenance, long-term slope armoring. Construction would be conducted by an excavator working from the dam crest requiring removal and replacement of the guardrail. It is likely that full pavement replacement would also be required due to damage from construction equipment and heavily loaded riprap haul trucks.

Figure 12. Armoring Repair – Mis-Matched Riprap

7.2.4 Seepage Monitoring

While not identified as a current dam safety concern, it is recommended that additional investigation of the seepage in the downstream groins be completed following the clearing and grubbing. Depending on results of the further investigation, a foundation drain at the abutment(s) or downstream toe of the dam may be recommended or monitoring the seepage from each abutment. Monitoring may be accomplished by a measurement weir installed along each downstream abutment groin.

The fishponds behind the dam are no longer in operation; however, historical photos indicate this area maintains water, periodically. Wet areas immediately below a dam can mask foundation seepage conditions, and can harbor unwanted rodents/animals, which in turn, can damage the dam. As part of the operation and maintenance of the dam, maintaining the downstream area in a dry condition is recommended.

7.2.5 Spillway Repair

The area immediately upstream of the flip bucket drop structure shows areas of scour. This appears to be an ongoing issue as some rock riprap has been placed in select areas, as shown in Figure 13. Scour in this location can lead to undermining of the structure and potential failure. FNI recommends installing properly sized and graded rock riprap along the entire upstream crest of the flip bucket drop structure for scour protection.

Figure 13. Riprap Scour Protection above the Flip Bucket Drop Structure

7.3 PERMITTING REQUIREMENTS

7.3.1 U.S. Army Corps of Engineers (USACE)

Since a portion of the work proposed may be performed in areas that could be considered waters of the United States (WOTUS) by the USACE, a brief review was made regarding potential permitting requirements under Section 404 of the Clean Water Act (CWA). Shawnee City Lake Nos. 1 and 2 Dams are located within the regulatory boundary of the Tulsa District USACE.

Acting under Section 404 of the CWA, the USACE regulates the discharge of dredged or fill material into WOTUS. WOTUS include navigable waters and may include other parts of the surface water tributary system down to the smallest of streams (e.g., tributaries that contain water only after a rain event), lakes, ponds, or other water bodies on those streams, and adjacent wetlands (e.g., sloughs, swamps, and some seasonally flooded areas) if they meet certain criteria. Although a jurisdictional determination has not been completed, it was assumed that Shawnee City Lakes, the spillway and channel downstream of the spillway are WOTUS. As such, any activities that would require the placement of fill material in WOTUS, including wetlands, must obtain authorization from the USACE prior to construction. It is anticipated that the riprap replacement and likely the clearing and grubbing activities will fall under Nationwide Permit (NWP) 3 – Maintenance.

NWP 3 authorizes the "repair, rehabilitation, or replacement of any previously authorized, currently serviceable structure… provided that the structure or fill is not to be put to uses differing from those uses specified or contemplated for it in the original permit or the most recently authorized modification. Minor deviations in the structure's configuration or filled area, including those due to changes in materials, construction techniques, requirements of other regulatory agencies, or current construction codes or safety standards that are necessary to make the repair, rehabilitation, or replacement are authorized. This NWP also authorizes the removal of previously authorized structures or fills. Any stream channel modification is limited to the minimum necessary for the repair, rehabilitation, or replacement of the structure or fill; such modifications, including the removal of material from the stream channel, must be immediately adjacent to the project". The preparation and submittal of a Pre-Construction Notification (PCN) to the USACE is anticipated for the upstream slope riprap and clearing and grubbing activities. The clearing and grubbing will likely require a working platform (haul road) at the toe of the slope impacting existing wetlands.

The regulatory time frame for the USACE to review and authorize projects that meet the terms and conditions of an NWP is 30 to 45 days if the PCN is administratively complete. However, the process could take longer if the USACE presents questions or comments on the application.

7.3.2 Oklahoma Water Resources Board (OWRB)

A dam alteration permit will be required to be issued by the OWRB. Completed plans, specifications, and design report are required to be submitted with brief permit forms. OWRB often reviews and responds to permit submittal packages within 60 days.

8.0 OPINION OF PROBABLE CONSTRUCTION COSTS

8.1 COST ESTIMATING METHODOLOGY

8.1.1 Level of Project Definition

AACE International (formerly the Association for the Advancement of Cost Engineering) defines five levels of cost estimates for a project in their Recommended Practice No. 17R-97. AACE classifications are a widely accepted guideline within the cost estimating community for defining level of project maturity and expected range of accuracy for associated project cost estimates. AACE classifications range from Level 5 to Level 1 for the lowest to highest levels of project definition. The purpose of the AACE classifications is to improve communication among project stakeholders involved in preparing, evaluating, and using cost estimates. The guidelines are intended to help avoid inappropriate decisions caused by misunderstanding cost estimates and what they are expected to represent.

This report documents the conceptual-level alternatives analysis of the Shawnee City Lake No. 1 Dam improvements and the corresponding Class 4 cost estimate. Per AACE, a Class 4 estimate corresponds to a project maturity level of 5 to 10% completeness. A Class 4 Opinion of Probable Construction Costs (OPCC) is suitable for a conceptual or feasibility study of a project. Per AACE, the true project construction cost for the proposed conceptual level would be expected to fall within -15 to +50% of the Class 4 OPCC. Table 18 summarizes the AACE cost estimate classifications with the corresponding expected accuracy ranges.

Table 18. AACE Generic Cost Estimate Classification Matrix

Estimate Class	Level of Project Definition (as a % of completion)	End Use	Expected Accuracy Range	Preparation Effort (Degree of effort relative index of 1)
Class 5	0% to 2%	Screening or feasibility	L: -20% to -50% H: +30% to +50%	1
Class 4	1% to 15%	Concept study or feasibility	L: -15% to -30% H: +20% to +50%	2 to 4
Class 3	10% to 40%	Budget authorization or control	L: -10% to -20% H: +10% to +30%	3 to 10
Class 2	30% to 75%	Control or bid/tender	L: -5% to -15% H: +5% to +20%	5 to 20
Class 1	65% to 100%	Check estimate or bid/tender	L: -3% to -10% H: +3% to +15%	10 to 100

8.1.2 Unit Prices

Preparation of an OPCC involves the use of data derived from several sources, with an overall goal of obtaining a reasonable and defensible expectation of costs for a specific level of project maturity. Sources of data used in preparation of the OPCC include, but are not limited to, the following:

- Construction data aggregation services
- Publicly available construction data
- Similar past projects performed by engineer
- Professional experience and engineering judgment

Unit prices shown in the OPCC are assumed to include direct project costs, contractor overhead, and profit for each line item. In other words, unit prices reflect the total unit cost of that line item to the owner. Except where explicitly noted, indirect project costs (i.e., bonds, safety program, quality control, surveying, insurance, warranties, taxes, etc.) are assumed to be subsidiary to the major construction work items listed in the OPCC.

8.1.3 Risks and Contingency

An OPCC is a prediction based on available records at a present time to represent a forecast of conditions at some point in the future. As such, an OPCC is necessarily an approximation, and thus has an inherent level of uncertainty. At a conceptual level, the OPCC is subject to risk which is reflected in the expected accuracy ranges provided in Table 18. An overall contingency of 30% has been included in the OPCC. This value was selected based on the intended maturity of the project at this stage, experience on similar past projects, and engineering judgment.

The contingency is the cost assigned to the unknowns in the definition of the project. It is intended to account for construction costs that have not yet been identified due to the project maturity and should be expected to be fully used for construction of the feasibility-level design concepts. The contingency is not a measure of estimate accuracy, and the range of accuracy provided in Table 18 is not affected by inclusion of contingency in the OPCC. Many project owners include their own contingency to a budgetary allocation to establish the amount of funding necessary to construct the project. This additional contingency is intended to provide a ceiling so that costs are more likely to fall below the budget allocation and additional funding requests are avoided.

8.1.4 Price Base

Unless otherwise stated, all dollar values presented in this report can be assumed to be nominal values with a price base of December 2024. If values are to be used in a year other than 2024, they should be adjusted for factors which affect nominal prices over time as appropriate.

8.1.5 Excluded Costs

The OPCC presented in this report does not include non-constructions costs, including the following:

- Project financing costs
- Engineering
- Environmental permitting
- Easement or property acquisition
- Geotechnical investigations
- Other on-site exploration costs
- Easement and right-of-way acquisition
- Legal costs
- Public outreach
- Owner administration and project management costs
- Construction management services
- Ongoing costs, including operation and maintenance.

8.2 OPINION OF PROBABLE CONSTRUCTION COSTS

Table 19 summarizes the conceptual-level OPCC developed for the recommended dam improvements.

Table 19. Estimated Construction Costs – Recommended Dam Improvements

Work Item	Base OPCC	Estimate Range		
		-15	to	+50
Mobilization/Demobilization	$185,800	$157,930	-	$278,700
Erosion and Sediment Controls	$53,000	$45,050	-	$79,500
Care of Water During Construction	$235,000	$199,750	-	$352,500
Upstream Riprap Replacement	$2,363,000	$2,008,550	-	$3,544,500
Downstream Slope/Toe Clear & Grub	$216,000	$183,600	-	$324,000
Spillway Repairs	$74,800	$63,580	-	$112,200
TOTAL	**$3,127,600**	**$2,658,460**	-	**$4,691,400**

It should be noted that the lump sum estimates for mobilization, erosion control, etc. were developed based on variable percentages, between 2 and 10%, of the total cost sum of measurable quantities. As noted in the previous sections, an overall contingency of 30% has been included in the OPCC for financial variability and would be further refined during design.

9.0 REFERENCES

American Society of Civil Engineers. (2016). *Minimum Design Loads for Buildings and Other Structures (ASCE 7-16).*

Applied Weather Associates. (2019). *Regional Probable Maximum Precipitation Study For Oklahoma, Arkansas, Louisiana, and Mississippi.*

Boulanger, R. W., & Idriss, I. (2014). *CPT and SPT Based Liquefaction Triggering Procedures.* Dept. of Civil & Environmental Engineering, UC Davis.

Bray, J., & Sancio, R. (2006). Assessment of the Liquefaction Susceptibility of Fine-Grained Soils. *Journal of Geotechnical and Geoenvironmental Engineering, 132*(9).

C.H. Guernsey Consultants. (1948). *South Fork Deer Creek Examination.*

Castellanos, B., Brandon, T., & VandenBerge, D. (2016). Use of Fully Softened Shear Strength in Slope Stability Analysis. *Landslides*, Vol. 13, No. 4.

Duncan, J., O'Neil, B., Brandon, T., & Vanden Berge, D. (2011). *Evaluation of Potential for Erosion in Levees and Levee Foundations.* CGPR Report #64.

Duncan, J., Wright, S., & Brandon, T. (2014). *Soil Strength and Slope Stability.*

Duncan, J., Wright, S., & Wong, K. (1990). Slope Stability during Rapid Drawdown. *H. Bolton Seed Memorial Symposium*, *2.*

Duncan, J., Wright, S., & Wong, K. (1990). Slope Stability during Rapid Drawdown. *Proceedings of H. Bolton Seed Memorial Symposium*, (p. Vol 2).

Fredlund, D., Xing, A., & Huang, S. (1994). Predicting the Permeability Function for Unsaturated Soils Using the Soil-Water Characteristic Curve. *Canadian Geotechnical Journal*, 31(4), 533-546.

Geocon. (1978). *Phase I Inspection Report .*

Harder, L. F., Hollister, N., & Hammond, W. (1990). *Ground Motion Amplification Through Earth Dams.*

Makdisi, F. I., & Seed, H. B. (1978). *A Simplified Procedure for Estimating Earthquake-Inuced Deformations in Dams and Embankments.*

NCHRP . (2008). *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments Report 611 (NCHRP 12-70).*

NRCS. (1986). *Urban Hydrology for Small Watersheds TR-55.*

NRCS. (n.d.). *National Engineering Handbook, NEH 628 Chapter 52.*

Oklahoma Geological Survey. (2015). *The University of Oklahoma.* Retrieved December 16, 2024, from Oklahoma Fault Database: https://www.ou.edu/ogs/data/fault

Oklahoma Geological Survey. (2021). Retrieved December 16, 2024, from The University of Oklahoma: https://ou.edu/content/dam/ogs/documents/ogqs/OGQ-100_Oklahoma_City_South_100K.pdf

Oklahoma Water Resource Board (OWRB). (n.d.). *Title 785, Ch. 25.*

Oklahoma Water Resources Board. (2011). *Hydrologic and Hydraulic Guidelines for Dams in Oklahoma.*

OWRB. (2023). *Shawnee City Lake No.1 Dam Consent Order.*

Terracon. (2022). *Earth Dam Inspection, Shawnee City Dam #1.* Oklhoma City, OK.

Terracon. (2024). *Geotechnical Engineering Report Shawnee Lake Dam No. 1 Investigation (Dam ID No. OK-11039).* Oklahoma City, OK.

U.S. Army Corps of Engineers. (1984). *HMR52 Probable Maximum Storm User's Manual.*

U.S. Army Corps of Engineers. (2000). *Technical Bases for Regulatory Guide for Soil Liquefaction.*

U.S. Army Corps of Engineers. (2014). *ER 1110-1-1807, Drilling in Earth Embankment Dams and Levees.*

U.S. Army Corps of Engineers. (2023). HEC-HMS v4.11.

U.S. Army Corps of Engineers. (2024). *HEC-RAS River Analysis System 2D User's Manual V6.6.*

U.S. Army Corps of Engineers. (2024). HEC-RAS v6.6.

U.S. Bureau of Reclamation. (1987). *Design of Small Dams.*

U.S. Bureau of Reclamation. (2014). *Design Standard No. 13 Embankment Dams.*

U.S. Department of Agriculture. (2010). *Part 630 Hydrology National Engineering Handbook Chapter 15.*

U.S. Department of Agriculture, N. R. (March 2019). *Technical Release 210-60 Earth Dams and Reservoirs.*

U.S. Department of Commerce. (1982). *Hydrometeorological Report no. 52.*

U.S. Geological Survey. (2022, September 27). *Interactive U.S. Fault Map.* Retrieved December 16, 2024, from https://www.usgs.gov/tools/interactive-us-fault-map

USACE. (2024). *National Inventory of Dams - Shawnee City Lake No. 1*. Retrieved from National Inventory of Dams: https://nid.sec.usace.army.mil/#/dams/system/OK11039/summary

USGS. (2014). *Unified Hazard Tool*. Retrieved from https://earthquake.usgs.gov/hazards/interactive/

Appendix A. Photo Log

FIGURE B-1

Preliminary Engineering Report Photo Location Map

Shawnee Twin Lakes #1 Dam

FREESE AND NICHOLS
5100 E. Skelly Drive
Suite 602
Tulsa, OK 74135

FNI PROJECT:	SWN24427
FILE:	H:\WR_DESIGN\Figures\Shawnee\Shawnee.mxd
COORDINATE SYSTEM:	NAD 1983 2011 STATEPLANE OKLAHOMA SOUTH FIPS 3502 FT US
DATE:	DECEMBER 2024
PREPARED BY:	LJD


Photo No. 1: Looking west across the crest of dam


Photo No. 2: View of minor damage to guard rail along the crest of dam.


Photo No. 3: View of asphalt patch near spillway.


Photo No. 4: Looking east across the upstream slope from the left abutment.


Photo No. 5: Photo of a large piece of concrete debris on the upstream slope. Rod is shown for scale.

Photo No. 6: View of riprap on the upstream slope. Notice the mismatched sizes of rock and large sandstone slabs. The scale rod shown is 4ft long.

Photo No. 7: View of a gap under a large sandstone boulder near the crest of the slope. The gap has been created from washout of finer material under the large riprap.


Photo No. 8: View of rodent hole on the upstream slope.


Photo No. 9: View of several large pieces of concrete among the riprap on the upstream slope.


Photo No. 10: Finer grained aggregate and soil along the upstream slope near the crest.


Photo No. 11: Looking west along the upstream slope.

Photo No. 12: View of damage to guard rail post on the upstream side of the road.

Photo No. 13: View of the intake tower from the upstream slope. During inspection, the water level was at approximate elevation 1072 ft, about 3ft lower than normal pool.

Photo No. 14: Looking east along the downstream slope from near the spillway.

Photo No. 15: Looking east across the crest and downstream slope. Notice a slight angle to the powerlines.

Photo No. 16: View of typical animal burrow along the downstream slope.

Photo No. 17:View of the downstream toe just east of the spillway. Notice the overgrowth along the toe.

Photo No. 18: Looking east along the downstream berm. The area below the berm was not easily visible for inspection.

Photo No. 19: Typical view of standing water from seepage on the downstream toe near the abutments. The water was mostly clear and not visibly flowing.


Photo No. 20: Looking east across the upstream side of the spillway.


Photo No. 21:View of the spillway area under the bridge.

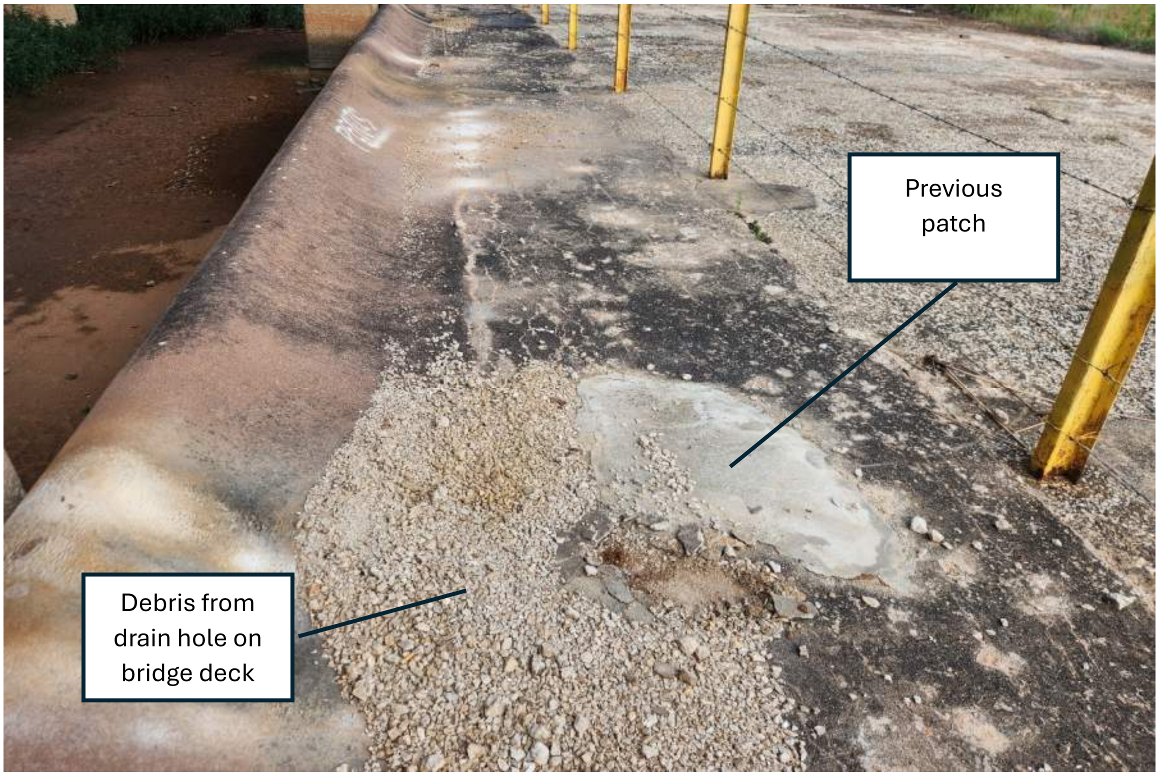


Photo No. 22: Ogee crest and apron. Typical view of spillway concrete with minor cracking, spalling, and delamination.

Photo No. 23: Looking upstream at the spillway crest.

Photo No. 24:View of right spillway training wall looking downstream.

Photo No. 25: View of the flip bucket spilling basin. Notice the localized scour just upstream of the structure that has been filled with riprap.

Photo No. 26: Additional scour upstream of the flip bucket walls.

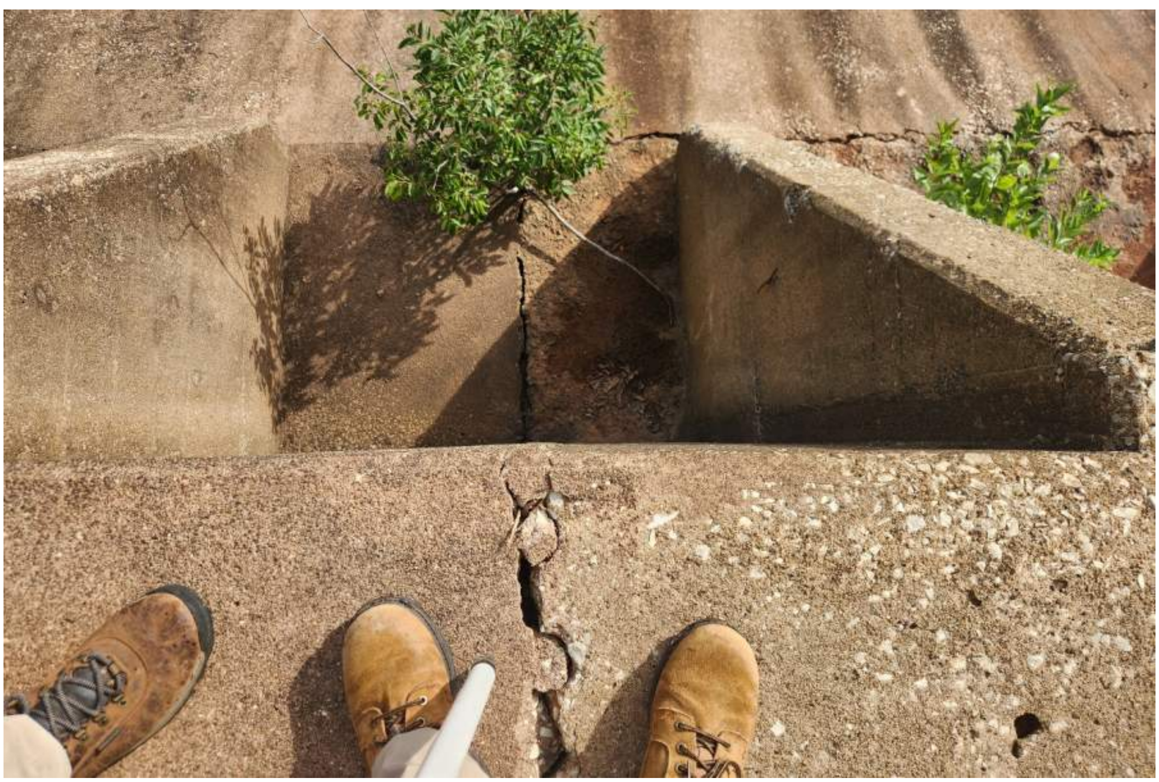


Photo No. 27:View of a concrete joint in the flip bucket and a small shrub growing in the joint.


Photo No. 28: Some cracking and spalling along a concrete joint in one of the flip buckets.


Photo No. 29: View of honeycombed concrete at base of back wall of flip bucket.

Photo No. 30: View of a concrete joint and some woody debris in the stilling basin.

Photo No. 31: View of the downstream side of the stilling basin back wall and buttresses.

Photo No. 32: Downstream side of stilling basin structure. Some displaced riprap from previous flows noted downstream of end sill. Concrete surface weathering on energy dissipators.

Photo No. 33: View of downstream side of right abutment of stilling basin structure.

Appendix B. Geotechnical Material Parameters and Modeling

Material Parameters

1.0 PURPOSE AND BACKGROUND

This worksheet develops shear strength and seepage material parameters for soil and rock materials for use during the slope stability analysis of the existing slope as part of the Twin Lake Dam #1 Evaluation project in Shawnee, Oklahoma. Field and laboratory data collected as part of the geotechnical investigation performed by Terracon from 2023-2024 was used in the parameter analysis and development. The borings primarily encountered layers of lean to sandy lean clay (CL), clayey sand (SC), and silty sand (SM). These materials were classified and evaluated separately as fill or residual material based on boring depth and as-built drawing stratigraphy. The bedrock encountered in the borings is associated with the Wellington Formation. This formation consists of red-brown shale and orange-brown fine-grained sandstone, containing much maroon mudstone conglomerate and chert conglomerate to the south.

2.0 REFERENCES

1) Shawnee Lake Dam No. 1 Investigation (Dam ID No. OK-11039) Geotechnical Engineering Report. (Terracon, 2024)
2) As-Built Drawings: Deer Creek Water Project (W.R. Holway Consulting Engineer, 1934)

3.0 UNIT WEIGHT AND CLASSIFICATION DATA

The range of dry and moist unit weights for soil materials are summarized in Table 1. Unit weight laboratory tests were only performed on CL fill materials; therefore, ranges for other materials are not provided and a typical moist unit weight was selected for the analysis.

Table 1. Dry/Moist Unit Weight Summary

Material	Material Type	Range Dry Unit Weight (pcf)	Range Moist Unit Weight (pcf)	Selected Moist Unit Weight (pcf)
Fill	CH	-	-	125
	CL	94 – 111	119 – 130	125
	SC	-	-	120
	SM	-	-	120
	SC-SM	-	-	120
Residual	CH	-	-	125
	CL	-	-	125
	SC	-	-	130
	SM	-	-	130
	SC-SM	-	-	125

The water content, percent passing the No. 200 sieve, liquid limit (LL) and plasticity index (PI) results for various materials are plotted in Figure 1 and Figure 2 for fill and residual material, respectively. Materials classified as ML, SP, and CL-ML on the borings are excluded from these figures since they were only identified in a few select samples and do not represent the overall material at the site. Classification data for fill and residual material is also summarized in Table 2 and Table 3, respectively, with the average and standard deviation listed for each soil group.

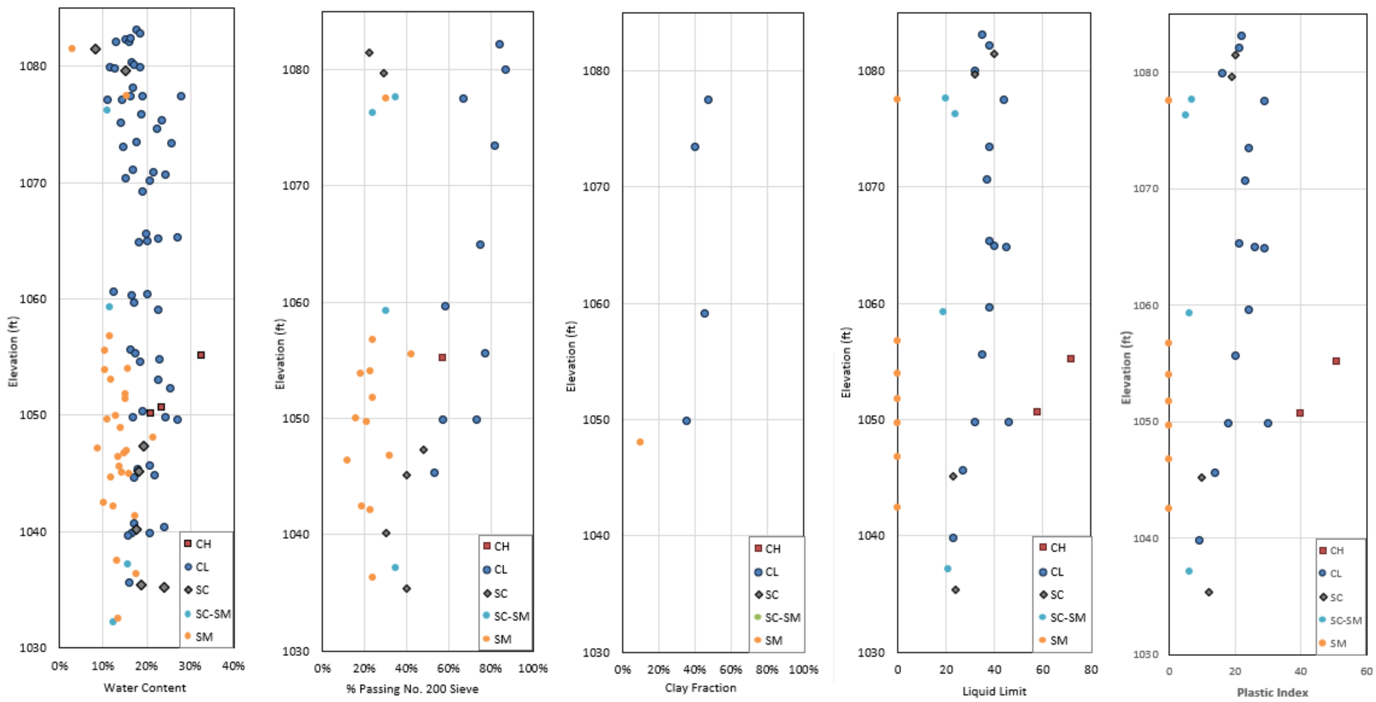


Figure 1. Classification Data vs Elevation (Fill)

Table 2. Classification Data Summary (Fill)

Material Type	Liquid Limit			Plasticity Index			%Passing No. 200 Sieve		
	Range	Ave	Std. Dev.	Range	Ave	Std. Dev.	Range	Ave	Std. Dev.
CH	58-72	65	9.90	40-51	46	7.78	57	57	-
CL	23-46	37	6.31	9-30	22	5.84	53-87	71	0.12
SC	23-40	30	7.93	10-20	15	4.99	22-48	35	0.09
SC-SM	19-24	21	2.16	5-7	6	0.82	24-35	31	0.05
SM	NP	NP	NP	NP	NP	NP	12-42	23	0.08

[1] NP = non-plastic materials

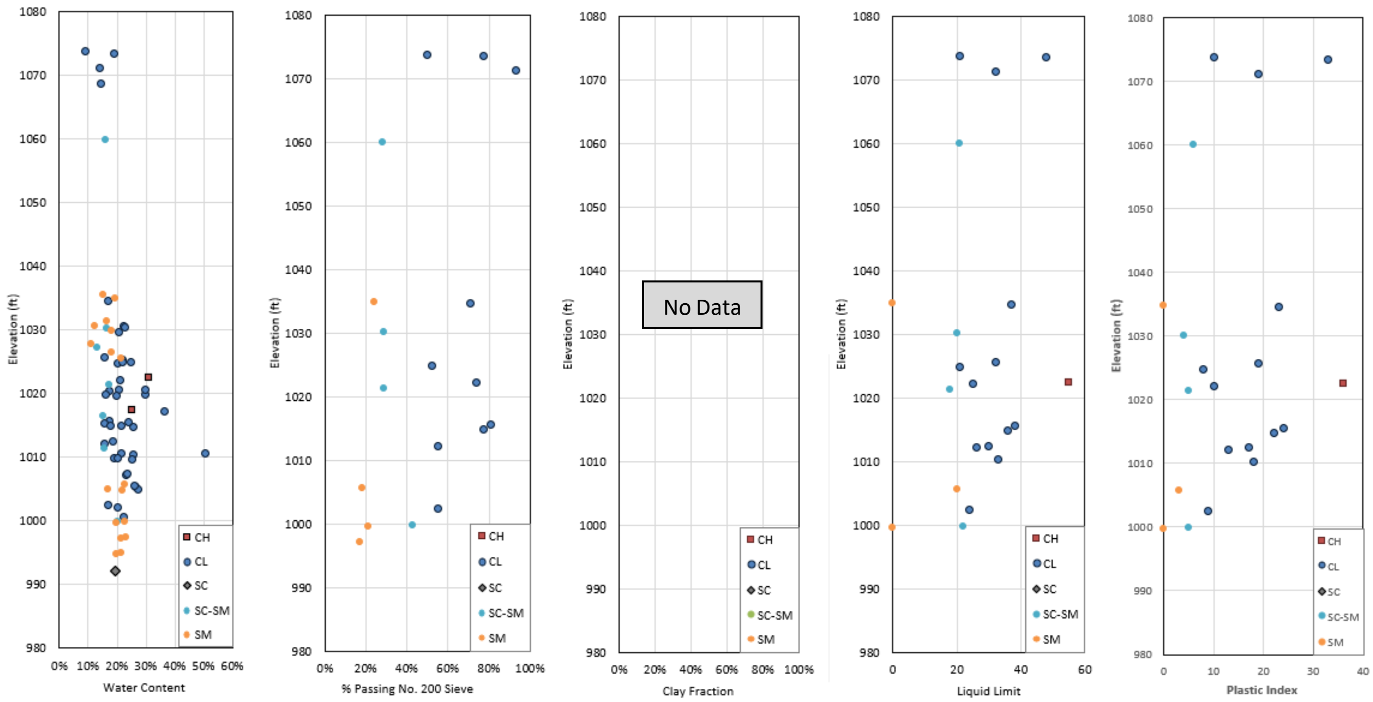


Figure 2. Classification Data vs Elevation (Residual)

Table 3. Classification Data Summary (Residual)

Material Type	Liquid Limit			Plasticity Index			%Passing No. 200 Sieve		
	Range	Ave	Std. Dev.	Range	Ave	Std. Dev.	Range	Ave	Std. Dev.
CH	55	55	-	36	36	-	-	-	-
CL	21-48	31	7.72	8-33	17	7.26	50-93	69	0.15
SC	NP	NP	-	NP	NP	-	-	-	-
SC-SM	18-22	20	1.71	4-6	5	0.82	28-43	32	0.07
SM	NP-20	7	11.55	NP-3	1	1.73	17-24	20	0.03

[1] NP = non-plastic materials

4.0 EFFECTIVE SHEAR STRENGTH

4.1 Secant Friction Angle by Correlation (Cohesive Soils)

Published correlations were used to compare predicted secant friction angles for cohesive materials in a normally consolidated or fully softened condition and were used in the selection of the friction angles. Values were computed using Stark and Hussain (2013), as provided in Figure 3. The chart includes highlighted areas that correspond to either the existing fill or the foundation materials over the range of LL values indicated by the laboratory data. The clay fraction for CL fill ranged from 35% to 47%. Therefore, the CL soils fall primarily within the range of Category 2. For the range of LL values from the laboratory tests, this correlation indicates secant friction angles between 25 and 35 degrees for the CL soils.

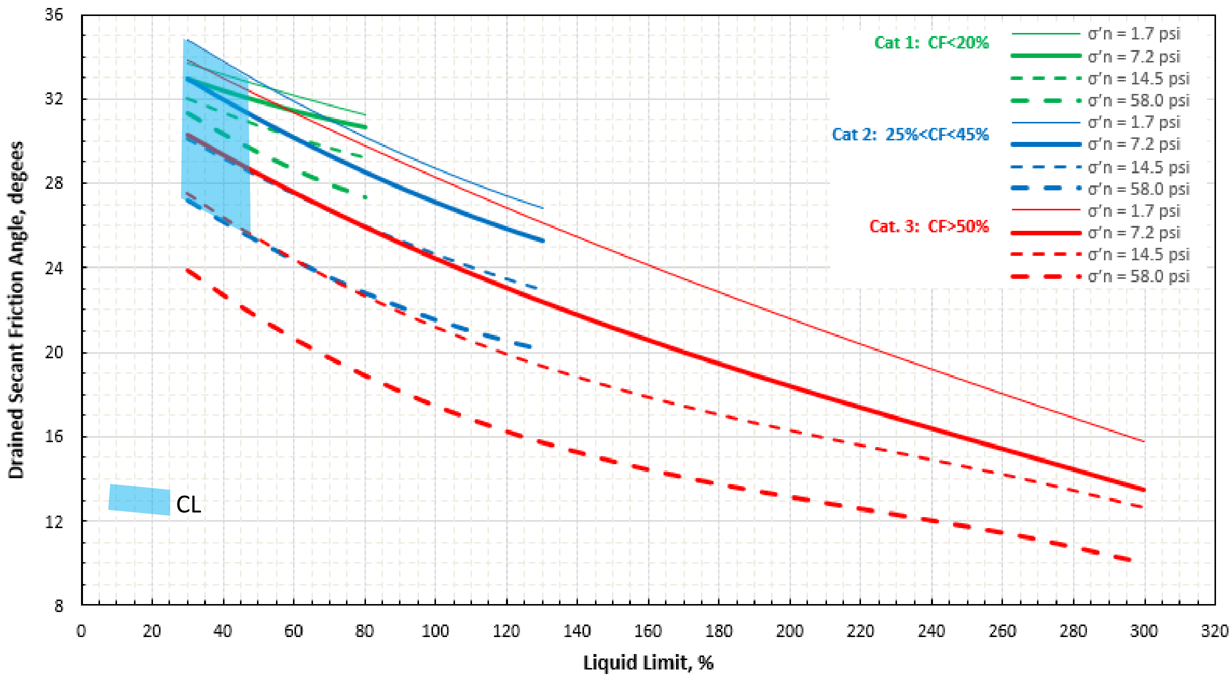
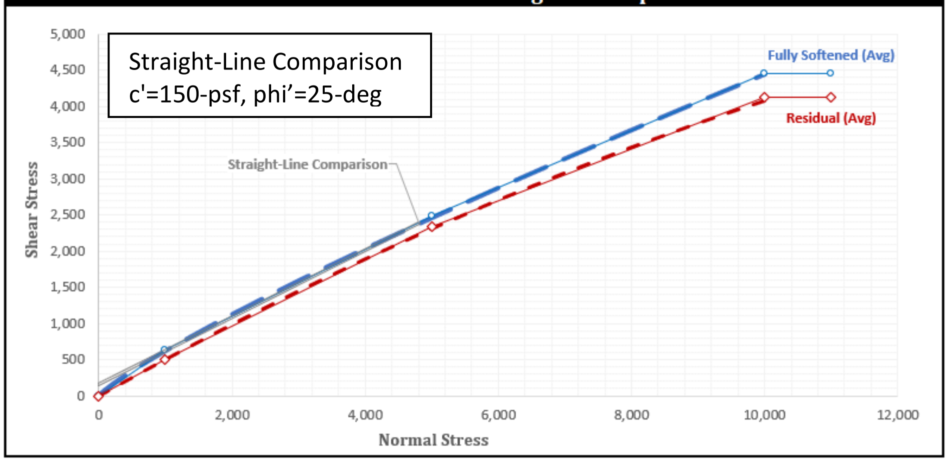


Figure 3. Secant Friction Angle for Clays, Stark and Hussain (2013)

4.2 Fully Softened Shear Strength Correlation (Cohesive Soils)

Fully softened shear strength curves developed by Castellanos et al (2016) were also evaluated for the lean clay. Average LL, PI and -200 values presented previously were used to develop fully softened curves and a straight-line comparison curve was selected based on the average fully softened curve. Figure 4 and Figure 5 show the fully softened curves and straight-line comparison for the CL fill and residual materials, respectively.

Dimensional Shear Strength Envelopes



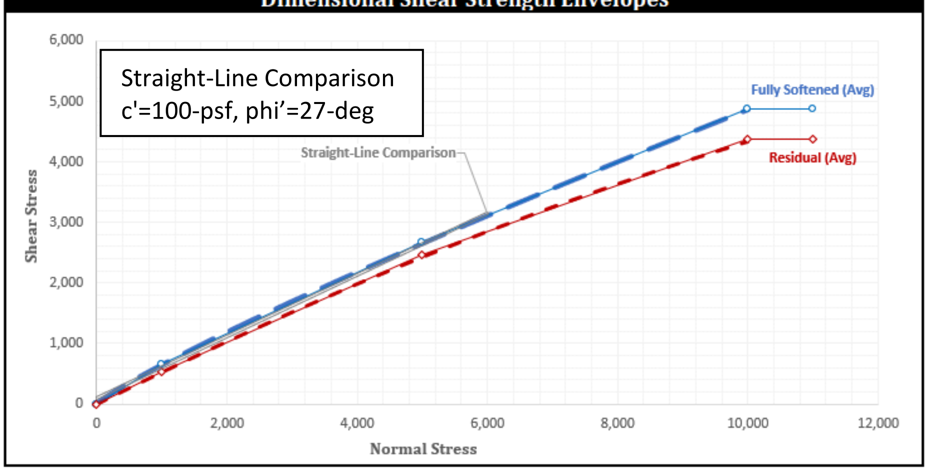
All Stress Units:	PSF	All Angle Units:	DEGREES

Summary of Input Data and Index Property Parameter Calculations

Material Description:	Lean Clay (Fill)				
Material Use/Loc.:	Shawnee, OK			Selected CF:	45.0 %
Liquid Limit:	37	Plasticity Index:	22	Cohesion Index (CI):	15.6
% Passing No. 200:	71	Clay Fraction (%CF):	45	CF*PI:	990

Figure 4. Fully Softened Curve for CL (Fill)

Dimensional Shear Strength Envelopes



All Stress Units:	PSF	All Angle Units:	DEGREES

Summary of Input Data and Index Property Parameter Calculations

Material Description:	Lean Clay (Residual)				
Material Use/Loc.:	Shawnee, OK			Selected CF:	40.0 %
Liquid Limit:	31	Plasticity Index:	17	Cohesion Index (CI):	11.7
% Passing No. 200:	69	Clay Fraction (%CF):	40	CF*PI:	680

Figure 5. Fully Softened Curve for CL (Residual)

4.3 Friction Angle by Correlation (Cohesionless Soils)

Blow counts and corrected blow counts were used to evaluate the friction angle for the sand materials. Correlations by Meyerhof were used to analyze effective friction angles for sands and gravels using the blow counts. The values for these correlations are for "clean" sands and silts and is recommended to reduce the values by 5 degrees for clayey sands. Plots showing the blow counts (N), corrected blow counts ((N1)60), and calculated friction angles are provided in Figure 6 and Figure 7 for fill and residual material respectively. Ranges and averages for the blow counts, corrected blow counts, and calculated friction angles are provided in Table 4 and Table 5. The calculated friction angles were capped at a maximum of 45 degrees.

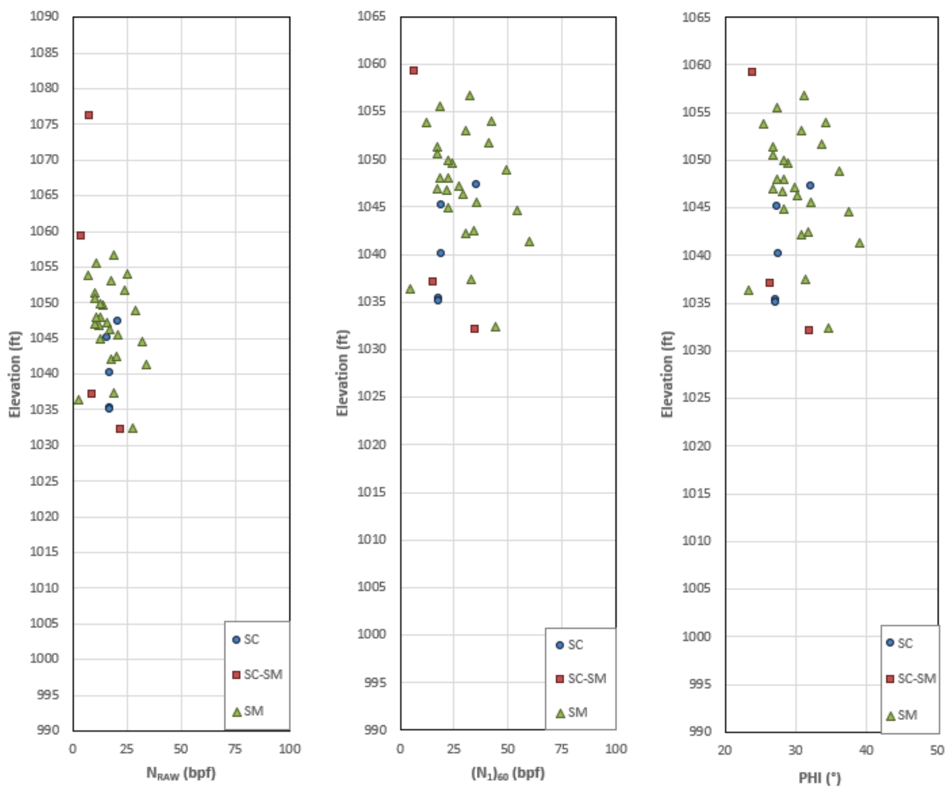


Figure 6. SPT and Friction Angle vs Elevation (Fill)

Table 4. Blow Count and Friction Angle Summary (Fill)

Material Type	Blow Count, N		Corrected Blow Count, (N1)60		Friction Angle	
	Range	Average	Range	Average	Range	Average
SC	16 – 17	17	18 – 19	19	27	27
SC-SM	4 – 22	11	7 – 35	18	24 – 32	27
SM	3 – 34	17	5 – 60	29	23 –39	30

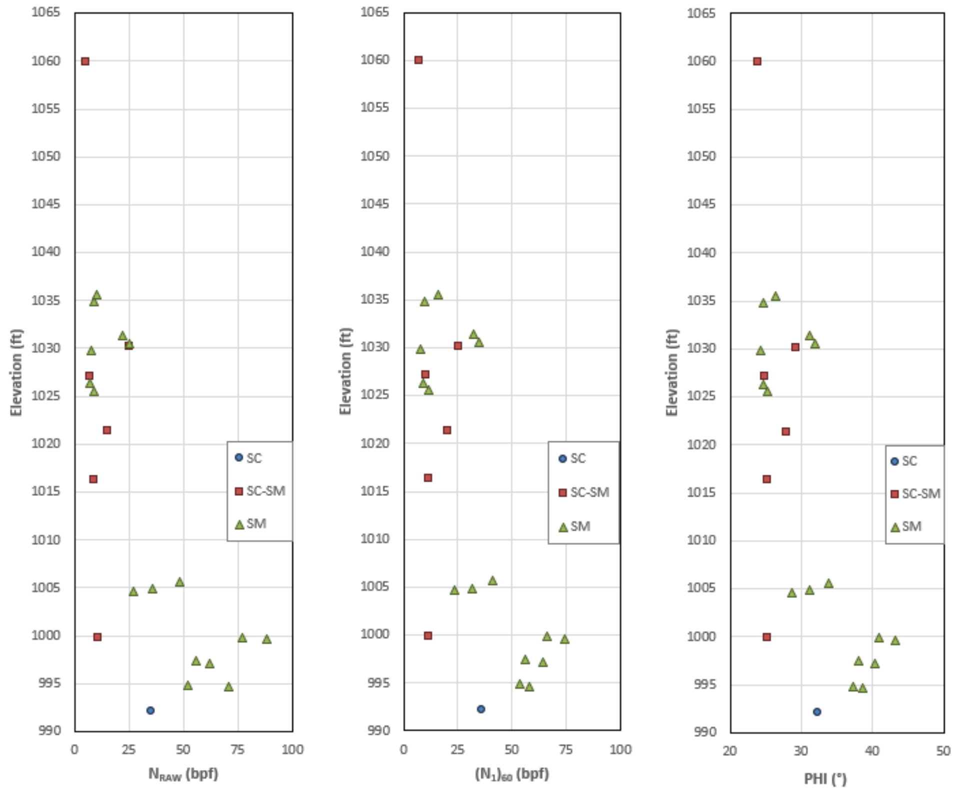


Figure 7. SPT and Friction Angle vs Elevation (Residual)

Table 5. Blow Count and Friction Angle Summary (Residual)

Material Type	Blow Count, N		Corrected Blow Count, (N1)60		Friction Angle	
	Range	Average	Range	Average	Range	Average
SC	35	35	36	36	32	32
SC-SM	5 – 25	12	7 – 25	14	24 – 29	26
SM	7 – 88	38	8 – 74	37	24 –43	33

5.0 ROCK PARAMETERS

The rock mass parameters were estimated using RocLab by Rocscience which utilized Hoek-Brown criteria for the highly weathered shale and weathered sandstone. Core samples were not obtained as part of Terracon's investigation; therefore, rock quality and compressive strength were estimated from visual descriptions on the logs and SPT blow count values, respectively. The density of the rock mass was estimated using an estimated density for similar materials. The resulting values to be used in the Hoek-Brown equations for the weathered and unweathered rock are summarized in Table 6.

Table 6. Selected Strength Parameter Summary for Encountered Material

Parameter	Highly Weathered Shale	Weathered Sandstone
Uniaxial Compressive Strength (σ_{ci})	20 ksf	30 ksf
Geologic Strength Index (GSI)	10 - 20	20 - 30
m_i	6	13
Disturbance Factor (D)	0	0
m_b	0.241 – 0.345	0.747 – 1.067
s	0.0000454 - 0.0001	0.0001 – 0.0004
a	0.585 - 0.544	0.544 – 0.522
Wet Density	130 pcf	134 pcf
Modulus Ratio	--	--

where: m_i = material constant for intact rock
D = disturbance factor resulting from blast damage and/or stress relaxation
m_b = reduced value (for the rock mass) of the material constant m_i (for intact rock)
s and a = constants which depend on the characteristics of the rock mass.

The GSI, m_i, and D values were conservatively estimated from the descriptions on the boring logs. The values for m_b, s, and a are calculated from the visual assessment values using the following equations.

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

The GSI is the Geologic Strength Index, which is a system based on observations of the rock mass used in determining the mechanical properties of the rock mass. The GSI chart concentrates on the description of two factors, rock structure and block surface conditions. From observations of the bedrock, A GSI range of about 10 to 20 was estimated for the highly weathered shale rock mass, and a GSI range of about 20 to 30 was estimated for the weathered sandstone.

The disturbance factor (D) ranges from 0 to 1, with 1 being the most disturbed. The disturbance factor is determined based on both the type of excavation and the methods being used. For small scale blasting in civil engineering slopes, the disturbance factor ranges from 0.7, for good blasting, to 1.0 for poor blasting. The disturbance factor typically only applies to the disturbed portion of the rock mass, from the surface to some depth below the surface. A disturbance factor of 0.2 was selected for the unweathered rock in excavations, with the assumption that the rock may undergo some disturbance due to excavation methods and rebound from overburden removal. A disturbance factor of zero was applied to the bedrock materials beneath the existing embankment sections, since they are considered to be relatively undisturbed.

An equivalent Mohr-Coulomb cohesion and friction angle was estimated by fitting an average linear relationship to the Hoek-Brown shear stress curve for the rock mass in the joint direction. RocLab uses a maximum confining stress (σ_{3max}) equivalent to about one-quarter of the uniaxial compressive strength in its estimate of the Mohr-Coulomb parameters.

The estimated rock mass parameters are summarized in Table 7 and will be used in the slope stability analyses and sliding stability analyses for the spillway structure.

Table 7: Rock Mass Strength Summary

Material	Cohesion, c (psf)	Friction angle, ϕ (degrees)
Highly Weathered Shale	300 – 450	14.4 – 17.7
Weathered Sandstone	920 – 1,160	23.6 – 26.7

6.0 ANALYSIS OF SEEPAGE PARAMETERS

6.1 Hydraulic Conductivity from Correlation

Hydraulic conductivity is commonly estimated from correlations and engineering judgment. USBR Design Manual No. 13 (2014) provides typical hydraulic conductivity values for a variety of materials and provides a suitable basis for the selection of estimated values to supplement those obtained from the field and laboratory testing. The typical values are presented in Figure 8, Figure 9, and Figure 10.

Unified Soil Classification	k_V Range (ft/yr or x10^{-6} cm/s)*
GM-SM	0.0 to 10.0
GM or GC	0.0 to 10.0
SP-SM	0.0 to 10.0
SM	0.0 to 10.0
SM-SC	0.0 to 3.0
SM-ML	0.0 to 10.0
SC	0.0 to 3.0
ML	0.0 to 10.0
ML-CL	0.0 to 1.0
CL	0.0 to 1.0
MH	0.0 to 0.1

Unified Soil Classification	k_V Range (ft/yr or x10^{-6} cm/s)
GP	2,000 to 1,000,000
GW	1,000 to 100,000
GP-SP	1,000 to 50,000
GW-SW	500 to 5,000
GM	10 to 500
SP (medium to coarse)	10,000 to 20,000
SP (fine to medium)	5,000 to 10,000
SP (very fine to fine)	500 to 5,000
SW	300 to 5,000
SP-SM	10 to 1,000
SM	10 to 500

Figure 8. Typical Vertical Hydraulic Conductivity of Natural Soil (USBR 2014)

Soil	k_H Range (ft/yr or 10^{-6} cm/s)
Gravel, open-work	>2,000,000
Gravel (GP)	200,000 to 2,000,000
Gravel (GW)	10,000 to 1,000,000
Sand, coarse (SP)	10,000 to 500,000
Sand, medium (SP)	1,000 to 100,000
Sand, fine (SP)	500 to 50,000
Sand (SW)	100 to 50,000
Sand, silty (SM)	100 to 10,000
Sand, clayey (SC)	1 to 1,000
Silt (ML)	1 to 1,000
Clay (CL)	~0 to 3

Figure 9. Typical Horizontal Hydraulic Conductivity of Natural Soil (USBR 2014)

Material	k_V Range (ft/yr or x10^{-8} cm/s)
Coarse sand and gravel	150,000 to 500,000
Medium to coarse sand	50,000 to 150,000
Fine to medium sand	10,000 to 50,000

Figure 10. Hydraulic Conductivity of Drain Material (USBR 2014)

The movement of water through unsaturated soil is influenced by suction behavior and can be characterized using soil-water characteristic curves (SWCC). When modeling seepage, the calculations along the phreatic surface are influenced by this behavior. Several empirical correlations are published that allow the approximation of the hydraulic conductivity function and the volumetric water content function based upon the soil classification. Estimation of the parameters was made using the SEEP/W module in GeoStudio 2023, which is based on Fredlund, Xing and Huang (1994). Functions were estimated for the soil materials based on grain size data, liquid limit, the estimated saturated water content, and hydraulic conductivity at saturation, which are summarized in Table 8 and Table 9.

Table 8. Parameters used to Estimate Hydraulic Conductivity Functions for Encountered Fill Material

Material Type	Grain Size Data Ranges		Liquid Limit Range	Selected Grain Size Data		Selected Liquid Limit	Estimated Saturated Water Content	Selected Hydraulic Conductivity (ft/sec)
	D10 (mm)	D60 (mm)		D10 (mm) (2)	D60 (mm)			
CL	-	0.031-0.082	23-46	0.0001	0.055	35	0.35	3E-08
SC(1)	-	-	-	-	-	-	0.28	3.3E-07
SC-SM	-	0.15	24-40	0.0001	0.15	35	0.28	1.3E-06
SM(1)	-	-	-	-	-	-	0.3	9.8E-06

(1)grain size and LL estimated from sample functions

(2)A value of 0.0001 was chosen for materials that had no D10 data

Table 9. Parameters used to Estimate Hydraulic Conductivity Functions for Encountered Residual Material

Material Type	Grain Size Data Ranges		Liquid Limit Range	Selected Grain Size Data		Selected Liquid Limit	Estimated Saturated Water Content	Selected Hydraulic Conductivity (ft/sec)
	D10 (mm)	D60 (mm)		D10 (mm) (2)	D60 (mm)			
CL	-	0.013	21-48	0.0001	0.013	35	0.35	3E-08
SC(1)	-	-	-	-	-	-	0.28	3.3E-07
SC-SM(1)	-	-	-	-	-	-	0.28	1.3E-06
SM	0.002	0.16	NP	0.002	0.16	NP	0.3	9.8E-06

(1)grain size and LL estimated from sample functions

(2)A value of 0.0001 was chosen for materials that had no D10 data

7.0 PARAMETER ANALYSIS SUMMARY

The strength parameters selected for design for the encountered materials are summarized in Table 10 and Table 11. The parameters for the CL materials were selected from the fully softened curves and straight-line comparisons previously discussed. CH materials were excluded from this summary since they are only found in small quantities in select borings. The parameters for the SC, SC-SM, and SM materials were selected from correlations based on blow-count values using an average value or value on the lower-end of the range. Total (CU) strength values for the clays were selected assuming the total cohesion is about twice the drained cohesion and assuming the total friction angle is about two-thirds the drained friction angle.

Table 10. Selected Strength Parameter Summary for Encountered Fill Material

Material Type	Moist Unit Weight (pcf)	Drained Strength		CU Strength	
		c' (psf)	ϕ' (deg)	c (psf)	ϕ (deg)
CL	125	150	25	300	17
SC	120	50	27	100	18
SC-SM	120	0	27	n/a	n/a
SM	120	0	30	n/a	n/a

Table 11. Selected Strength Parameter Summary for Encountered Residual Material

Material Type	Moist Unit Weight (pcf)	Drained Strength		CU Strength	
		c' (psf)	ϕ' (deg)	c (psf)	ϕ (deg)
CL	125	100	27	200	18
SC	130	50	32	n/a	n/a
SC-SM	125	0	26	n/a	n/a
SM	130	0	33	n/a	n/a
Shale	130	400	16	n/a	n/a
Sandstone	134	950	24	n/a	n/a

The seepage parameters selected for the seepage analysis are presented in Table 12 and Table 13 below.

Table 12: Selected Seepage Parameter Summary for Encountered Fill Material

Material Type	Permeability		Kv/Kh Ratio
	Kh (cm/sec)	Kh (ft/sec)	
CL	5E-08	1.6E-08	0.25
SC	4E-05	1.3E-06	0.25
SC-SM	2E-05	6E-07	0.25
SM	3E-04	9.8E-06	0.33
Shale	1.01E-07	3.3E-09	0.1
Sandstone	9.14E-07	3E-08	1

Table 13: Selected Seepage Parameter Summary for Encountered Residual Material

Material Type	Permeability		Kv/Kh Ratio
	Kh (cm/sec)	Kh (ft/sec)	
CL	2E-06	6.6E-08	0.25
SC	4E-05	1.3E-06	0.25
SC-SM	2E-05	6E-07	0.25
SM	3E-04	9.8E-06	0.33
Shale	1.01E-07	3.3E-09	0.1
Sandstone	9.14E-07	3E-08	1

8.0 TECHNICAL REFERENCES

Castellanos, B.A., Brandon, T.L., VandenBerge, D.R., "Use of Fully Softened Shear Strength in Slope Stability Analysis" Landslides, Vol. 13, No. 4, (2016)

Duncan, J. M. "Friction Angles for Sand, Gravel and Rockfill," Notes of a lecture presented at the Kenneth L. Lee Memorial Seminar, Long Beach, California, (April 28, 2004)

Duncan, J. M., Wright, S. G., "Soil Strength and Slope Stability," John Wiley & Sons (2005)

Duncan, J. M., Horz, R. C. and Yang, T. L. "Shear Strength Correlations for Geotechnical Engineering (CGPR #4)," Virginia Tech Department of Engineering, (August 1989)

Electrical Power Research Institute Report EL-6800, Manual on Estimating Soil Properties (1990)

Federal Highway Administration Report IF-02-034, "Geotechnical Engineering Circular 5 - Evaluation of Soil and Rock Properties" (2002)

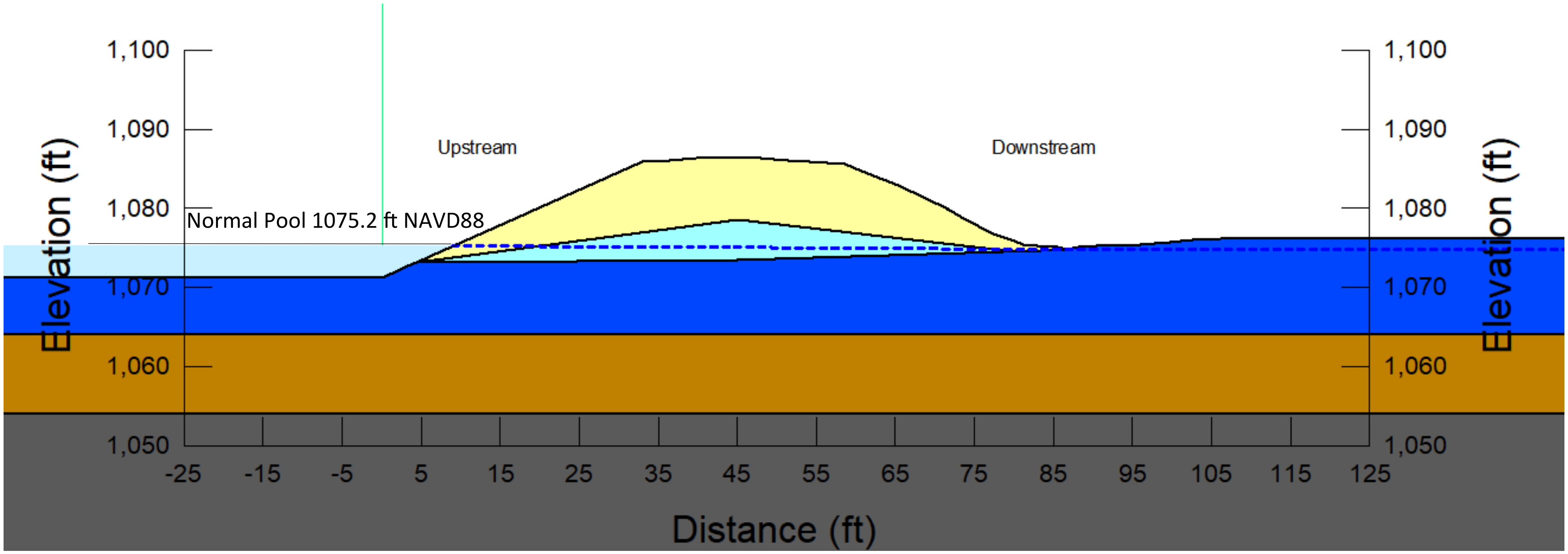
Fredlund and Xing, *Predicting the Permeability Function for Unsaturated Soils Using the Soil-Water Characteristic Curve*. Canadian Geotechnical Journal, 31(4), 533-546, 1994.

Gamez, J. A. and Stark, T. D. "Fully Softened Shear Strength at Low Stresses for Levee and Embankment Design." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 140, No. 9, (September 2014)

Stark, T. D., and Hussain, M. "Empirical Correlations: Drained Shear Strength for Slope Stability Analyses." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 139, No. 6, (June 2013)

U.S. Bureau of Reclamation. Design Standard No. 13: Embankment Dams, (September 2014)

Seepage and Slope Stability Figures



Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Rotation (°)	Sat Kx (ft/sec)	Volumetric Water Content	Compressibility (/psf)
	Fill - Clayey Sand (SC)	Saturated / Unsaturated	Fill-Clayey Sand (SC)	Fill - Clayey Sand (SC)	0.25	0			
	Fill - Lean Clay (CL)	Saturated / Unsaturated	Fill-Lean Clay (CL)	Fill - Lean Clay (CL)	0.25	0			
	Residual - Lean Clay (CL)	Saturated / Unsaturated	Residual-Lean Clay (CL)	Residual - Lean Clay (CL)	0.25	0			
	Sandstone	Saturated Only			1	0	3e-08	0	0
	Shale	Saturated Only			0.1	0	3.3e-09	0	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Geometry **Embankment Cross Section Geometry (STA: 5+50)**	**A.1**
PREPARED: IJD			

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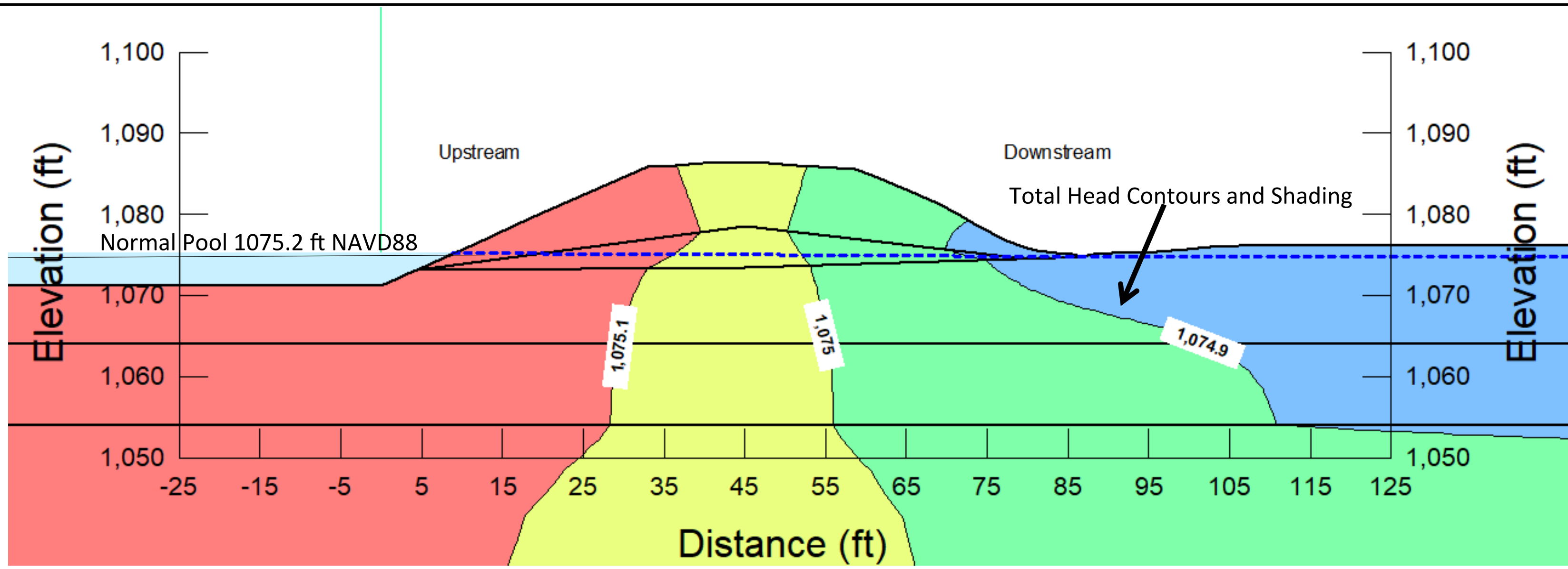


Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Rotation (°)	Sat Kx (ft/sec)	Volumetric Water Content	Compressibility (/psf)
	Fill - Clayey Sand (SC)	Saturated / Unsaturated	Fill-Clayey Sand (SC)	Fill - Clayey Sand (SC)	0.25	0			
	Fill - Lean Clay (CL)	Saturated / Unsaturated	Fill-Lean Clay (CL)	Fill - Lean Clay (CL)	0.25	0			
	Residual - Lean Clay (CL)	Saturated / Unsaturated	Residual-Lean Clay (CL)	Residual - Lean Clay (CL)	0.25	0			
	Sandstone	Saturated Only			1	0	3e-08	0	0
	Shale	Saturated Only			0.1	0	3.3e-09	0	0

FNI PRO-	SWN24427
DATE:	DECEMBER 2024
PREPARED:	IJD

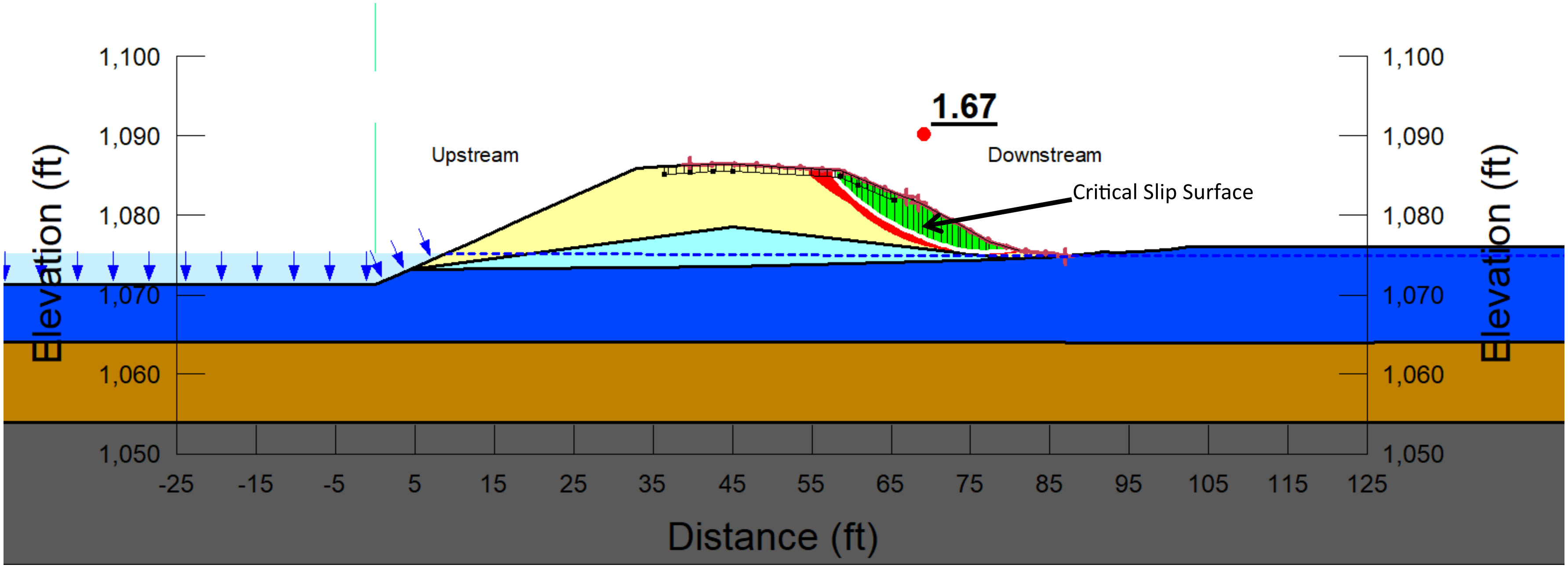
FREESE AND NICHOLS
5100 EAST SKELLY DRIVE
SUITE 602
TULSA, OKLAHOMA 74135

City of Shawnee
Shawnee Twin Lake Dam #1

Steady State Seepage Analysis
Embankment Cross Section (STA: 5+50) - Normal Pool

PLATE

A.2



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)
	Fill - Clayey Sand (SC)	Mohr-Coulomb	120	50	27	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0
	Sandstone	Mohr-Coulomb	134	950	24	0
	Shale	Mohr-Coulomb	130	400	16	0

FNI PRO- SWN24427 DATE: DECEMBER 2024 PREPARED: IJD	FREESE AND NICHOLS 5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	City of Shawnee **Shawnee Twin Lake Dam #1** Static Slope Stability Analysis—Steady State Seepage **Embankment Cross Section (STA: 5+50) - Downstream Slope at Normal Pool**	**PLATE** **A.3**

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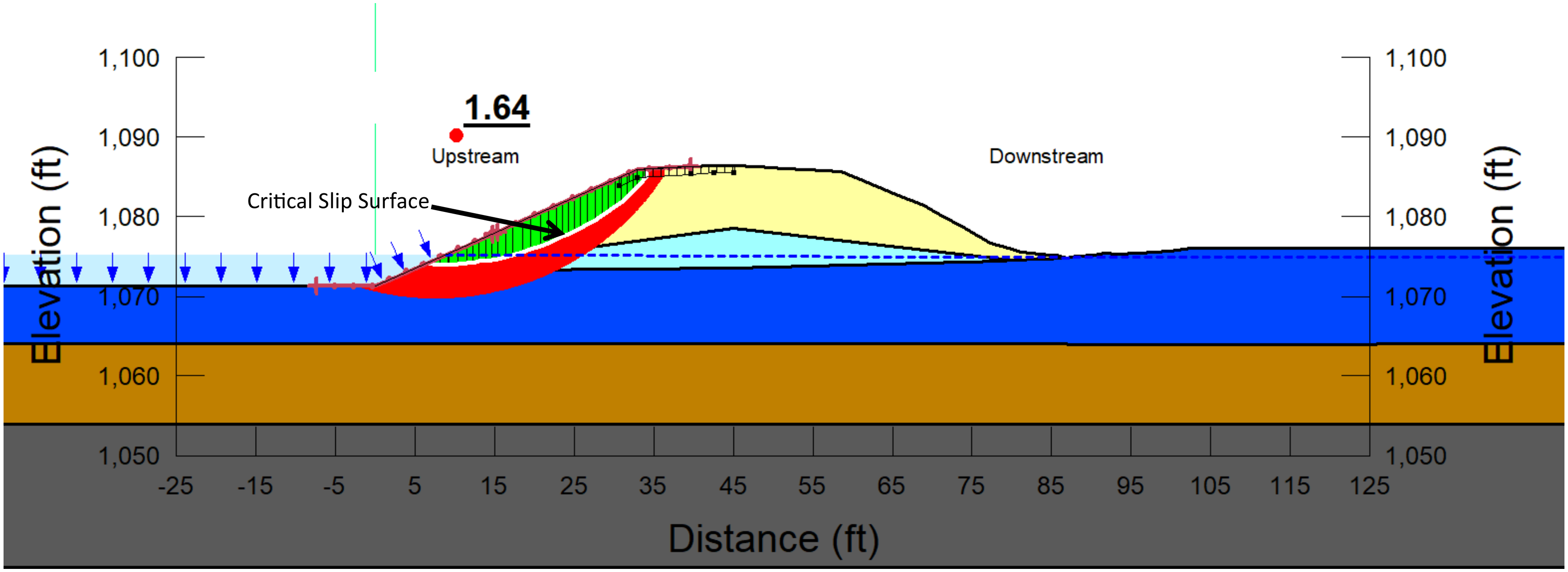

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)
	Fill - Clayey Sand (SC)	Mohr-Coulomb	120	50	27	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0
	Sandstone	Mohr-Coulomb	134	950	24	0
	Shale	Mohr-Coulomb	130	400	16	0

FNI PRO-: SWN24427

DATE: DECEMBER 2024

PREPARED: IJD

FREESE AND NICHOLS
5100 EAST SKELLY DRIVE
SUITE 602
TULSA, OKLAHOMA 74135

City of Shawnee

Shawnee Twin Lake Dam #1

Static Slope Stability Analysis— Steady State Seepage

Embankment Cross Section (STA: 5+50) - Upstream Slope at Normal Pool

PLATE

A.4

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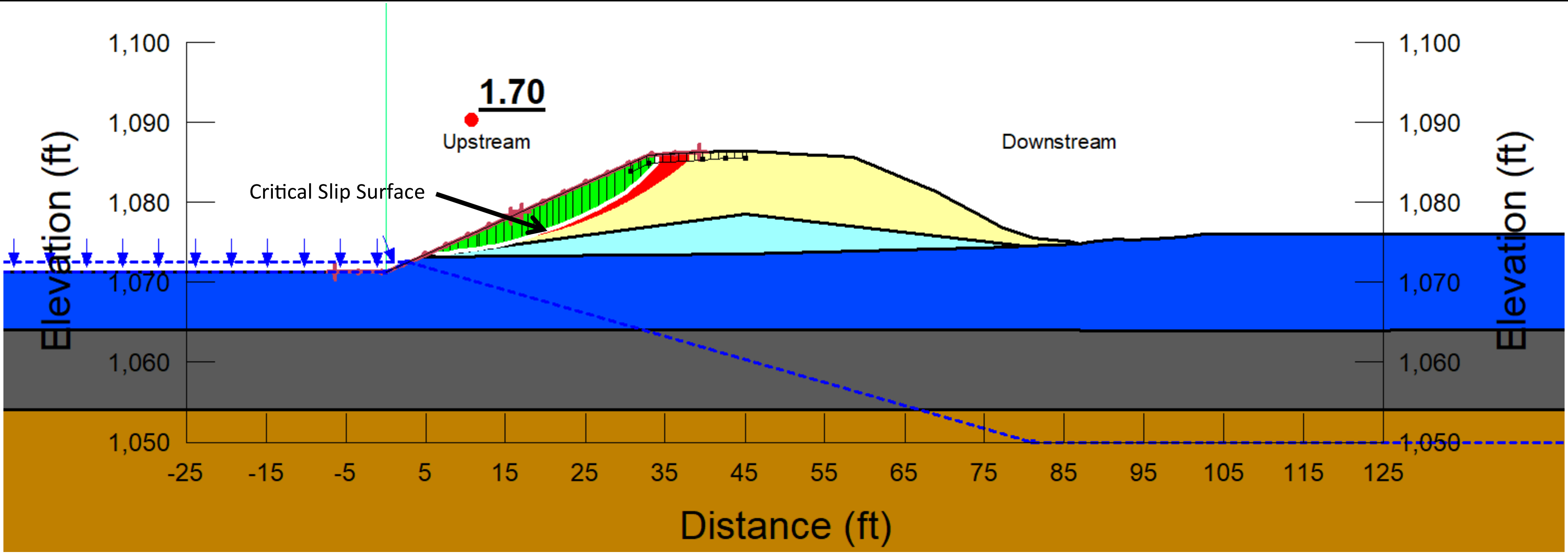

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Surface	Piezometric Surface After Drawdown
	Fill - Clayey Sand (SC)	Mohr-Coulomb	120	50	27	0	100	18	1	2
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17	1	2
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18	1	2
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0	1	2
	Shale	Mohr-Coulomb	130	400	16	0	0	0	1	2

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Rapid Drawdown **Embankment Cross Section (STA: 5+50) - Rapid Drawdown**	**A.5**
PREPARED: IJD			

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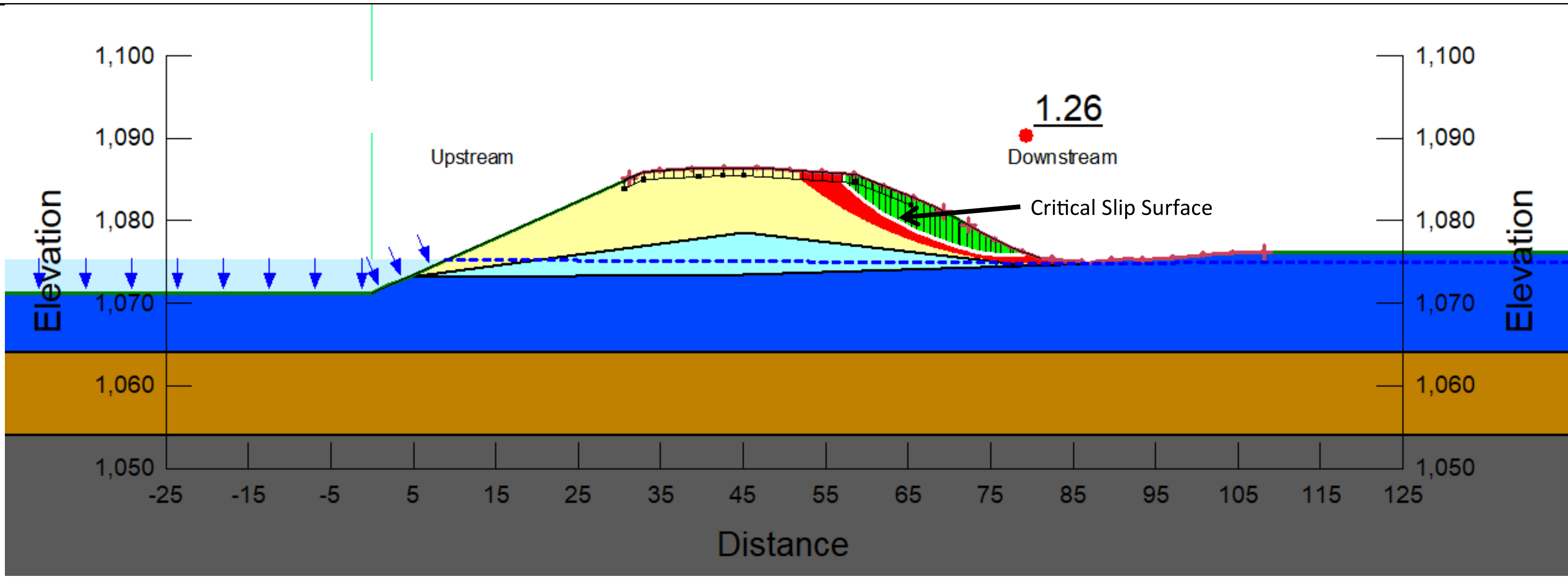

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Sand (SC)	Mohr-Coulomb	120	50	27	0	100	18
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Seismic Slope Stability Analysis—Effective Strength **Embankment Cross Section (STA: 5+50) - Downstream Slope at Normal Pool**	**A.6**
PREPARED: IJD			

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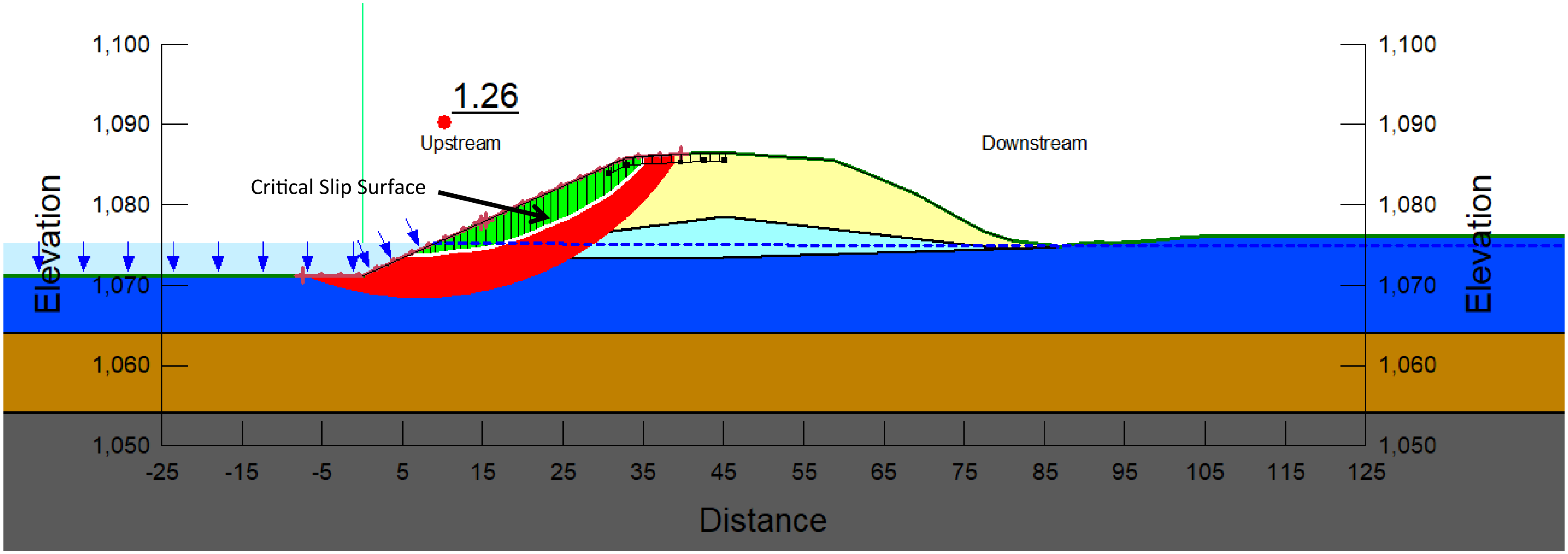

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Sand (SC)	Mohr-Coulomb	120	50	27	0	100	18
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Seismic Slope Stability Analysis—Effective Strength **Embankment Cross Section (STA: 5+50) - Upstream Slope at Normal Pool**	**A.7**
PREPARED: IJD			

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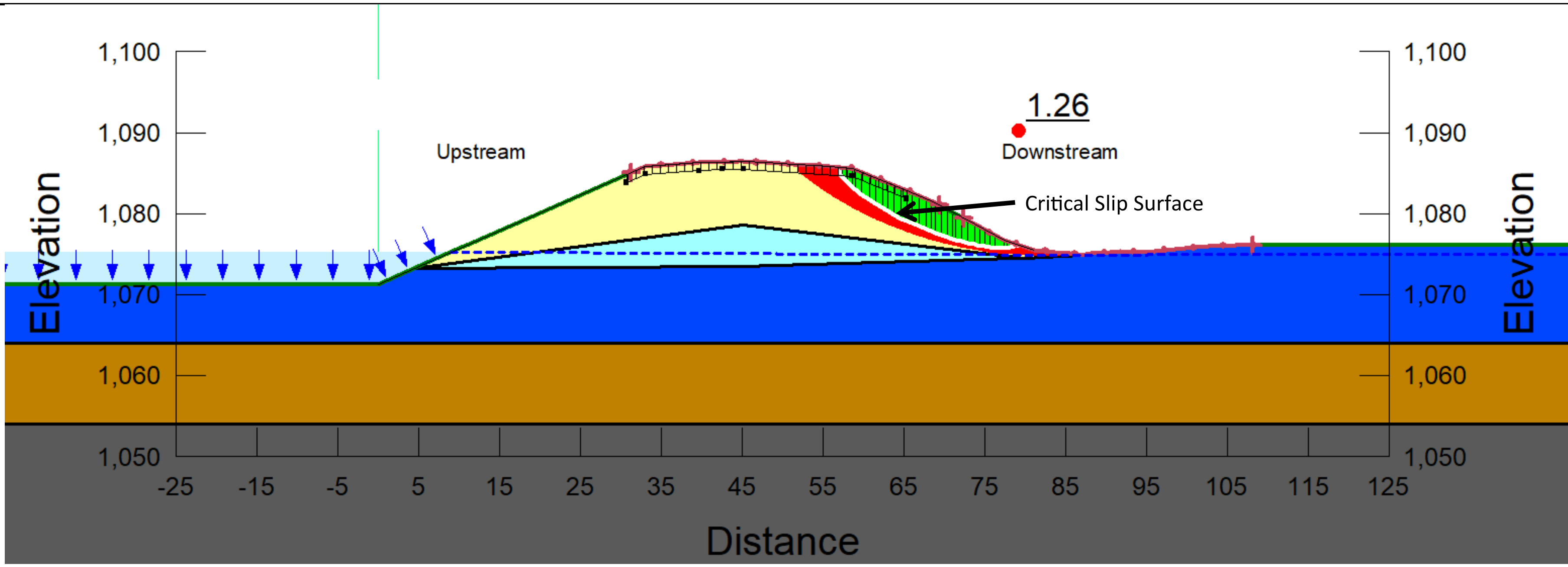

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Sand (SC)	Mohr-Coulomb	120	50	27	0	100	18
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO- SWN24427

DATE: DECEMBER 2024

PREPARED: IJD

FREESE AND NICHOLS

5100 EAST SKELLY DRIVE
SUITE 602
TULSA, OKLAHOMA 74135

City of Shawnee

Shawnee Twin Lake Dam #1

Seismic Slope Stability Analysis—Undrained Strength

Embankment Cross Section (STA: 5+50) - Downstream Slope at Normal Pool

PLATE

A.8

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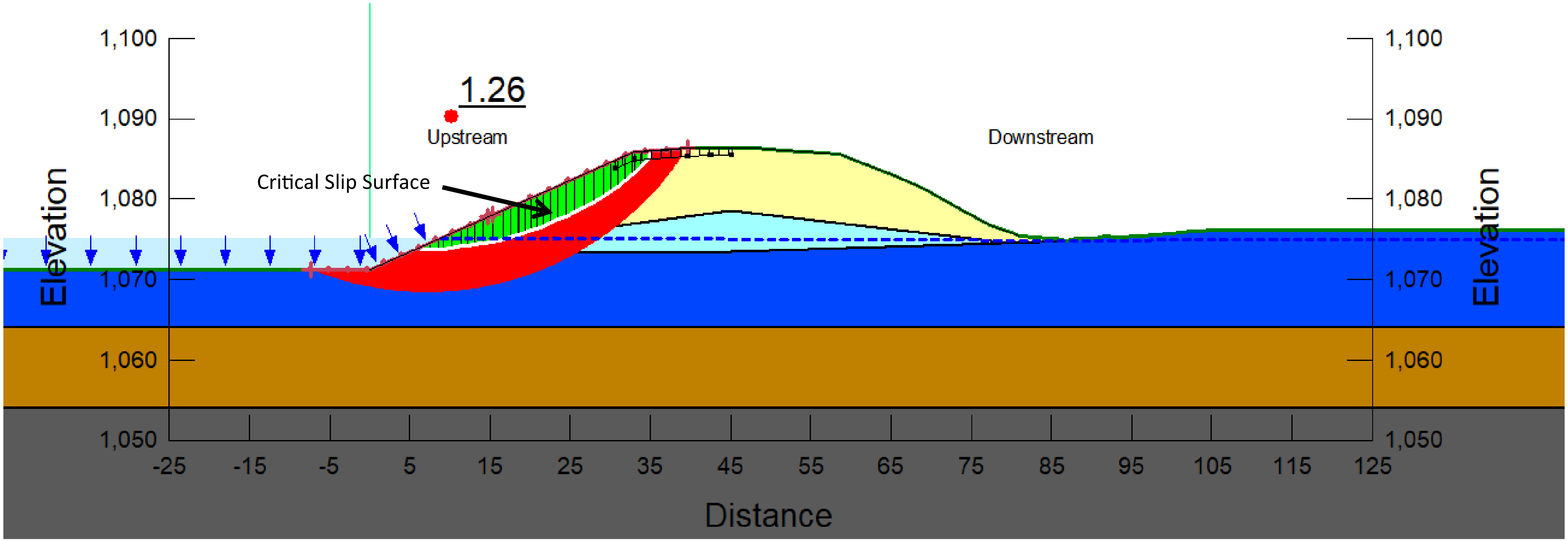

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Sand (SC)	Mohr-Coulomb	120	50	27	0	100	18
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Seismic Slope Stability Analysis—Undrained Strength **Embankment Cross Section (STA: 5+50) - Upstream Slope at Normal Pool**	**A.9**
PREPARED: IJD			

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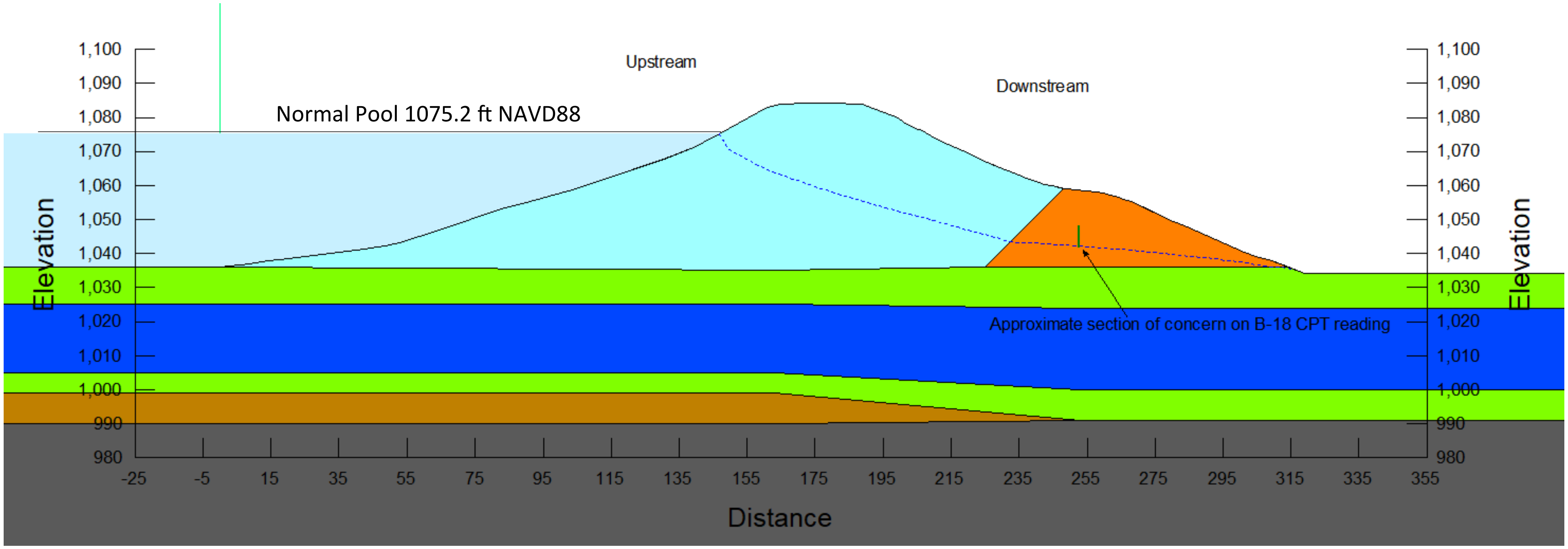

Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Rotation (°)	Sat Kx (ft/sec)	Volumetric Water Content	Compressibility (/psf)
	Fill - Clayey Silty Sand (SC-SM)	Saturated / Unsaturated	Fill-Clayey Silty Sand (SC-SM)	Fill - Clayey Silty Sand (SC-SM)	0.25	0			
	Fill - Lean Clay (CL)	Saturated / Unsaturated	Fill-Lean Clay (CL)	Fill - Lean Clay (CL)	0.25	0			
	Residual - Lean Clay (CL)	Saturated / Unsaturated	Residual-Lean Clay (CL)	Residual - Lean Clay (CL)	0.25	0			
	Residual - Silty Sand (SM)	Saturated / Unsaturated	Residual-Silty Sand (SM)	Residual - Silty Sand (SM)	0.33	0			
	Sandstone	Saturated Only			1	0	3e-08	0	0
	Shale	Saturated Only			0.1	0	3.3e-09	0	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Geometry **Embankment Cross Section Geometry (STA: 17+00)**	**A.10**
PREPARED: IJD			

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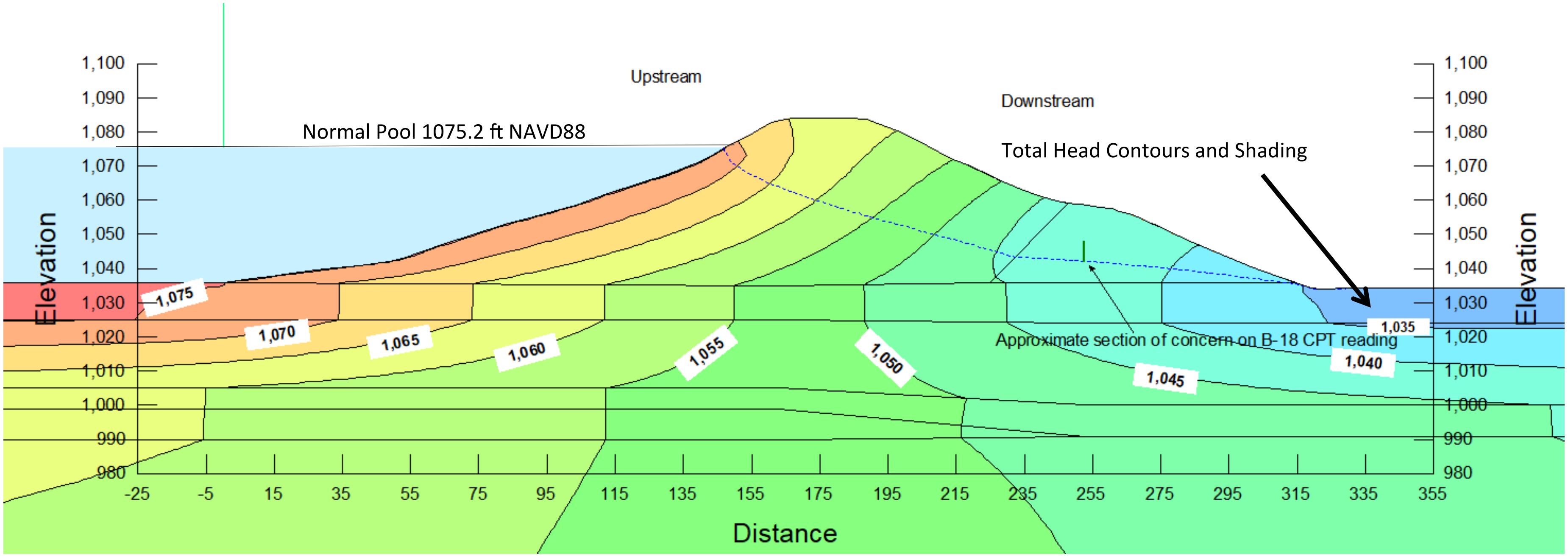

Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Rotation (°)	Sat Kx (ft/sec)	Volumetric Water Content	Compressibility (/psf)
	Fill - Clayey Silty Sand (SC-SM)	Saturated / Unsaturated	Fill-Clayey Silty Sand (SC-SM)	Fill - Clayey Silty Sand (SC-SM)	0.25	0			
	Fill - Lean Clay (CL)	Saturated / Unsaturated	Fill-Lean Clay (CL)	Fill - Lean Clay (CL)	0.25	0			
	Residual - Lean Clay (CL)	Saturated / Unsaturated	Residual-Lean Clay (CL)	Residual - Lean Clay (CL)	0.25	0			
	Residual - Silty Sand (SM)	Saturated / Unsaturated	Residual-Silty Sand (SM)	Residual - Silty Sand (SM)	0.33	0			
	Sandstone	Saturated Only			1	0	3e-08	0	0
	Shale	Saturated Only			0.1	0	3.3e-09	0	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Steady State Seepage Analysis **Embankment Cross Section (STA: 17+00) - Normal Pool**	**A.11**
PREPARED: IJD			

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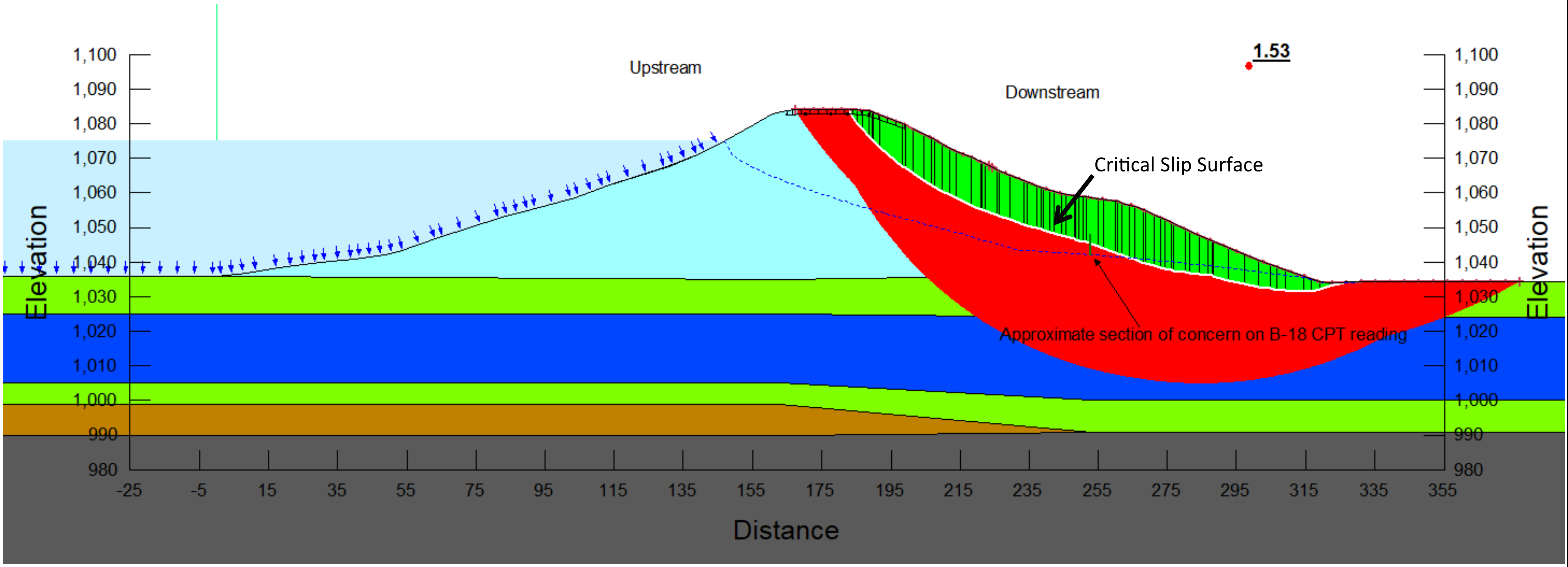

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)
	Fill - Clayey Silty Sand (SC-SM)	Mohr-Coulomb	120	0	27	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0
	Residual - Silty Sand (SM)	Mohr-Coulomb	130	0	33	0
	Sandstone	Mohr-Coulomb	134	950	24	0
	Shale	Mohr-Coulomb	130	400	16	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Static Slope Stability Analysis—Steady State Seepage **Embankment Cross Section (STA: 17+00) - Downstream Slope at Normal Pool**	**A.12**
PREPARED: IJD			

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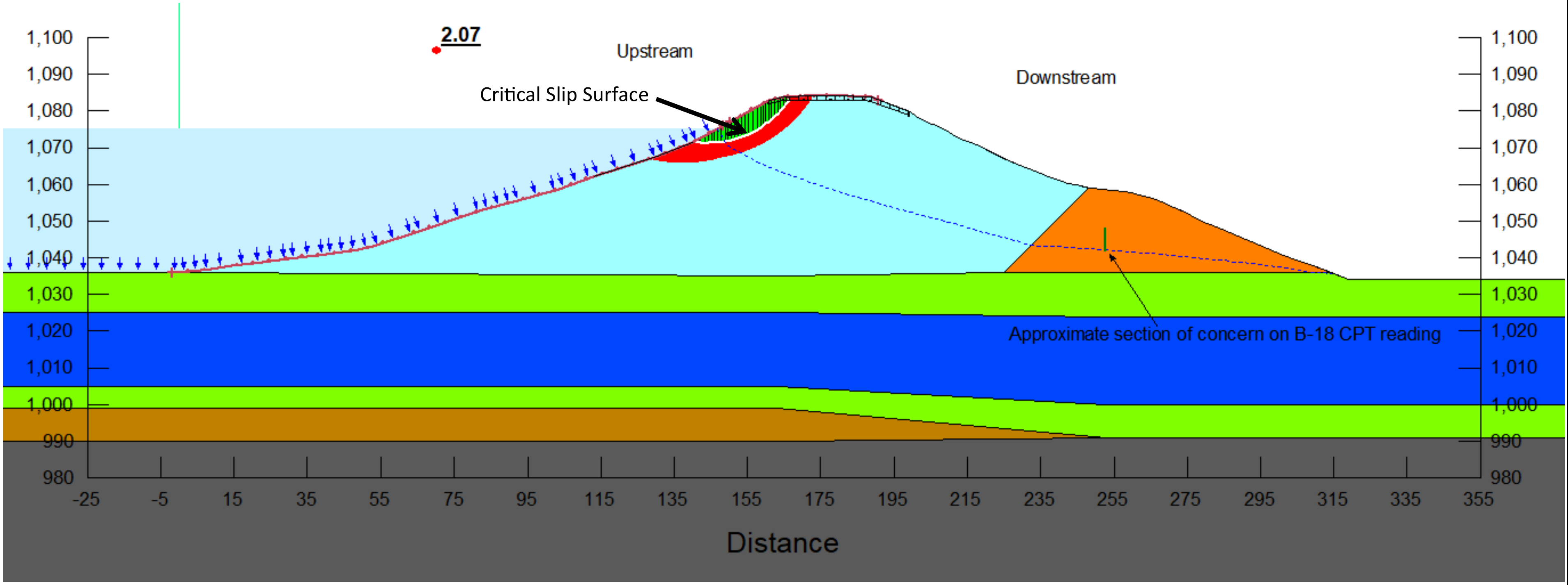

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)
	Fill - Clayey Silty Sand (SC-SM)	Mohr-Coulomb	120	0	27	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0
	Residual - Silty Sand (SM)	Mohr-Coulomb	130	0	33	0
	Sandstone	Mohr-Coulomb	134	950	24	0
	Shale	Mohr-Coulomb	130	400	16	0

FNI PRO- SWN24427

DATE: DECEMBER 2024

PREPARED: IJD

FREESE AND NICHOLS

5100 EAST SKELLY DRIVE
SUITE 602
TULSA, OKLAHOMA 74135

City of Shawnee

Shawnee Twin Lake Dam #1

Static Slope Stability Analysis—Steady State Seepage

Embankment Cross Section (STA: 17+00) - Upstream Slope at Normal Pool

PLATE

A.13

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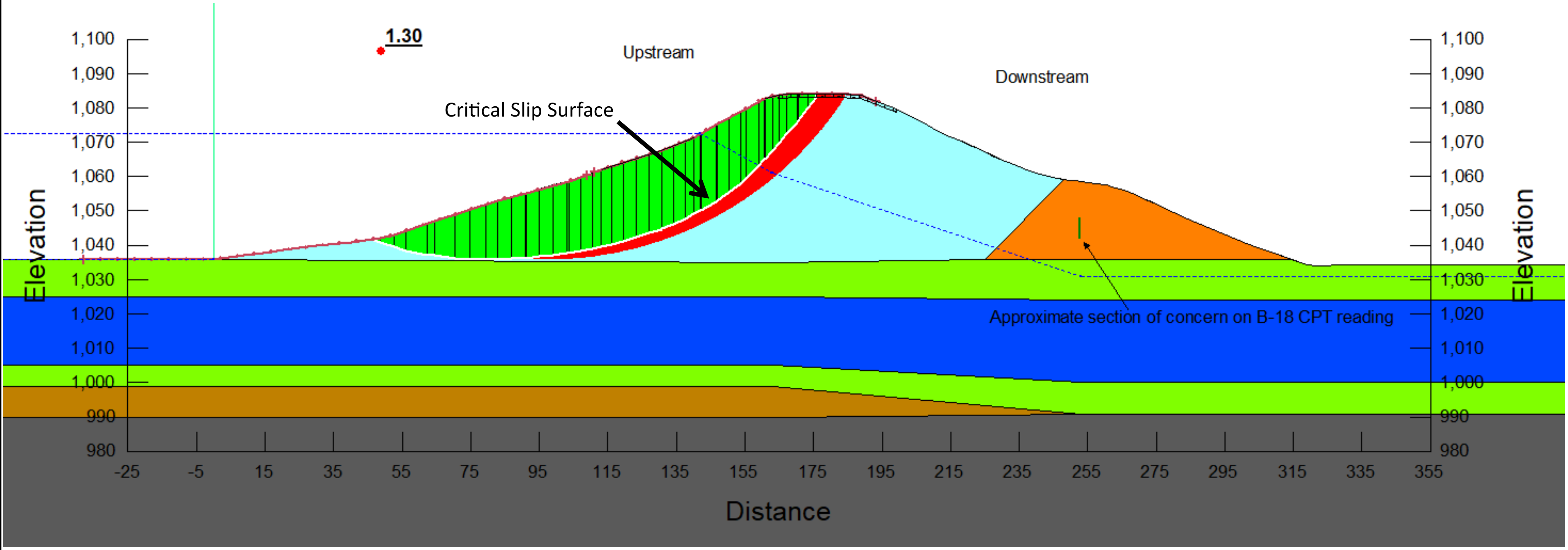

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Surface	Piezometric Surface After Drawdown
	Fill - Clayey Silty Sand (SC-SM)	Mohr-Coulomb	120	0	27	0	0	0	1	2
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17	1	2
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18	1	2
	Residual - Silty Sand (SM)	Mohr-Coulomb	130	0	33	0	0	0	1	2
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0	1	2
	Shale	Mohr-Coulomb	130	400	16	0	0	0	1	2

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Rapid Drawdown **Embankment Cross Section (STA: 17+00) - Rapid Drawdown —Upstream Slope**	**A.14**
PREPARED: IJD			

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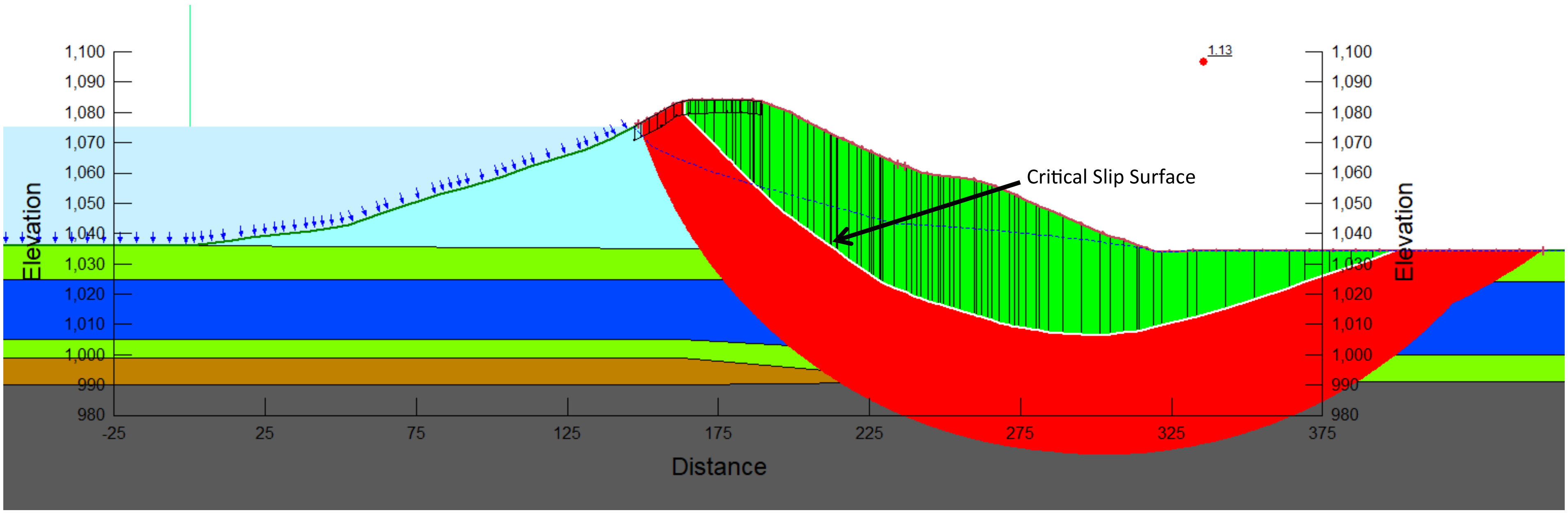

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Silty Sand (SC-SM)	Mohr-Coulomb	120	0	27	0	0	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Residual - Silty Sand (SM)	Mohr-Coulomb	130	0	33	0	0	0
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO- SWN24427	FREESE AND NICHOLS	City of Shawnee **Shawnee Twin Lake Dam #1**	**PLATE**
DATE: DECEMBER 2024	5100 EAST SKELLY DRIVE SUITE 602 TULSA, OKLAHOMA 74135	Seismic Slope Stability Analysis—Effective Strength **Embankment Cross Section (STA: 17+00) - Downstream Slope at Normal Pool**	**A.15**
PREPARED: IJD			

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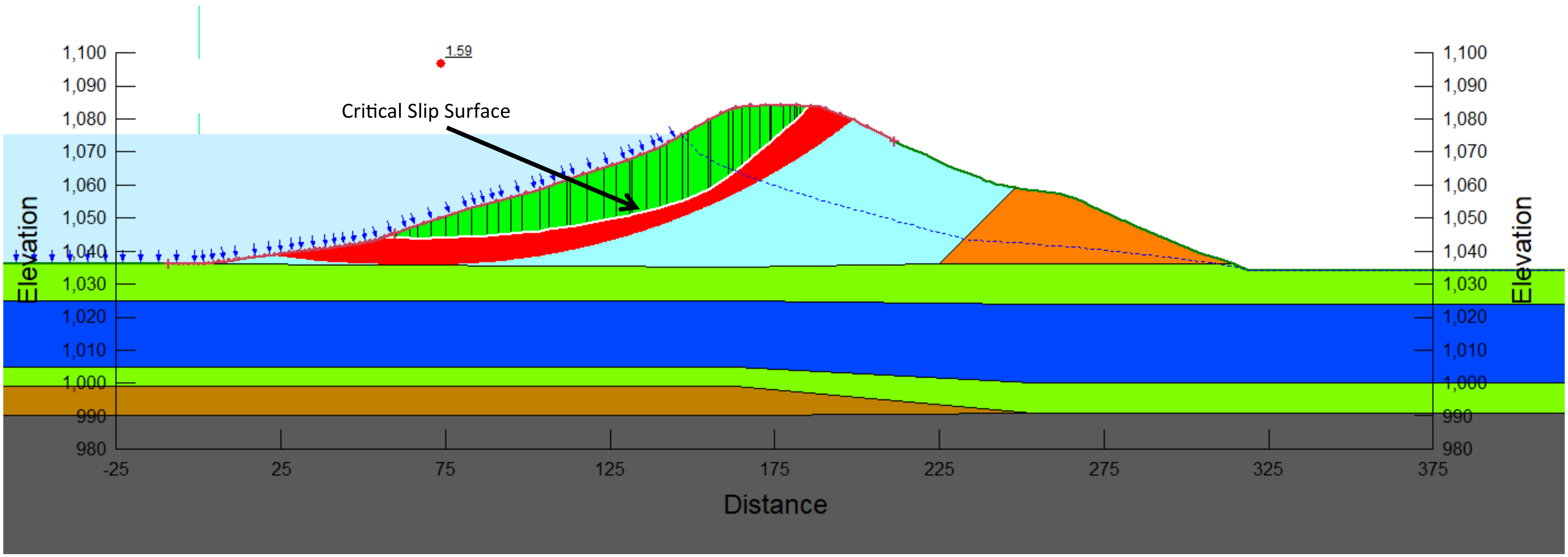

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Silty Sand (SC-SM)	Mohr-Coulomb	120	0	27	0	0	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Residual - Silty Sand (SM)	Mohr-Coulomb	130	0	33	0	0	0
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO-	SWN24427
DATE:	DECEMBER 2024
PREPARED:	IJD

FREESE AND NICHOLS
5100 EAST SKELLY DRIVE
SUITE 602
TULSA, OKLAHOMA 74135

City of Shawnee
Shawnee Twin Lake Dam #1

Seismic Slope Stability Analysis—Effective Strength
Embankment Cross Section (STA: 17+00) - Upstream Slope at Normal Pool

PLATE
A.16

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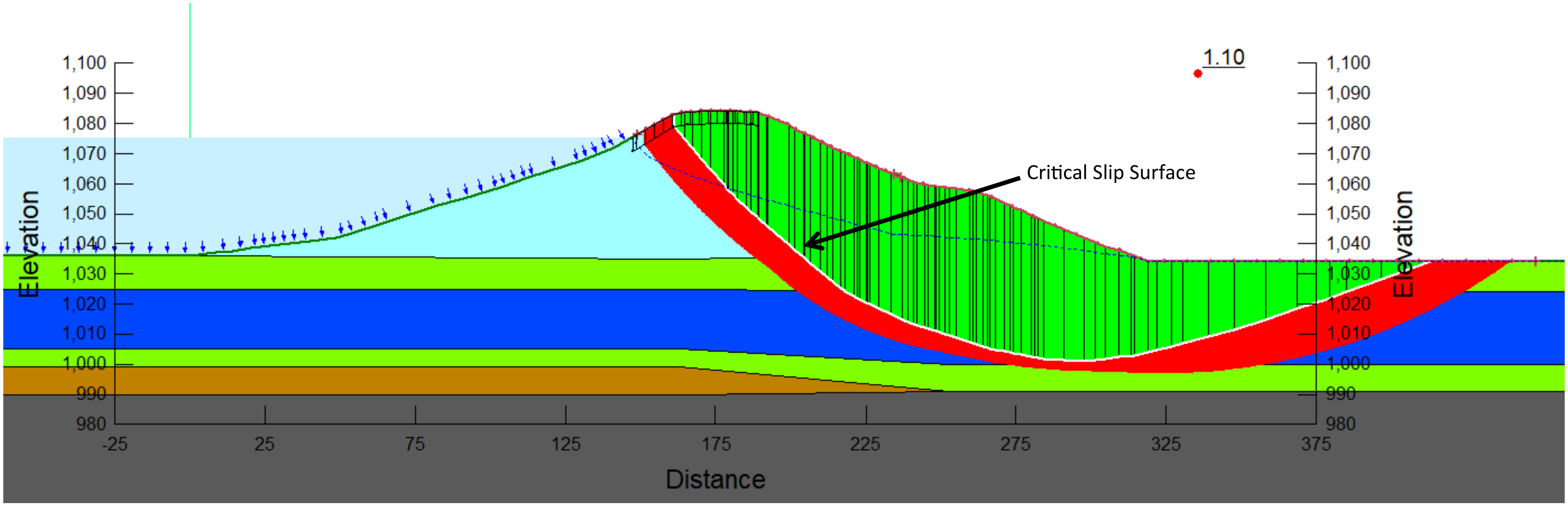

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Silty Sand (SC-SM)	Mohr-Coulomb	120	0	27	0	0	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Residual - Silty Sand (SM)	Mohr-Coulomb	130	0	33	0	0	0
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO-	SWN24427
DATE:	DECEMBER 2024
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FREESE AND NICHOLS
5100 EAST SKELLY DRIVE
SUITE 602
TULSA, OKLAHOMA 74135

City of Shawnee
Shawnee Twin Lake Dam #1

Seismic Slope Stability Analysis—Undrained Strength
Embankment Cross Section (STA: 17+00) - Downstream Slope at Normal Pool

PLATE
A.17

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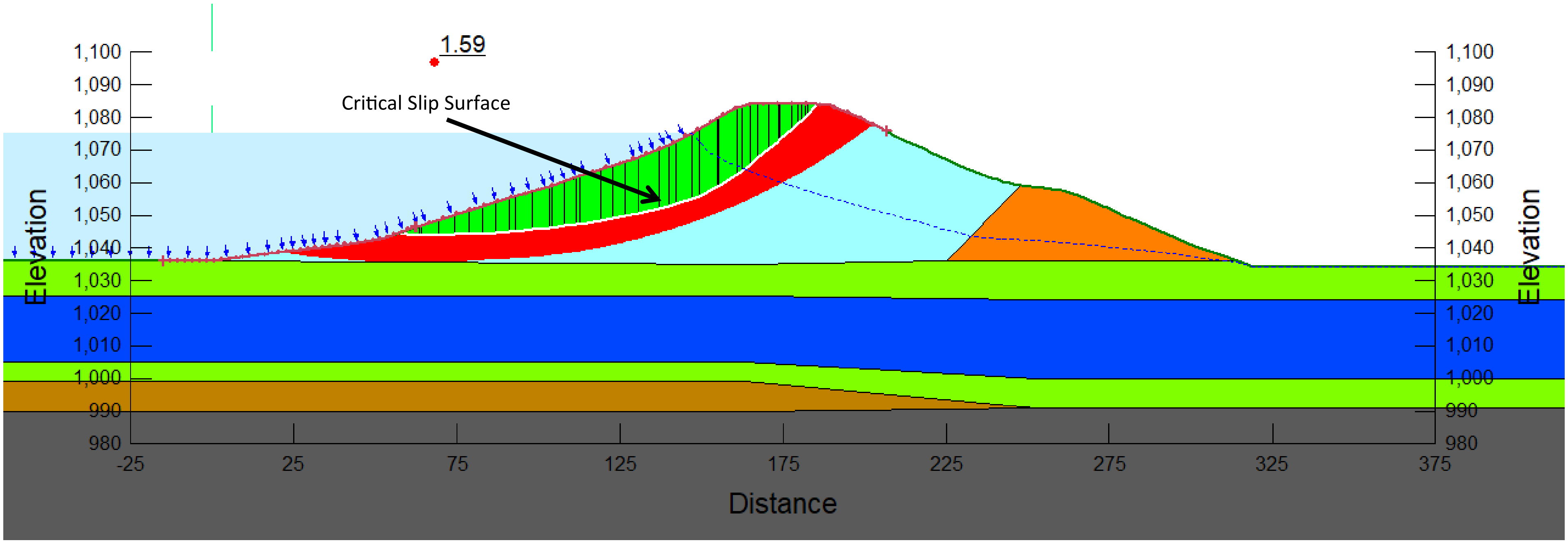

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Fill - Clayey Silty Sand (SC-SM)	Mohr-Coulomb	120	0	27	0	0	0
	Fill - Lean Clay (CL)	Mohr-Coulomb	125	150	25	0	300	17
	Residual - Lean Clay (CL)	Mohr-Coulomb	125	100	27	0	200	18
	Residual - Silty Sand (SM)	Mohr-Coulomb	130	0	33	0	0	0
	Sandstone	Mohr-Coulomb	134	950	24	0	0	0
	Shale	Mohr-Coulomb	130	400	16	0	0	0

FNI PRO-	SWN24427
DATE:	DECEMBER 2024
PREPARED:	IJD

FREESE AND NICHOLS
5100 EAST SKELLY DRIVE
SUITE 602
TULSA, OKLAHOMA 74135

City of Shawnee
Shawnee Twin Lake Dam #1

Seismic Slope Stability Analysis—Undrained Strength
Embankment Cross Section (STA: 17+00) - Upstream Slope at Normal Pool

PLATE
A.18

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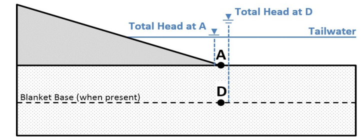
Internal Erosion

INTERNAL EROSION ANALYSIS WORKSHEET

Project Name: Shawnee Twin Lake Dam #1
Project No.: SWN24427
Initials & Date: IJD 11/22/24
Headwater Condition: 1075.2

All elevation units are feet-msl
Analysis depth refered to as Point "D"



DATA ENTRY											RESULTS							
Station/Location	Depth Description	Analysis Type	Toe Surface Elevation	Headwater Elevation	Tailwater Elevation	Saturated Unit Wt., pcf	Elevation at Point "D"	Head at Point "D"	Horiz. Piping Length, ft	Section Flux, cfs (per foot)	Critical Gradient (i_c)	Vert. Exit Gradient (i_{ev})	Effective Safety Factor	Total Safety Factor	Seepage Severity, gpm/ft-H/100'	Seepage Severity Category	Horiz. Exit Gradient (i_{eh})
West Section	1' Below Surface	Heave	1074.8	1075.2		125.0	1073.8	1074.9	87	7.9E-09	**1.00**	**0.03**	**33.44**	**N/A**	**7.39E-12**	**Negligible**	**12.37**
West Section	Intermediate Depth	Heave	1074.8	1075.2		125.0	1071.8	1074.9	87	7.9E-09	**1.00**	**0.02**	**60.19**	**N/A**	**7.39E-12**	**Negligible**	**12.37**
West Section	5' Below Surface	Heave	1074.8	1075.2		125.0	1069.8	1074.9	87	7.9E-09	**1.00**	**0.01**	**83.60**	**N/A**	**7.39E-12**	**Negligible**	**12.37**
West Section + 100 ft	1' Below Surface	Heave	1076.1	1075.2		125.0	1075.1	1074.9	187	8.6E-11	**1.00**	**0.00**	**Stable**	**N/A**	**8.00E-14**	**Negligible**	**5.75**
West Section + 100 ft	Intermediate Depth	Heave	1076.1	1075.2		125.0	1073.1	1074.9	187	8.6E-11	**1.00**	**0.00**	**Stable**	**N/A**	**8.00E-14**	**Negligible**	**5.75**
West Section + 100 ft	5' Below Surface	Heave	1076.1	1075.2		125.0	1071.1	1074.9	187	8.6E-11	**1.00**	**0.00**	**Stable**	**N/A**	**8.00E-14**	**Negligible**	**5.75**
Center Section	1' Below Surface	Heave	1034.3	1075.2		130.0	1033.3	1034.6	319	7.3E-06	**1.08**	**0.34**	**3.16**	**N/A**	**6.79E-09**	**Negligible**	**3.37**
Center Section	Intermediate Depth	Heave	1034.3	1075.2		130.0	1031.3	1034.9	319	7.3E-06	**1.08**	**0.22**	**4.98**	**N/A**	**6.79E-09**	**Negligible**	**3.37**
Center Section	5' Below Surface	Heave	1034.3	1075.2		130.0	1029.3	1035.1	319	7.3E-06	**1.08**	**0.17**	**6.50**	**N/A**	**6.79E-09**	**Negligible**	**3.37**
Center Section + 100 ft	1' Below Surface	Heave	1034.3	1075.2		130.0	1033.3	1034.3	419	4.4E-07	**1.08**	**0.00**	**Stable**	**N/A**	**4.05E-10**	**Negligible**	**2.57**
Center Section + 100 ft	Intermediate Depth	Heave	1034.3	1075.2		130.0	1031.3	1034.3	419	4.4E-07	**1.08**	**0.00**	**1083.33**	**N/A**	**4.05E-10**	**Negligible**	**2.57**
Center Section + 100 ft	5' Below Surface	Heave	1034.3	1075.2		130.0	1029.3	1034.3	419	4.4E-07	**1.08**	**0.00**	**773.81**	**N/A**	**4.05E-10**	**Negligible**	**2.57**

Liquefaction

Liquifiable soils are generally shallow deposits of loose, saturated, cohesionless soils with little fines deposited in the Holocene era or more recently. Some low plasticity, fine grained soil deposits may be susceptible to liquefaction during intense seismic shaking when their natural moisture content is near the liquid limit (Bray and Sancio, 2006). Their criteria indicated that fine-grained soils are potentially liquefiable if one of the following criteria are met:

- The Plasticity Index (PI) is less than 18 indicates susceptibility, or
- The PI is less than 12, and the moisture content to liquid limit (LL) ratio of greater than 0.85 indicates susceptibility or
- The PI is between 12 and 18 and moisture content to liquid limit (LL) ratio is greater than 0.80 indicates moderate susceptibility

Review of the soil samples obtained from the borings located within the footprint of the dam indicated that the soils encountered at Shawnee City Lake No. 1 primarily consisted of sandy clays and clayey sands. These soils generally had a PI less than 18, but higher than 12 with a water content less than 85% of the LL, indicating potentially non-liquefiable soils based on the criteria provided above. A graphical summary of the Atterberg limit determination tests for all samples are shown in Figure 1.

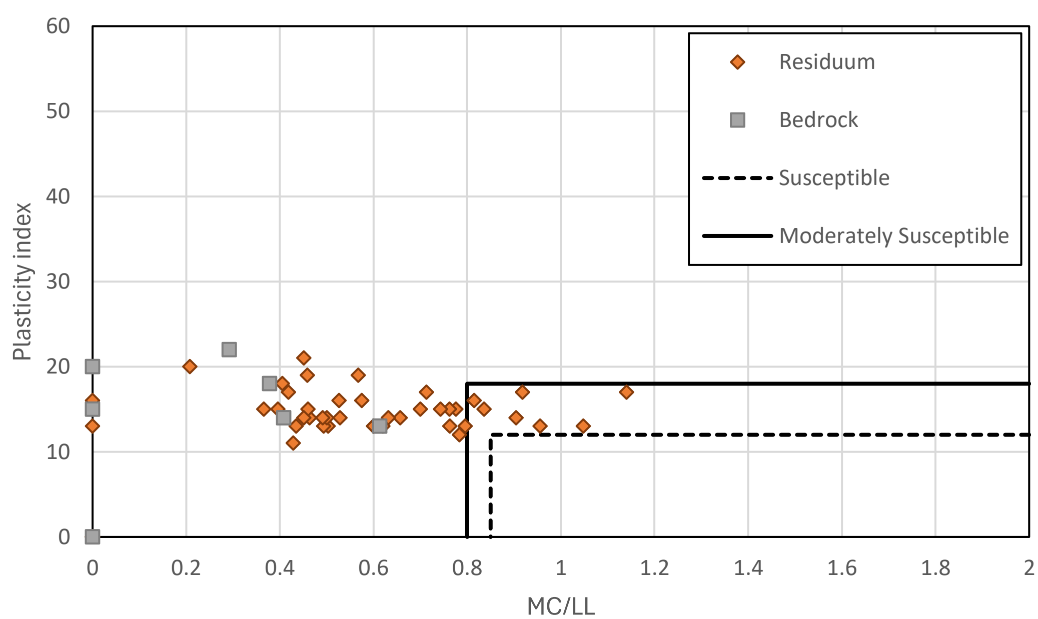


Figure 1. Potentially Liquefiable Cohesive Soils

Although some soils with PIs lower than 18 were encountered within the embankment, these soils are characterized as not susceptible to liquefaction in accordance with the liquefaction criteria provided above, except for two samples in Boring 14. As shown in Figure 1, these parameters fall into the category that is moderately susceptible. Table 1 presents the potentially liquefiable soils from Boring 14, as well as the soils that have a MC/LL ratio of above 0.8.

Table 1. Summary of Low Plasticity Cohesive Soils

Borehole No.	Sample Depth (ft)	USCS	Material	Liquid Limit	Plasticity Index	Moisture Content (%)
6	78.5	SM	Silty Sand	20	17	22.8
10	53.5	SC-SM	Clayey Silty Sand	20	16	16.3
12	28.5	CL	Lean Clay	25	15	20.9
14	13.5	CL	Lean Clay	23	14	20.8
14	28.5	Cl	Lean Clay	21	13	22.0
14	53.5	SC-SM	Clayey Silty Sand	22	17	20.2
16	33.5	SC-SM	Clayey Silty Sand	18	13	17.2

While the soils observed within the dam footprint consisted primarily of clays of varying sand content, several test samples classified as silty sand (SM), clayey sand (SC), clayey silty sand (SC-SM), and poorly graded sand (SP) of varying clay content were observed. Based on the boring logs, 14 field standard penetration test results (SPT) specimens were observed as being potentially liquefiable based on the SPT-N blow counts and PI values. Cohesionless deposits with blow counts less than 15 blows per foot encountered during the geotechnical investigation could be marginally liquefiable from earthquake shaking, including sections occurring beneath the dam embankment and were considered potentially liquefiable and included in the analysis. The soil specimens included in the liquefaction potential analyses are summarized in Table 2.

Table 2. Cohesionless Soils Screened for Liquefaction Potential

Boring No.	Soil Type (USCS)	Material	Depth Below Existing Ground (ft)	SPT Value (uncorrected)	Density[1]
8	SM	Silty Sand	48.5	9	Loose
8	SM	Silty Sand	53.5	8	Loose
11	SC-SM	Clayey Silty Sand	23.5	5	Loose
12	SC-SM	Clayey Silty Sand	13.5	9	Loose
12	SC-SM	Clayey Silty Sand	23.5	7	Loose
14	SP	Poorly Graded Sand	18.5	9	Loose
14	SP	Poorly Graded Sand	23.5	7	Loose
14	SC-SM	Clayey Silty Sand	53.5	11	Medium Dense
15	SM	Silty Sand	18.5	10	Loose
15	SM	Silty Sand	28.5	9	Loose
16	SM	Silty Sand	18.5	3	Very Loose
16	SM	Silty Sand	28.5	7	Loose
16	SC-SM	Clayey Silty Sand	33.5	15	Medium Dense

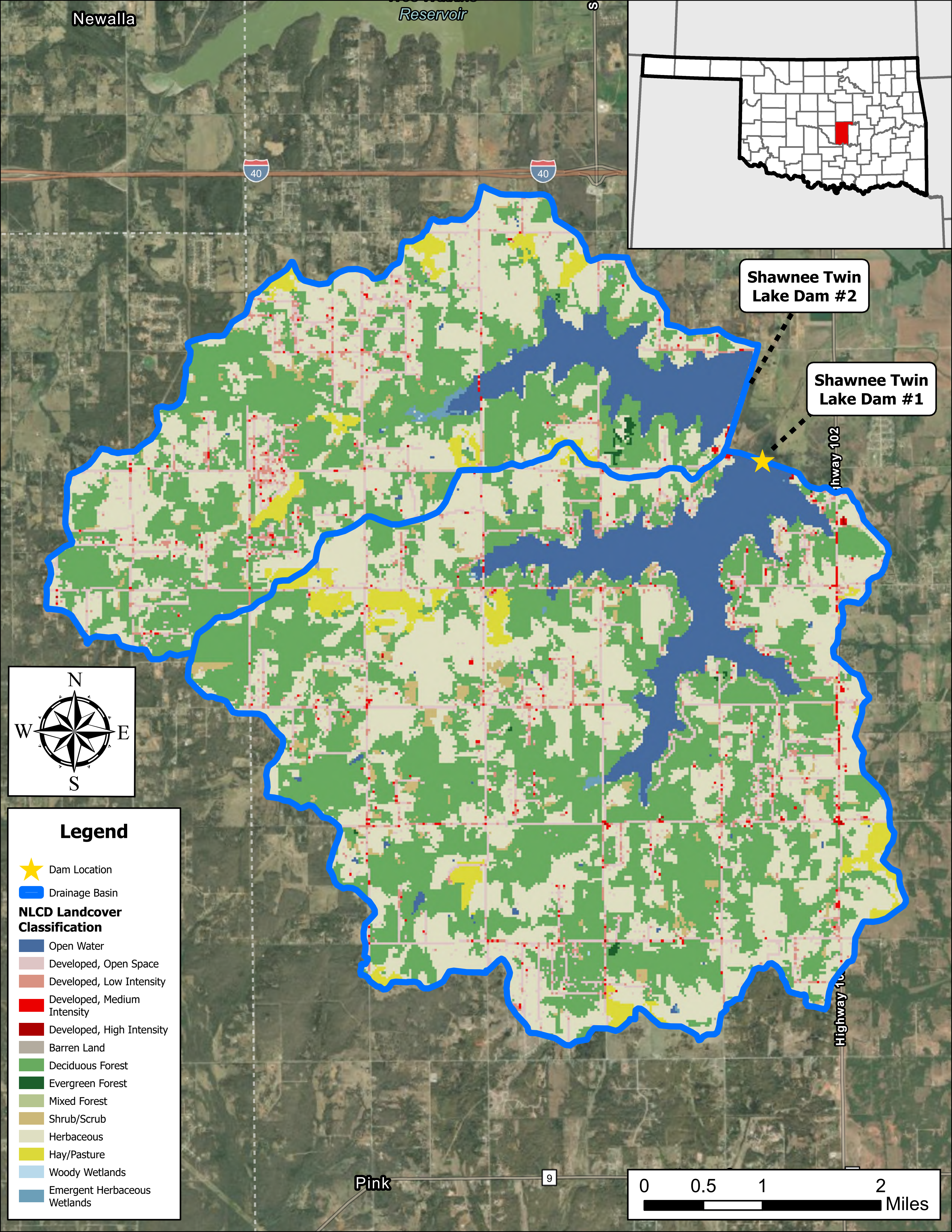
Boring No.	Soil Type (USCS)	Material	Depth Below Existing Ground (ft)	SPT Value (uncorrected)	Density[1]
16	SC-SM	Clayey Silty Sand	38.5	9	Loose
16	SC-SM	Clayey Silty Sand	43.5	10	Loose

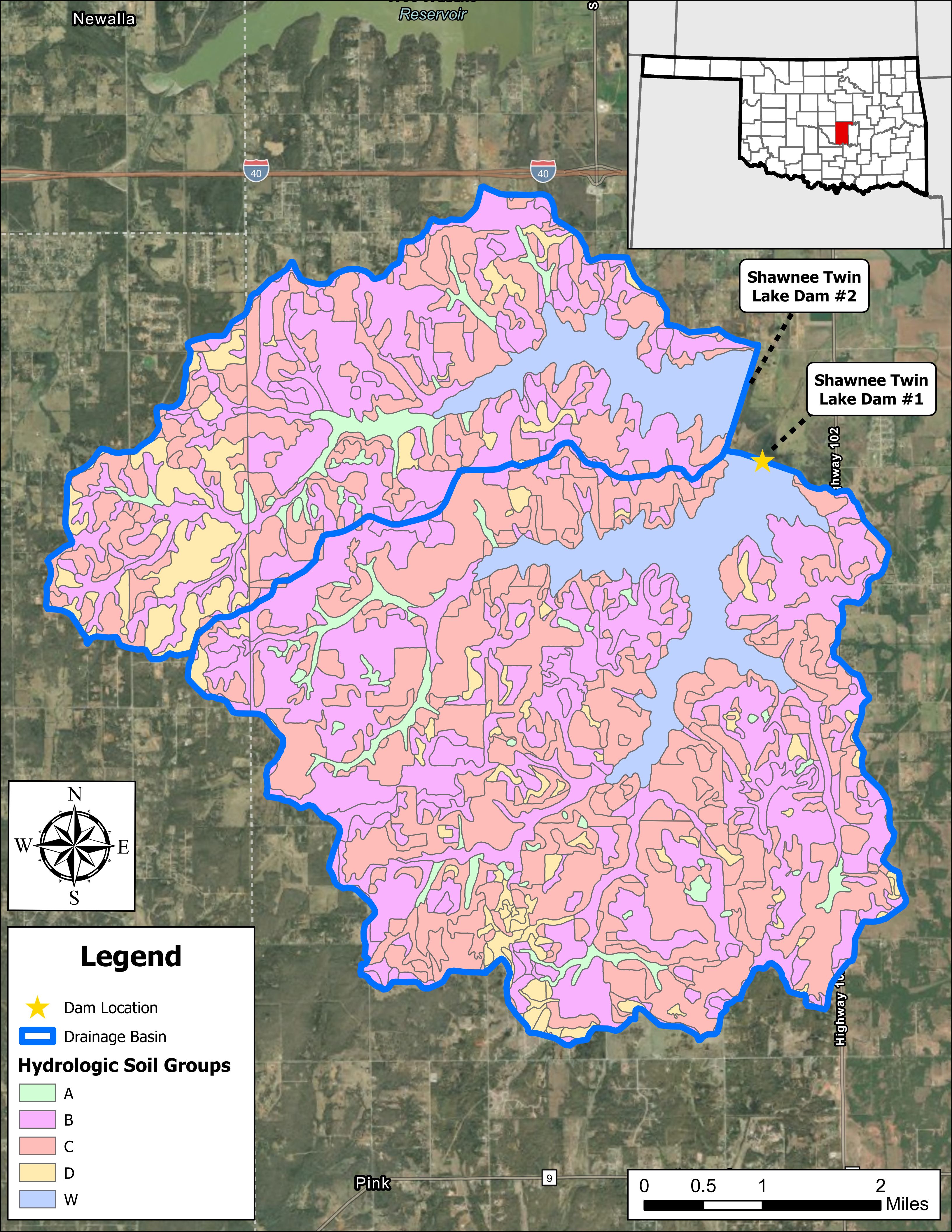
[1] Based on Table 52-2 from NEH 628, Chapter 52

Based on the USGS Unified Hazard Tool, a B/C boundary peak ground acceleration (PGA) of 0.3162g was obtained for recurrence of once in 10,000 years. Additionally, an earthquake moment magnitude (M_w) value of 5.7 was utilized in calculations based on the mean earthquake magnitude (over all sources) from the USGS Unified Hazard Deaggregation Tool.

FNI did not perform liquefaction triggering analyses using available in-situ testing results on-site. Based on the relative density and index testing, some sand layers are potentially liquefiable during the design earthquake. At minimum, FNI recommends that a liquefaction triggering analysis using the Simplified Method, or more robust methods be performed. If continuous zones of liquefaction are computed, a post-liquefaction slope stability analysis and/or deformation analysis should be considered.

Appendix C. H&H Calculations and Modeling


Newalla
Reservoir
40
40
Shawnee Twin Lake Dam #2
Shawnee Twin Lake Dam #1
Highway 102
Highway 102
Pink
9
N
W
E
S
Legend
Dam Location
Drainage Basin
NLCD Landcover Classification
Open Water
Developed, Open Space
Developed, Low Intensity
Developed, Medium Intensity
Developed, High Intensity
Barren Land
Deciduous Forest
Evergreen Forest
Mixed Forest
Shrub/Scrub
Herbaceous
Hay/Pasture
Woody Wetlands
Emergent Herbaceous Wetlands
0 0.5 1 2
Miles



Newalla
Reservoir
Shawnee Twin Lake Dam #2
Shawnee Twin Lake Dam #1
Highway 102
40
40
N
S
E
W
Legend
Dam Location
Drainage Basin
Hydrologic Soil Groups
A
B
C
D
W
Pink
9
0
0.5
1
2
Miles

CURVE NUMBER DATA

Project	Shawnee Lake Dam	Location	Shawnee, OK
Engineer	Taylor Green	Date	10/25/2024
Check	Kyle Jacobs	Date	10/25/2024
QC	Jennifer Gaines	Date	11/18/2024

Curve Number Summary Table

Basin	Area (ac)	Area (mi2)	ARC I CN	ARC II CN	ARC III CN
Shawnee#1	13605.92	21.26	52	72	86
Shawnee#2	7338.53	11.47	52	72	86

Curve Number Lookup Table

NLCD	NLCD	TR-55	Curve Number							
Grid Code	Classification	Classification	A	A/D	B	B/D	C	C/D	D	W
11	Open Water	Water	100	100	100	100	100	100	100	100
21	Developed, Open Space	Streets and Roads - Paved; open ditch	83	93	89	93	92	93	93	100
22	Developed, Low Intensity	Low Density Residential acre	51	84	68	84	79	84	84	100
23	Developed, Medium Intensity	Medium Density Residential quarter	61	87	75	87	83	87	87	100
24	Developed, High Intensity	High Density Residential	77	92	85	92	90	92	92	100
31	Barren Land	Fallow - Bare	77	94	86	94	91	94	94	100
41	Deciduous Forest	Woods - Good	30	77	55	77	70	77	77	100
42	Evergreen Forest	Woods - Fair	36	79	60	79	73	79	79	100
43	Mixed Forest	Woods - Good	30	77	55	77	70	77	77	100
52	Shrub/Scrub	Brush - Good	30	73	48	73	65	73	73	100
71	Herbaceuous	Pasture - Fair	49	84	69	84	79	84	84	100
81	Hay/Pasture	Pasture - Good	39	80	61	80	74	80	80	100
82	Cultivated Crops	Row Crops SR - Good	67	89	78	89	85	89	89	100
90	Woody Wetlands	Woods - Poor	45	83	66	83	77	83	83	100
95	Emergent Herbaceuous Wetlands	Pasture - Poor	68	89	79	89	86	89	89	100

Subbasin Soil and Land Use Data

Basin	Soil Group	NLCD Grid Code	Area (acres)	Polygon CN	Area-Weighted CN
Shawnee#1	A	11	6.91	100	0.05
Shawnee#1	B	11	32.34	100	0.24
Shawnee#1	C	11	46.29	100	0.34
Shawnee#1	D	11	5.24	100	0.04
Shawnee#1	W	11	1191.22	100	8.76
Shawnee#1	A	21	25.28	39	0.07
Shawnee#1	B	21	440.56	61	1.98
Shawnee#1	C	21	427.05	74	2.32
Shawnee#1	D	21	53.92	80	0.32
Shawnee#1	W	21	3.75	100	0.03
Shawnee#1	A	22	3.26	61	0.01
Shawnee#1	B	22	130.36	75	0.72
Shawnee#1	C	22	128.56	83	0.78
Shawnee#1	D	22	12.27	87	0.08
Shawnee#1	W	22	1.63	100	0.01
Shawnee#1	A	23	0.75	77	0.00
Shawnee#1	B	23	35.94	85	0.22
Shawnee#1	C	23	35.17	90	0.23
Shawnee#1	D	23	4.18	92	0.03
Shawnee#1	W	23	4.34	100	0.03
Shawnee#1	B	24	2.23	82	0.01
Shawnee#1	C	24	2.63	94	0.02
Shawnee#1	D	24	0.34	95	0.00
Shawnee#1	W	24	0.59	100	0.00
Shawnee#1	B	31	2.58	86	0.02
Shawnee#1	C	31	0.74	91	0.00
Shawnee#1	W	31	1.82	100	0.01
Shawnee#1	A	41	218.07	30	0.48
Shawnee#1	B	41	2521.70	55	10.19
Shawnee#1	C	41	2508.06	70	12.90
Shawnee#1	D	41	219.44	77	1.24
Shawnee#1	W	41	19.86	100	0.15
Shawnee#1	A	42	0.57	36	0.00
Shawnee#1	B	42	0.04	60	0.00
Shawnee#1	C	42	6.93	73	0.04

Basin	Soil Group	NLCD Grid Code	Area (acres)	Polygon CN	Area-Weighted CN
Shawnee#1	B	43	3.29	55	0.01
Shawnee#1	C	43	7.16	70	0.04
Shawnee#1	W	43	0.13	100	0.00
Shawnee#1	A	52	12.69	30	0.03
Shawnee#1	B	52	99.54	48	0.35
Shawnee#1	C	52	137.01	65	0.65
Shawnee#1	D	52	15.48	73	0.08
Shawnee#1	W	52	0.97	100	0.01
Shawnee#1	A	71	112.18	49	0.40
Shawnee#1	B	71	1768.61	69	8.97
Shawnee#1	C	71	2657.14	79	15.43
Shawnee#1	D	71	256.71	84	1.58
Shawnee#1	W	71	27.39	100	0.20
Shawnee#1	A	81	48.97	39	0.14
Shawnee#1	B	81	197.78	61	0.89
Shawnee#1	C	81	127.65	74	0.69
Shawnee#1	D	81	21.52	80	0.13
Shawnee#1	A	90	0.44	86	0.00
Shawnee#1	B	90	0.57	86	0.00
Shawnee#1	W	90	0.10	100	0.00
Shawnee#1	B	95	10.10	80	0.06
Shawnee#1	C	95	1.38	80	0.01
Shawnee#1	D	95	2.48	80	0.01
Shawnee#1	W	95	2.01	100	0.01
Shawnee#2	A	11	1.09	100	0.01
Shawnee#2	B	11	18.88	100	0.26
Shawnee#2	C	11	25.61	100	0.35
Shawnee#2	D	11	1.31	100	0.02
Shawnee#2	W	11	679.50	100	9.26
Shawnee#2	A	21	18.97	39	0.10
Shawnee#2	W	23	2.79	100	0.04
Shawnee#2	B	24	0.22	82	0.00
Shawnee#2	C	24	0.35	94	0.00
Shawnee#2	D	24	0.27	95	0.00
Shawnee#2	W	24	0.66	100	0.01
Shawnee#2	C	31	0.10	91	0.00
Shawnee#2	W	31	0.12	100	0.00
Shawnee#2	A	41	137.22	30	0.56
Shawnee#2	B	41	1449.77	55	10.87
Shawnee#2	C	41	853.73	70	8.14
Shawnee#2	D	41	456.75	77	4.79
Shawnee#2	W	41	7.05	100	0.10
Shawnee#2	B	42	7.20	60	0.06
Shawnee#2	C	42	8.77	73	0.09
Shawnee#2	A	43	0.81	30	0.00
Shawnee#2	B	43	5.27	55	0.04
Shawnee#2	C	43	3.87	70	0.04
Shawnee#2	D	43	0.89	77	0.01
Shawnee#2	W	43	0.10	100	0.00
Shawnee#2	A	52	9.22	30	0.04
Shawnee#2	B	52	57.78	48	0.38
Shawnee#2	C	52	41.57	65	0.37
Shawnee#2	D	52	12.27	73	0.12
Shawnee#2	A	71	100.37	49	0.67
Shawnee#2	B	71	986.56	69	9.28
Shawnee#2	C	71	1221.48	79	13.15
Shawnee#2	D	71	260.40	84	2.98
Shawnee#2	W	71	9.89	100	0.13
Shawnee#2	A	81	33.10	39	0.18
Shawnee#2	B	81	122.08	61	1.01
Shawnee#2	C	81	77.83	74	0.78
Shawnee#2	D	81	1.06	80	0.01
Shawnee#2	A	90	1.33	86	0.02
Shawnee#2	C	90	0.08	86	0.00
Shawnee#2	W	90	0.14	100	0.00
Shawnee#2	A	95	6.95	80	0.08
Shawnee#2	B	95	8.45	80	0.09
Shawnee#2	C	95	1.62	80	0.02
Shawnee#2	D	95	0.02	80	0.00
Shawnee#2	W	95	12.01	100	0.16

Project	Shawnee Lake Dam	Location	Shawnee, OK
Engineer	Taylor Green	Date	10/25/2024
Check	Kyle Jacobs	Date	10/25/2024
QC	Jennifer Gaines	Date	11/18/2024

Time of Concentration
NRCS Lag equation (Shawnee#1)
Watershed A

	ARCII	
Flow Length	15,762	Feet
Slope	5.4	%
CN	72	
Lag	1.57	Hours
Lag	94.5	Minutes
Time of Conc.	2.62	Hours
Time of Conc.	157.5	Minutes
Velocity	2.8	Feet/Sec

630.1502 Methods for estimating time of concentration

Two primary methods of computing time of concentration were developed by the Natural Resources Conservation Service (NRCS) (formerly the Soil Conservation Service (SCS)).

(a) Watershed lag method

The SCS method for watershed lag was developed by Mockus in 1961. It spans a broad set of conditions ranging from heavily forested watersheds with steep channels and a high percent of runoff resulting from subsurface flow, to meadows providing a high retardance to surface runoff, to smooth land surfaces and large paved areas.

$$L = \frac{\ell^{0.8}(S+1)^{0.7}}{1{,}900\,Y^{0.5}} \qquad \text{(eq. 15–4a)}$$

Applying equation 15–3, $L=0.6T_c$, yields:

$$T_c = \frac{\ell^{0.8}(S+1)^{0.7}}{1{,}140\,Y^{0.5}} \qquad \text{(eq. 15–4b)}$$

where:
L = lag, h
T_c = time of concentration, h
ℓ = flow length, ft
Y = average watershed land slope, %
S = maximum potential retention, in

$$= \frac{1{,}000}{cn'} - 10$$

where:
cn' = the retardance factor

Flow length (ℓ)—In the watershed lag method of computing time of concentration, flow length is defined as the longest path along which water flows from the watershed divide to the outlet. In developing the regression equation for the lag method, the longest flow path was used to represent the hydraulically most distant point in the watershed. Flow length can be measured using aerial photographs, quadrangle sheets, or GIS techniques. Mockus (USDA 1973) developed an empirical relationship between flow length and drainage area using data from Agricultural Research Service (ARS) watersheds. This relationship is:

$$\ell = 209\,A^{0.6} \qquad \text{(eq. 15–5)}$$

where:
ℓ = flow length, ft
A = drainage area, acres

Land slope (Y), percent—The average land slope of the watershed, as used in the lag method, not to be confused with the slope of the flow path, can be determined in several different ways:

- by assuming land slope is equal to a weighted average of soil map unit slopes, determined using the local soil survey
- by using a clinometer for field measurement to determine an estimated representative average land slope
- by drawing three to four lines on a topographic map perpendicular to the contour lines and determining the average weighted slope of these lines
- by determining the average of the land slope from grid points using a dot counter
- by using the following equation (Chow 1964):

$$Y = \frac{100(CI)}{A} \qquad \text{(eq. 15–6)}$$

where:
Y = average land slope, %
C = summation of the length of the contour lines that pass through the watershed drainage area on the quad sheet, ft
I = contour interval used, ft
A = drainage area, ft^2 (1 acre = 43,560 ft^2)

Retardance factor—The retardance factor, cn′, is a measure of surface conditions relating to the rate at which runoff concentrates at some point of interest. The term “retardance factor” expresses an inverse relationship to “flow retardance.” Low retardance factors are associated with rough surfaces having high degrees of flow retardance, or surfaces over which flow will be impeded. High retardance factors are associated with smooth surfaces having low degrees of flow retardance, or surfaces over which flow moves rapidly.

Project	Shawnee Lake Dam	Location	Shawnee, OK
Engineer	Taylor Green	Date	10/25/2024
Check	Kyle Jacobs	Date	10/25/2024
QC	Jennifer Gaines	Date	11/18/2024

Time of Concentration
NRCS Lag equation (Shawnee#2)
Watershed A

	ARCII	
Flow Length	22,518	Feet
Slope	5.8	%
CN	72	
Lag	2.02	Hours
Lag	121	Minutes
Time of Conc.	3.36	Hours
Time of Conc.	201.6	Minutes
Velocity	3.1	Feet/Sec

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Ogee Crest Discharge Coefficients (USBR)

Design Head (ft)	7.35	P/H	0.41
Spillway Height (ft)	3		
Crest Elevation (ft)	1075.2		
Downstream Apron Elevation (ft)	1074.07		
Upstream Face Slope (H:V)	Vertical		
Actual (net) Crest Length (ft)	296.9		
Number of Piers			
Pier Contraction Coefficient	Square-nosed piers, 0.02		
Abutment Contraction Coefficient	Square Abutments, 0.2		

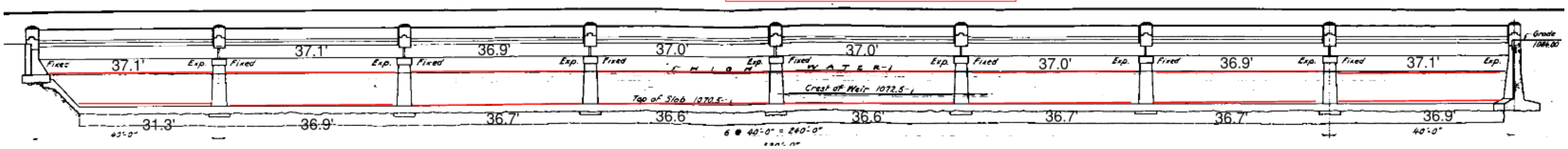
Total Head Elevation (ft)	Total Head (ft)	He/Ho	Eff. Crest Length (ft)	Discharge Coefficient Co	Coef Ratio C/Co	Coef Ratio Ci/Cv	Coef Ratio Apron Cs/Co	Corrected Discharge Coeff C	Discharge (cfs)
1075.2	0.00	0.00	296.90	3.76	Out of limit	1.000	#VALUE!	#VALUE!	#VALUE!
1076	0.80	0.11	296.58	3.76	0.82	1.000	1.00	3.10	658
1077	1.80	0.24	296.18	3.76	0.86	1.000	1.00	3.24	2,319
1078	2.80	0.38	295.78	3.76	0.90	1.000	0.97	3.26	4,511
1079	3.80	0.52	295.38	3.76	0.92	1.000	0.94	3.26	7,127
1080	4.80	0.65	294.98	3.76	0.95	1.000	0.92	3.27	10,129
1081	5.80	0.79	294.58	3.76	0.97	1.000	0.90	3.28	13,483
1082	6.80	0.93	294.18	3.76	0.99	1.000	0.88	3.29	17,178
1083	7.80	1.06	293.78	3.76	1.01	1.000	0.87	3.31	21,166
1084	8.80	1.20	293.38	3.76	1.02	1.000	0.86	3.33	25,480
1085	9.80	1.33	292.98	3.76	1.04	1.000	0.86	3.35	30,098
1086	10.80	1.47	292.58	3.76	1.05	1.000	0.85	3.37	35,009
1087	11.80	1.61	292.18	3.76	Out of limit	1.000	0.85	3.18	37,683
1088	12.80	1.74	291.78	3.76	Out of limit	1.000	0.84	3.16	42,279
1089	13.80	1.88	291.38	3.76	Out of limit	1.000	0.84	3.15	47,040
1090	14.80	2.01	290.98	3.76	Out of limit	1.000	0.83	3.14	51,960
1091	15.80	2.15	290.58	3.76	Out of limit	1.000	0.83	3.13	57,030
1092	16.80	2.29	290.18	3.76	Out of limit	1.000	0.83	3.11	62,239
1093	17.80	2.42	289.78	3.76	Out of limit	1.000	0.83	3.10	67,565
1094	18.80	2.56	289.38	3.76	Out of limit	1.000	0.82	3.10	73,025

	Elev (ft)
Spillway Crest	1075.2
Bottom Chord	1082.55
Opening Height	7.35
Opening Centroid	1078.875

Opening length crest 288.4

*NOTE: Crest of spillway snip based on 1930s design drawings. Crest value listed in calculations based on 2024 survey and were therefore used.

UPSTREAM PILLARS

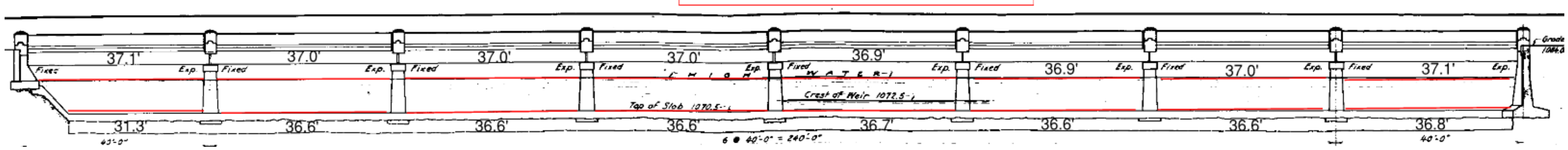


Area	251.37	271.95	270.48	270.48	270.48	270.8475	270.48	271.95

Area Total 2148.037 sf

Opening length crest 287.8

DOWNSTREAM PILLARS



Area	251.37	270.48	270.48	270.48	270.8475	270.8475	270.48	271.5825

Area Total 2146.567 sf

$$\boxed{Q = CoAg\sqrt{2gd}}$$

Where:

Co = 0.60 (or as set in Settings)

Ag = clear opening area in sqft (sqm)

g = 32.16 (9.8) gravity

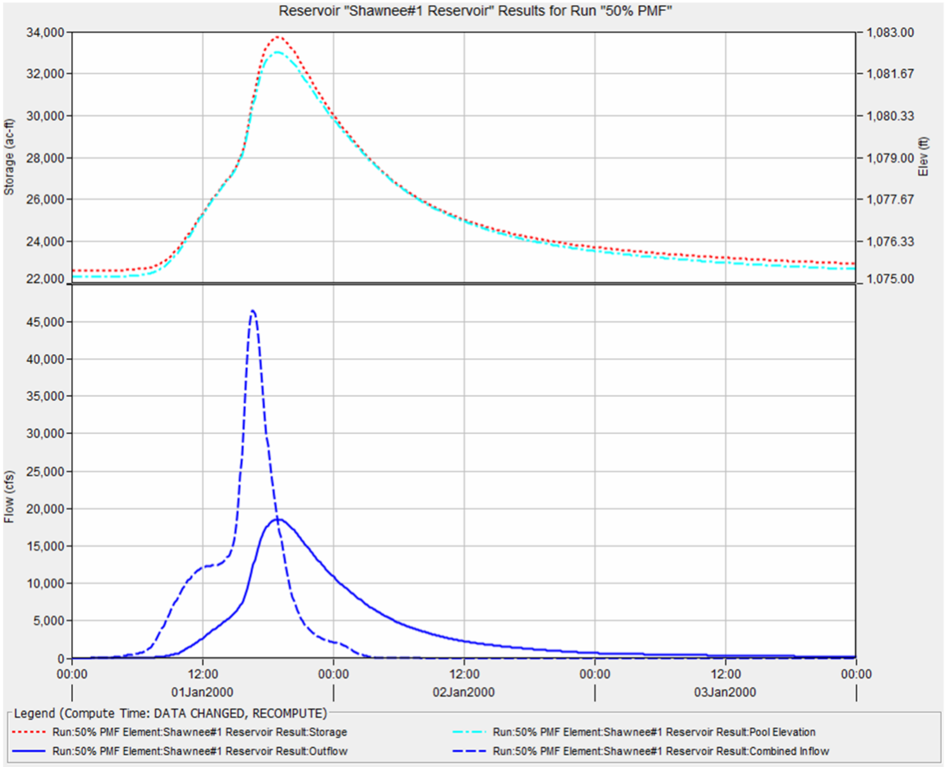
d = head as measured from the centroid in ft (m)

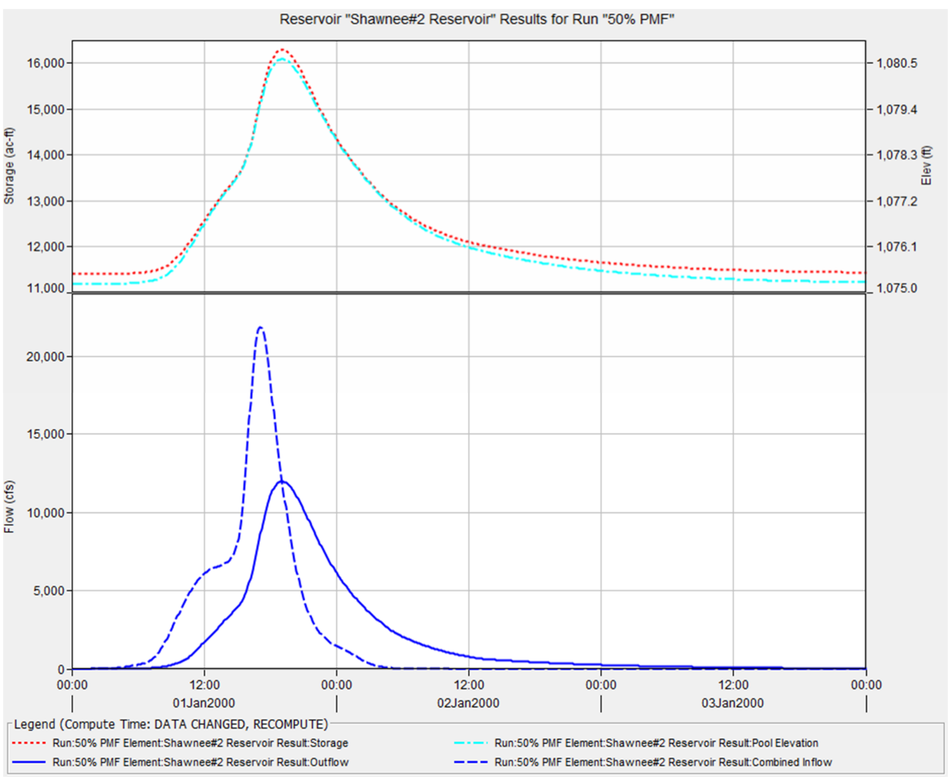
Co	0.6
g	32.16
Centroid	1078.875

Elevation	Discharge (Orifice) (cfs)
1082.55	0
1083	20978.81
1084	23383.84
1085	25563.59
1086	27571.55
1087	29442.89
1088	31202.2
1089	32867.47

50% PMF

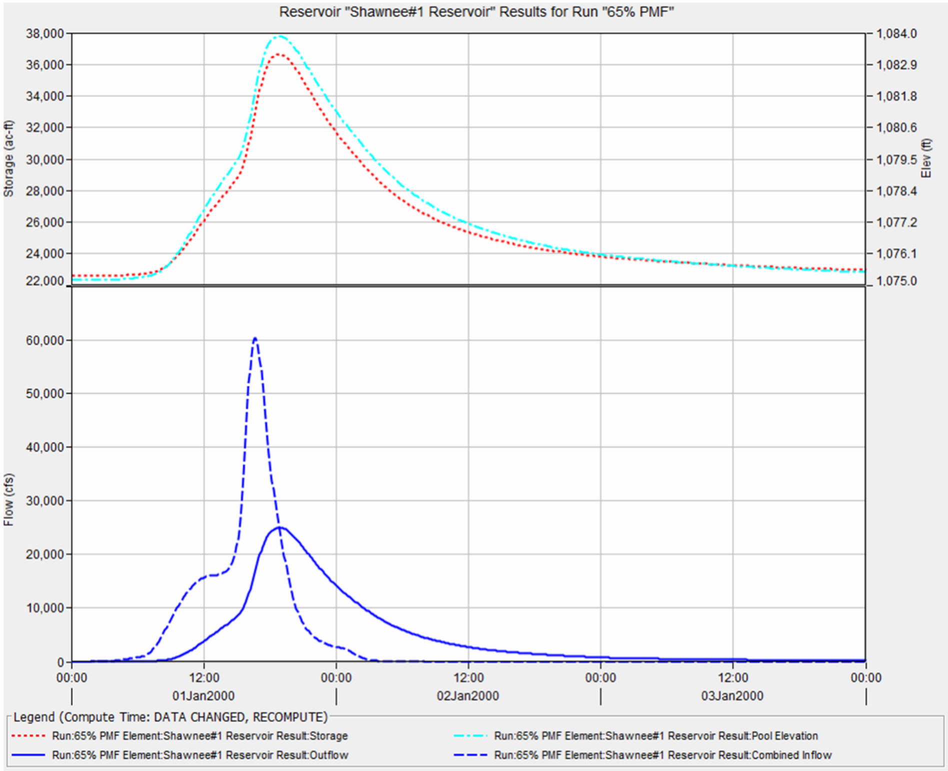
Reservoir Inflow

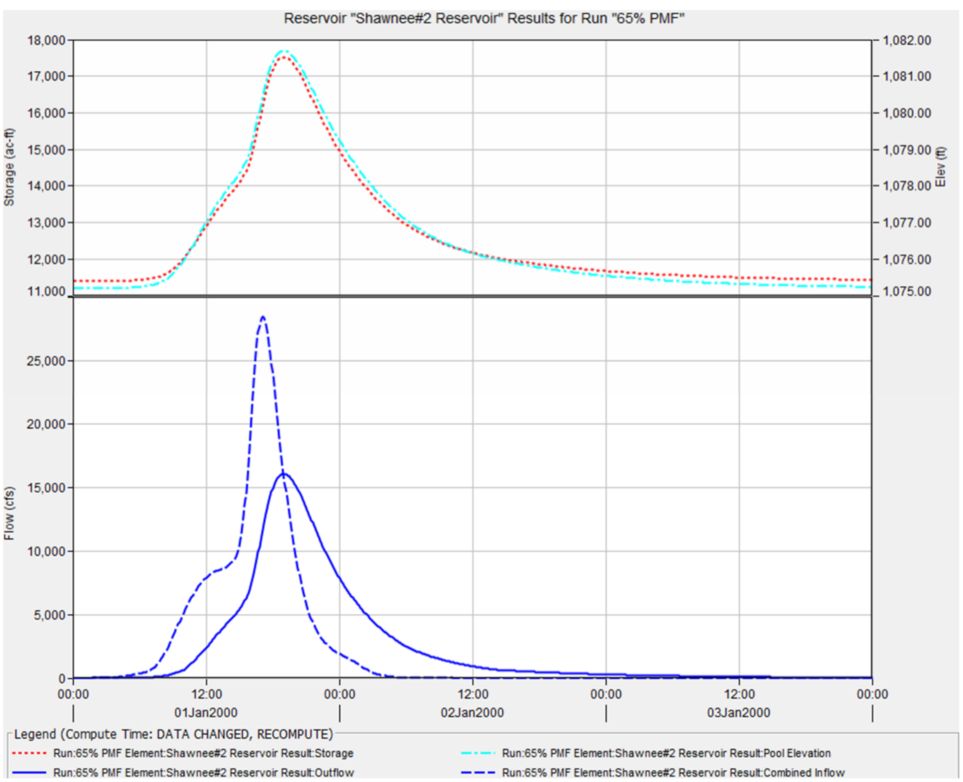




65% PMF

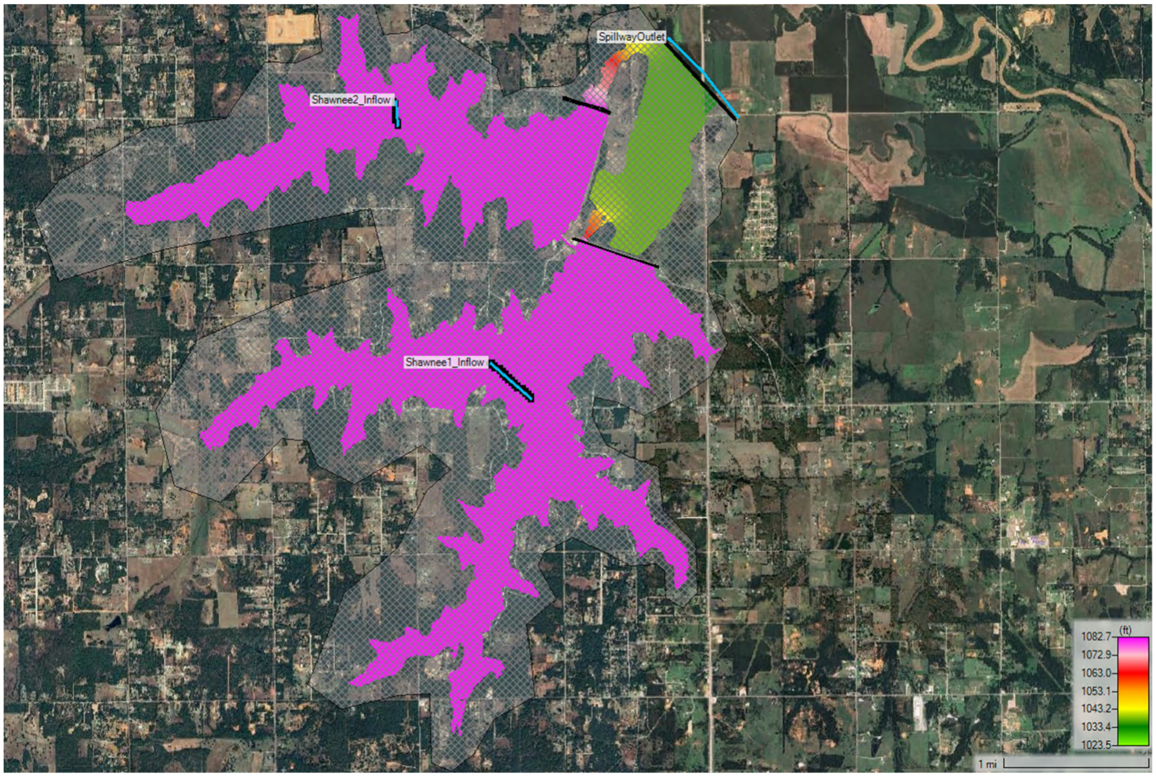
Reservoir Inflow





50% PMF

Hydraulic Mapping



65% PMF

Hydraulic Mapping