

PORTAGE LAKES, OHIO – EAST RESERVOIR DAM: HISTORY, EVALUATION AND REMEDIATION

Pete Nix¹
Donald Bruce²

ABSTRACT

East Reservoir Dam is a high-hazard, sand fill embankment located just south of Akron, Ohio that is owned and operated by the Ohio Department of Natural Resources (ODNR). The dam was constructed in the 1830's as part of the feeder system for the Ohio and Erie Canal system. The dam is approximately 1,300 feet long and has a maximum height of 23 feet. There is significant residential development around the lake and a heavily-traveled road, Portage Lakes Drive, runs along the crest of the embankment.

The embankment was constructed using the natural fine sands at the site and the foundation soils also consist of fine sands and silts. The dam failed once, during the flood of 1913, sending floodwaters into downtown Akron. The embankment was rebuilt in 1916 with a new spillway and outlet works. Unfortunately, the 1916 spillway and outlet works are woefully inadequate for current dam design standards and only pass around 25% of the PMF.

Tetra Tech's challenge on the project was to design an overtopping solution that solved the dam safety issues while minimizing the impacts to the local property owners and Portage Lakes Drive. Due to the overtopping and erodibility of the embankment and foundation, the overtopping solution had to essentially be a three-story structure that could retain 23' of water with no passive resistance. RCC, a new downstream structure, and new overtopping structures constructed in the lake were all considered.

The upstream overtopping structure was determined to be the most economical solution that minimized impacts to adjacent property owners. Large diameter sheet pile cells, capped with concrete, were originally considered for the new structure. However, a more economical solution was developed using innovative deep soil mixing techniques to construct a gravity dam and cutoff wall within a berm placed in the lake. This gravity section was designed using the Corps' Gravity Dam Design requirements and global stability, as well as overturning and sliding, were all considered in the design. This innovative design technique solved the dam safety issues and minimized impacts while providing the most economical solution to ODNR.

I. INTRODUCTION

East Reservoir is part of the Portage Lakes Reservoirs, located just south of Akron, Ohio. In the 1820's canal building was at its peak and the state legislature commissioned a study that ultimately resulted in the construction of several canal systems in Ohio. In

¹ P.E. - Senior Program Manager, Tetra Tech (New Albany, Ohio) Pete.Nix@tetrattech.com

² P.E. - Soil-Cement Consultant, Geosystems (Venetia, Pennsylvania) dabruce@geosystemsbruce.com

1825, work began on the Ohio & Erie Canal, which ran from Portage Summit (Portage Lakes) to Cleveland. In order to supply water to the canal system and to maintain a required four-foot depth, a system of reservoirs was constructed near an area now referred to as Portage Lakes. The system of reservoirs found at Portage Lakes consists of East Reservoir, West Reservoir, North Reservoir, Long Lake, Turkeyfoot Lake, and Nimisila Reservoir. These bodies of water are connected through a system of channels and outlet works. Figure 1 presents a regional map of the area.

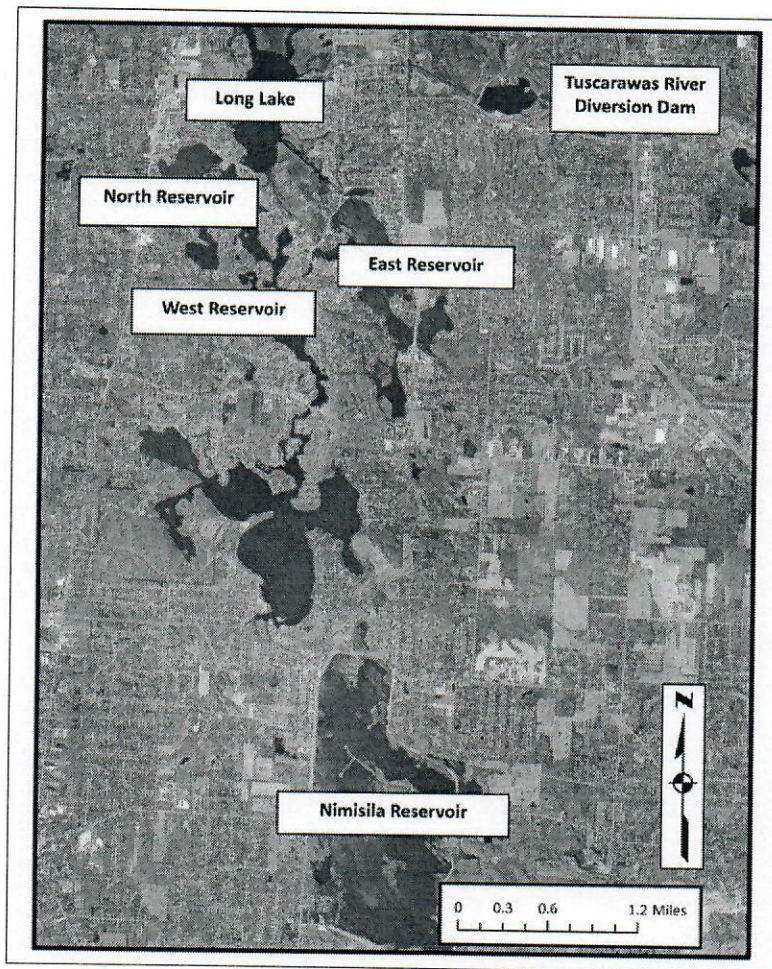


Figure 1. Regional Map (*showing major reservoirs of Portage Lakes*)

The location was ideal for supplying water to the canals since it was the highest point in the canal system, which provided the inspiration for the county name, Summit County. Drainage north of the area flows to Lake Erie while drainage to the south flows to the Ohio River. Also, not by accident, the Portage Lakes name was inspired by the use of this area as a portage years prior to the canal by the Native Americans. This point provided a

relatively short overland portage between navigable waters between the Ohio River and Lake Erie watersheds.

Akron prospered as a result of the increased commerce brought by the canal system. The canal system faced competition from the creation of railroad lines in the 1850s. The use of the canal declined steadily until disaster struck. On March 23, 1913, a severe storm wreaked havoc on several states in the Midwest. Snowmelt in conjunction with heavy rains, sustained over several days, led to reservoir outlet works and dams in the region being overwhelmed. Many of the lock and dam systems and embankment dams in the area failed, causing catastrophic damage to the canal system. The embankment at East Reservoir failed, either by overtopping or piping, causing significant damage in downtown Akron.

Figure 2 shows a photograph of East Reservoir, after the breach in 1913. Although records are sparse, the available information suggests that the portion of the East Reservoir dam that failed and was rebuilt soon thereafter with the addition of the current north spillway in 1915. After the 1913 flood, the canal system was for all intents and purposes abandoned due to the extensive damage to the system. The reservoirs of Portage Lakes remain and are today used primarily for recreational purposes. In 1989, management of the reservoir system was transferred to the Department of Natural Resources.



Figure 2. Photo showing East Reservoir drained after 1913 dam failure
(Photo courtesy of Cuyahoga Falls Library)

II. PROJECT BACKGROUND

East Reservoir Lake has a surface area of 235 acres at normal pool elevation 988.40 (NAVD 88). The pool volume is approximately 6,896 ac-ft at normal pool elevation. The drainage area of the lake is 4.88 square miles (13.99 square miles when combined with West Reservoir) and is located entirely within Summit County. Figure 3 presents a map of the East Reservoir dam and related features.

East Reservoir Dam is comprised of one main embankment and two smaller embankments that are separated from the main embankment by several hundred feet of apparent natural ground. The total length of the embankments is approximately 1,300 feet. The top of the dam is at elevation 992.40 (NAVD 88), which provides a maximum hydraulic height of 25 feet. The downstream slope of each embankment is approximately 2H:1V. The upstream slope of the main embankment is approximately 2.5H:1V, and that of the smaller embankment to the north is 2H:1V. A paved road, Portage Lakes Drive, extends along the entire crest of all three embankments and the crest width varies from approximately 40 to 50 feet. A service spillway is located within each of the smaller embankments. The spillways are referred to as either the north or south spillway, depending on their location with respect to the main embankment.

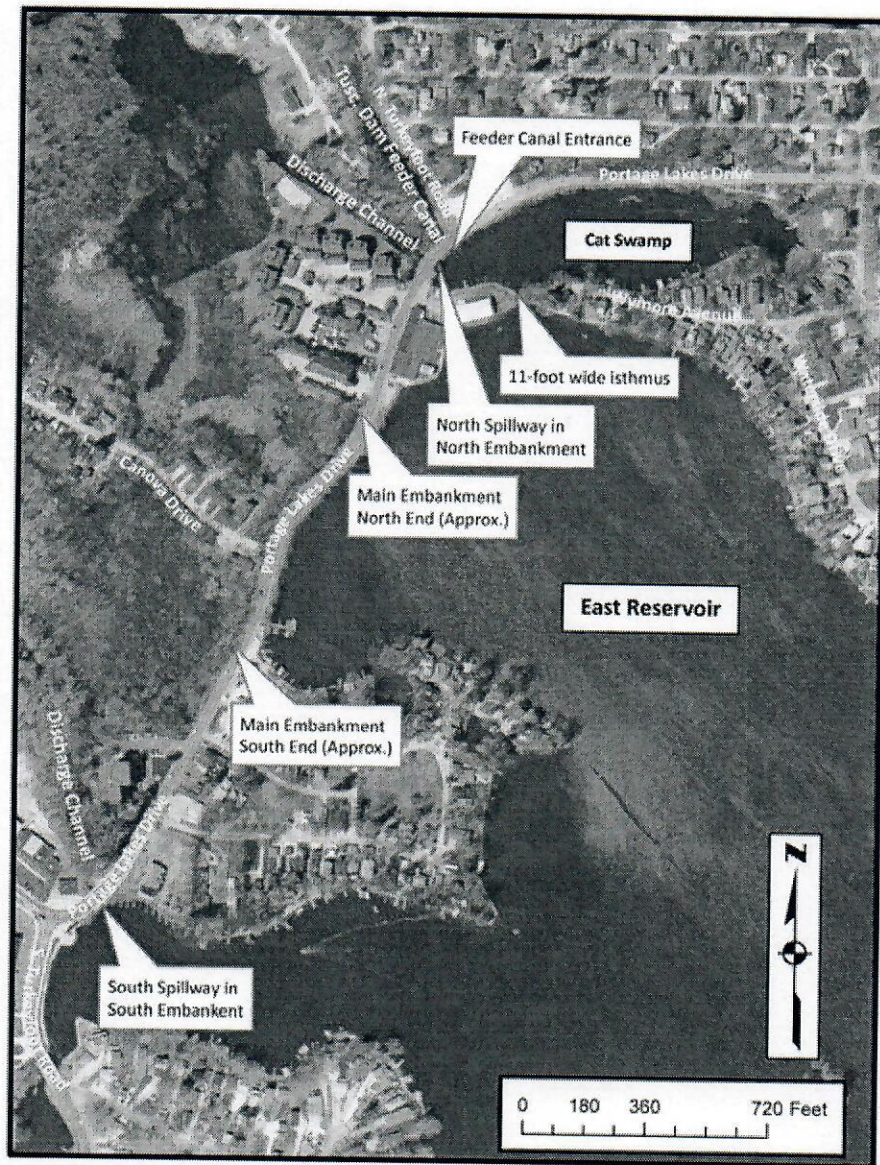


Figure 3. Map Showing East Reservoir Features

The north spillway is a concrete gravity structure with a 20-inch wide and 28-foot long broad-crested weir. A gatehouse is located in front of the spillway, which reduces the effective weir length for higher discharges. Two 30-inch diameter vitrified tiles extend from the gatehouse through the spillway structure. Flow through the tiles is controlled by sluice gates. Both the spillway and the outlet works discharge into a horizontal concrete slab before emptying into a plunge pool type basin in the discharge channel. A single-span bridge carries traffic over the spillway.

The south spillway is a concrete chute spillway with a 12-inch wide and approximately 21-foot long, broad-crested weir. The slope of the discharge chute is 6H:1V. No actual stilling basin exists for the south spillway, as the discharge chute discharges directly into the downstream channel. A single-span bridge (Portage Lakes Drive) extends over the discharge chute. The fixed-crest weirs in both spillways maintain the normal pool level at elevation 988.40 (NAVD 88), which provides approximately 4 feet of freeboard.

Since the dam was originally constructed during the 1820s and 1830s, little is known about the construction details of the dam other than some details regarding the construction of the north spillway in 1915. The subsurface data suggest that the dam embankment is composed primarily of fine-grained sand with silt (SP-SM) and silty sand (SM). The foundation soils profile generally consists of, in descending order; six feet of sand (SP), 12 feet of silt (ML), and 20+ feet of sand with silt (SP-SM). Rock is reportedly up to 150 feet deep in portions of the buried valley where the project site is situated. The general soil profile is shown in Figure 4.

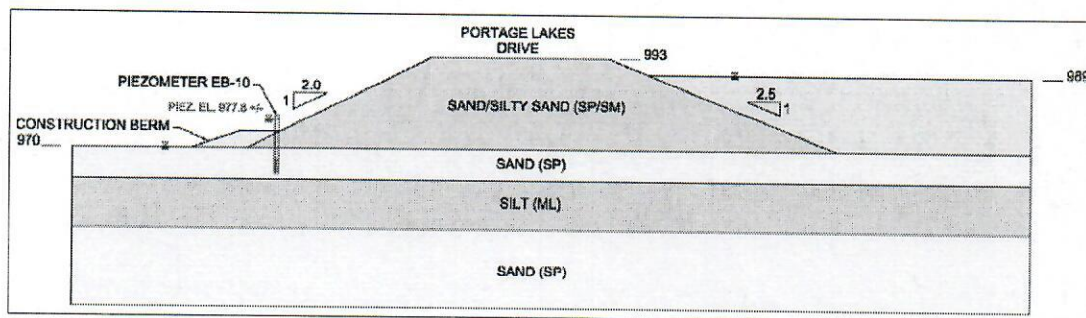


Figure 4. General Soil Profile

III. HYDROLOGIC AND HYDRAULIC EVALUATION AND ANALYSES

The maximum routed water surface elevations for East Reservoir and West Reservoir (connected hydraulically) for the 24-hour and 72-hour PMF model runs, as well as the 100-Year frequency event model run, are presented in Table 1. The values shown in this table represent the maximum worst-case scenarios from the various storm centering and storm duration HEC-1 models from the "Portage Lakes Hydrologic and Hydraulic Technical Report" (Tetra Tech, 2015).

Table 1 – Routing Results for the Critical 12.5% PMF, 100% PMF and 100-Year Frequency Event Model Runs (*Elevations NGVD 29*)

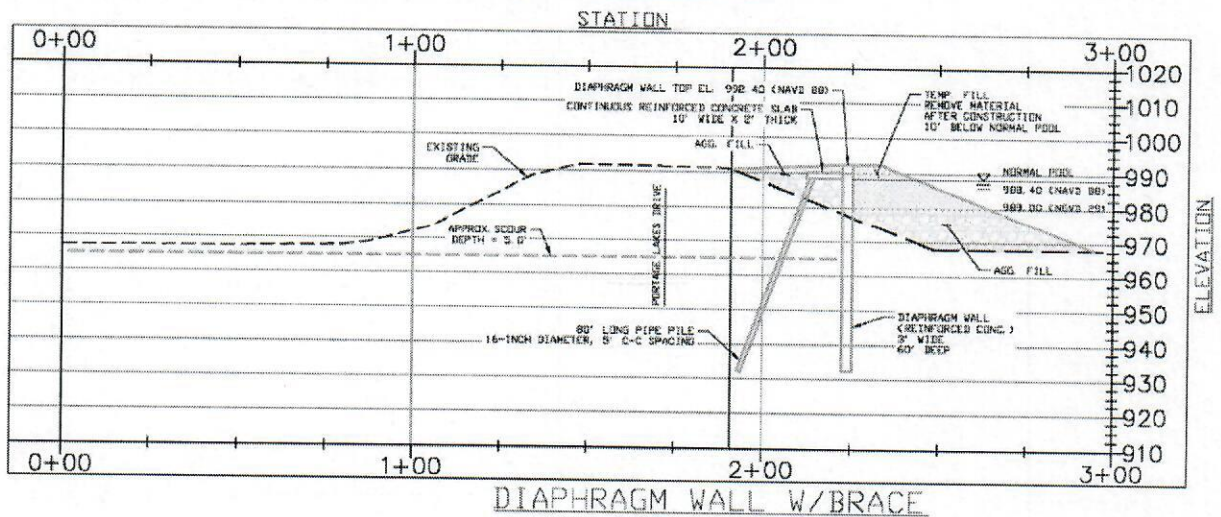
East-West Reservoir						
Ratio of Storm	100-Year		24-hour PMF		72-hour PMF	
	2015 Analysis	Depth Over Dam (ft.)	2015 Analysis	Depth Over Dam (ft.)	2015 Analysis	Depth Over Dam (ft.)
	WSEL		WSEL		WSEL	
0.12	--	--	990.47	0	990.56	0
1	990.52	0	994.63	1.63	994.72	1.72

East Reservoir Dam is overtopped significantly by both, the 24-hour and 72-hour PMF storm events. The analyses also indicated the duration of the overtopping was nearly 8 hours. Consequently, an overtopping solution was required to meet the state's dam safety regulations.

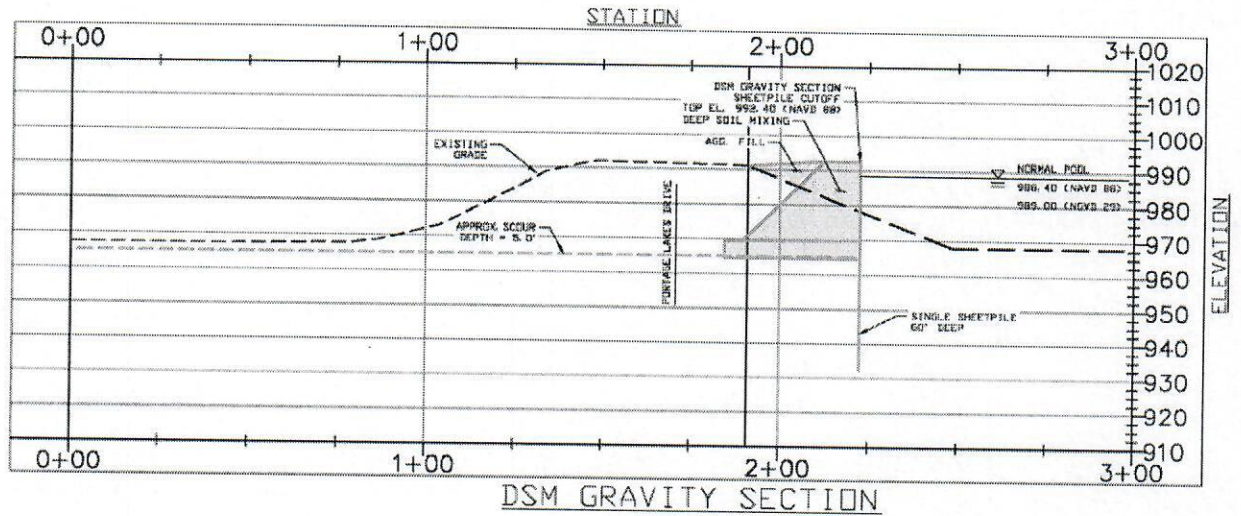
IV. ALTERNATIVES CONSIDERED

The following solutions were considered for final design:

Diaphragm Wall with Brace or Double Sheetpile Wall with Brace

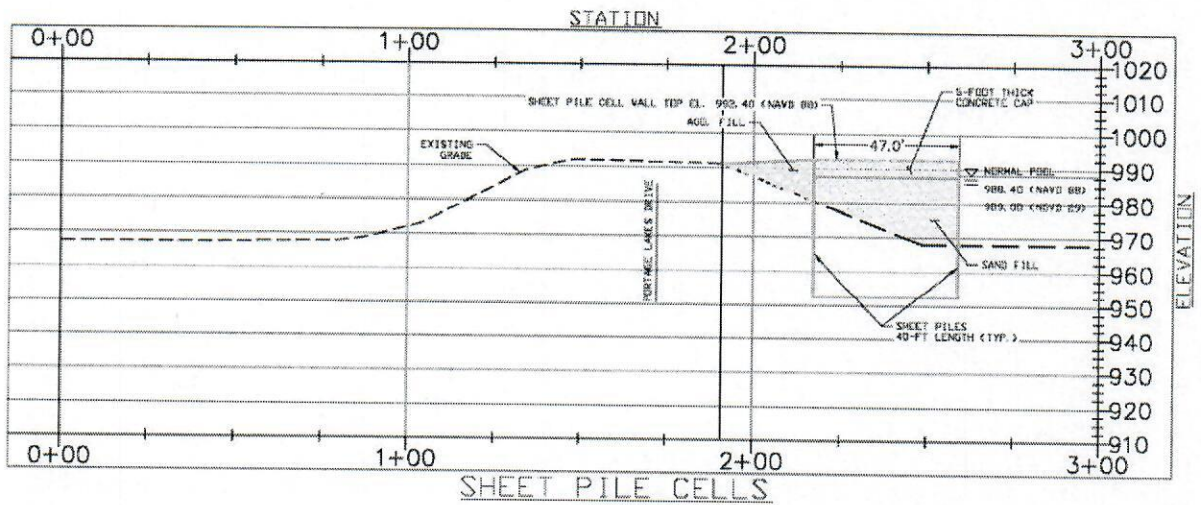


DSM Gravity Section

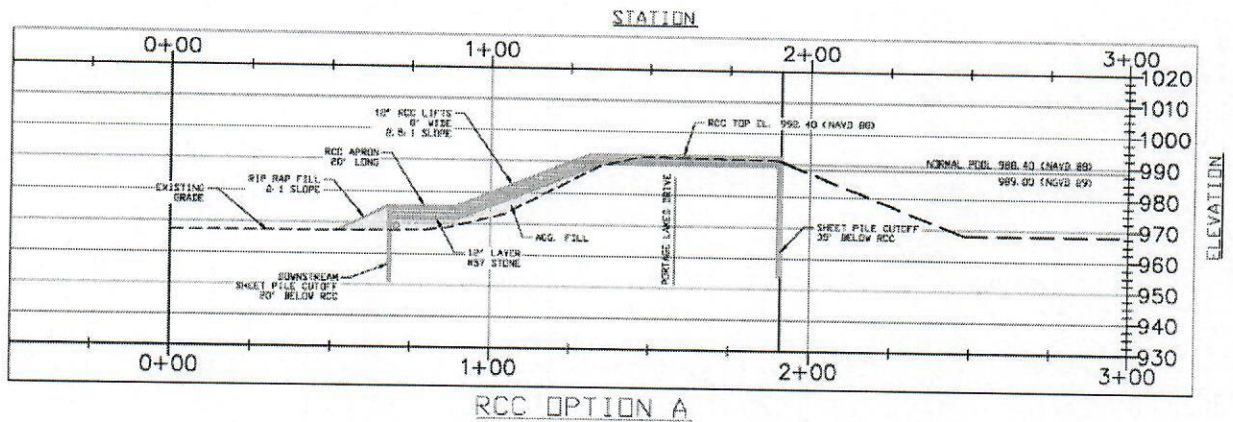


Note that DSM (deep soil mixing) and soil-cement are used interchangeably in this paper.

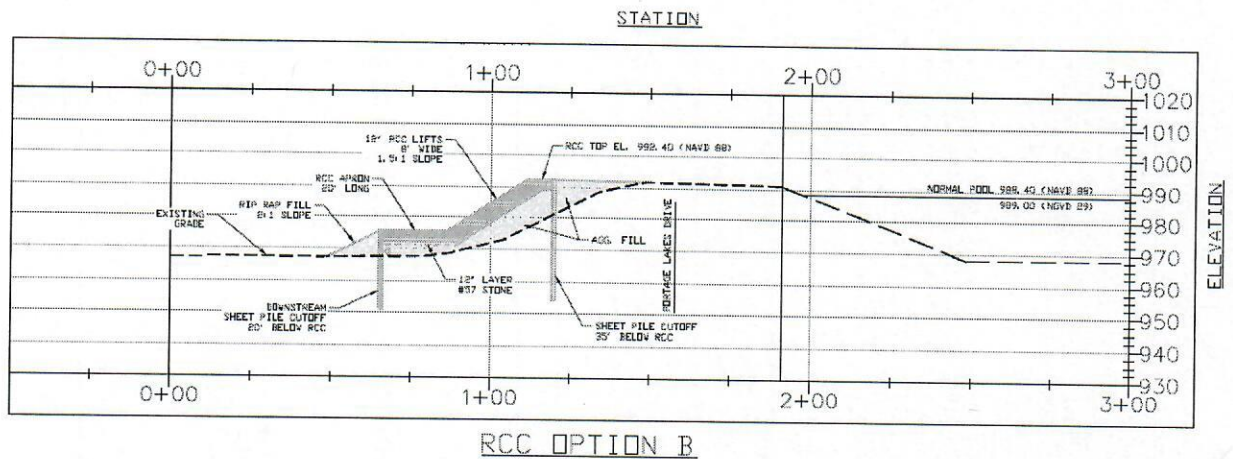
Sheetpile Cell Wall



Roller Compacted Concrete (RCC) on Existing Embankments (RCC Option A)



RCC on Downstream Embankment Berm (RCC Option B)



V. CONSTRUCTION COST ESTIMATES

Preliminary cost estimates were prepared for each of the considered alternatives. These costs are presented in Table 2. As can be seen from the costs, the most cost-effective alternative was the DSM Gravity Section constructed upstream of the existing embankment. The costs assume that this alternative would be constructed in the wet, without disrupting the normal pool or the normal operations of the lake.

Table 2 - Preliminary Construction Cost Estimate

Portage Lake State Park East Reservoir Dam Improvements Summit County, Ohio Project No. DNR-130036			Preliminary Costs Tetra Tech Alternatives Analysis	
EAST RESERVOIR EMBANKMENTS (\$ MILLIONS)				
TYPE	MAIN	NORTH	SOUTH	TOTAL
Diaphragm Wall w/Brace	\$16.9	\$16.5	\$11.2	\$45
Composite Double Sheetpile Wall w/Brace	\$11.7	\$12.0	\$7.9	\$32
DSM Gravity Section	\$10.8	\$11.1	\$7.7	\$30
Cell Wall	\$13.8	\$12.7	\$8.4	\$35
RCC				
Option A	\$18.6	\$15.2	\$7.7	\$42
Option B	\$21.2	\$15.2	\$7.7	\$45

VI. FINAL DESIGN SUBSURFACE EXPLORATION

In order to support final design efforts, a subsurface exploration of Portage Lakes East Reservoir was conducted between the dates of September 9, 2015 and February 2, 2016. The drilling was performed by DLZ American Drilling, Inc., and was conducted under the supervision of Tetra Tech and our on-site geologist. The exploration consisted of a total of 22 explorations with 13 SPT borings and 9 CPT soundings. Of the 22 explorations, 12 were performed in the lake from a barge. The SPT borings were drilled to depths ranging from 16.5 to 95.0 feet. Similarly, the CPT soundings were drilled to depths ranging from 43.3 to 72.3 feet.

The final design borings (see Figure 5 below) generally confirmed the granular nature of the embankment and foundation soils but cohesive soils were encountered in some areas. In addition, in certain areas, the final design borings identified a relatively complex and variable profile of discontinuous layers of more permeable sands and gravels and cohesive soils. In some areas, these cohesive soils were near the anticipated base of the proposed DSM gravity section. The subsurface conditions for the south embankment, main embankment, and in the marina and north embankment area are described in the paragraphs below.

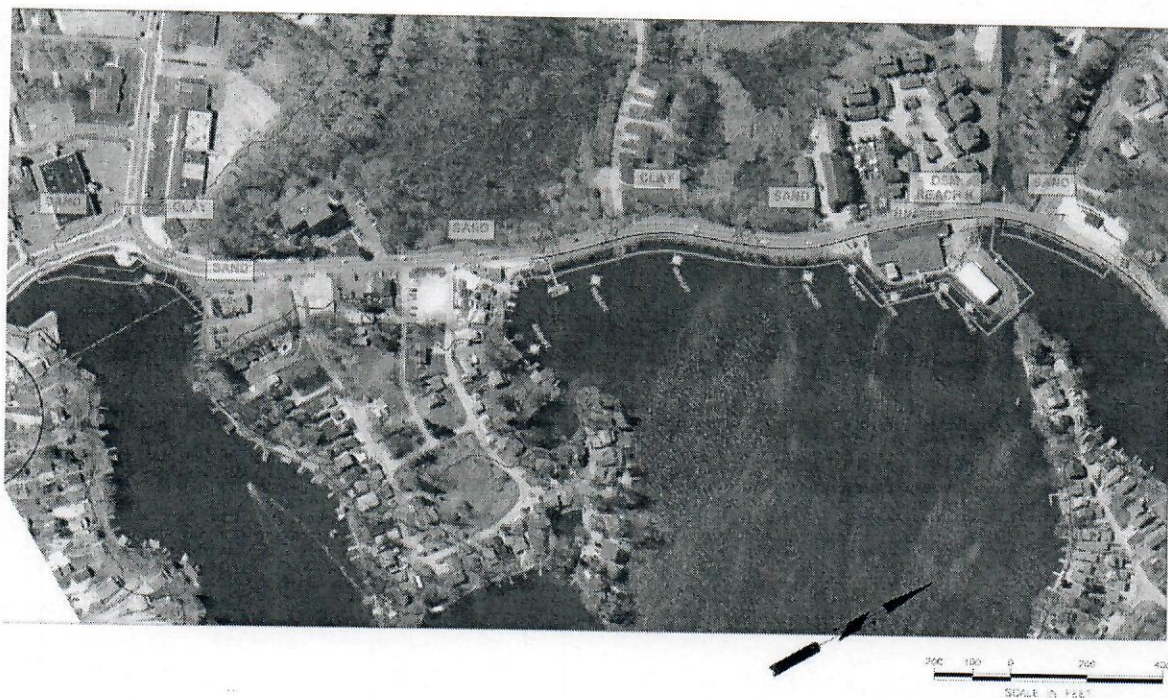


Figure 5 – Boring Locations and Reach Materials

Five borings were drilled near the south embankment; four SPT borings and one CPT sounding. The boring designations are EB-20-SPT, EB-21-SPT, EB-22-CPT, EB-23-SPT, and EB-24-SPT. The SPT borings generally encountered interbedded layers of poorly-graded sand (SP) and well-graded sand with silt (SW-SM) down to an elevation of 978.0 feet, approximate bottom of the south embankment, and continuing to an elevation of 905.0 feet. The borings did encounter pockets of silty clay (CL-ML) of varying thickness between elevations 972.0 and 950.0 feet. The lone CPT boring encountered similar material as encountered in the SPT borings. Results of the CPT boring indicate silt and clay mixtures to between elevations 974.0 to 962.0 feet and clean sands to the termination of the boring at elevation 916.0 feet.

Ten borings were located near the main embankment of the dam; five SPT and five CPT soundings. Boring designations are EB-25-SPT, EB-26-CPT, EB-27-1-SPT, EB-27-2-SPT, EB-28-CPT, EB-29-SPT, EB-30-CPT, EB-31-SPT, EB-32-CPT, and EB-33-CPT. The SPT borings generally encountered interbedded layers of poorly to well graded sand (SW, or SP) and silty sand (SM) in the embankment and foundation materials. EB-25-SPT did encounter a silt (ML) layer between elevations 944.8 and 934.3 feet. The CPT borings encountered similar materials as encountered in the SPT borings. The results of the CPT data indicated sandy material throughout with thin gravel seams interbedded within. EB-30-CPT differed from other CPT borings as it encountered silty soils between elevations 966.0 and 958.0 feet.

Seven borings were drilled near the Kieffer marina, the north embankment of the dam; four SPT and three CPT soundings. The borings near the north embankment contained the most frequent variations of soil types and layers encountered at the project site. The borings are designated as EB-34-CPT, EB-35B-CPT, EB-35-CPT, EB-36-SPT, EB-37-

SPT, EB-38-SPT, and EB-39-SPT. Borings EB-34 and EB-35B encountered similar conditions, which were clean to silty sands and pockets of gravelly sand throughout the boring depths. The remaining borings encountered varying layers and thickness of lean clay (CL), silty clay (CL-ML), silty sand (SM), and poorly-graded sand with silt (GW-GM).

Groundwater measurements were obtained during drilling and at the completion of drilling. Seepage was first encountered in borings EB-20 and EB-24, at elevations of 985.5 and 989.7 feet, respectively. Ground water level during time of drilling in the borings along the crest of the main embankment ranged from 976.2 feet to 973.8 feet in elevation in borings EB-27-1, EB-29, and EB-31. Boring EB-27-2 encountered seepage at an elevation of 974.4 feet. Borings EB-37, EB-38 and EB-39 encountered seepage between elevations of 973.6 and 986.8 feet. Although normal pool elevation is 988.3 feet, ground water levels represent the conditions during the last reading and should be expected to fluctuate with seasonal change and variations in the pool.

VII. FINAL DESIGN LABORATORY PROGRAM

A laboratory testing program was created to determine the engineering properties of the subsurface materials. Classification testing was conducted on select samples and consisted of moisture content tests, particle-size analysis, and Atterberg Limit testing. Shear strength testing consisted of three consolidated undrained (CIU) tests (ASTM D4767), five unconsolidated undrained (UU) tests (ASTM D2850), and six direct shear (DS) testing (ASTM D3080).

VIII. REACH DETERMINATIONS

As mentioned above, the majority of the project alignment is characterized as having primarily granular, cohesionless subsurface profiles. However, in certain areas, the results of the subsurface exploration program revealed that fine-grained cohesive soils are present near the anticipated level of the bottom of the proposed DSM gravity section. A third area of difference was near the marina at the right abutment of the main embankment. The soils in this area were mixed and included granular soils mixed with firmer cohesive soils.

Based on the review of the subsurface conditions, the alignment was divided into reaches of similar subsurface conditions. These included two reaches in the south embankment, three reaches in the main embankment, and a single reach in the north embankment. The reaches are described in more detail below.

IX. SOUTH EMBANKMENT

In the borings for the south embankment alignment, two types of profiles were encountered. Near the south spillway, the borings encountered softer clayey soils to an elevation of 962, where sandy soils were encountered. However, the borings along the remainder of the south embankment alignment general encountered granular soils or very stiff to hard fine-grained soils from the lakebed to the depths of the borings. The south embankment reaches are described in more detail in the paragraphs below.

Reach S-1 (Sand): This reach is located in the southern portion of the south embankment area and the subsurface profile is represented by the findings of EB-20-SPT and EB-21-SPT. The findings of the SPT borings revealed that the proposed base will generally be situated in granular soils consisting of poorly-graded sand (SP) and poorly-graded sand with silt (SP-SM). Sporadic, thin, layers of silty clay (CL-ML) were encountered in the foundation. However, the estimated undrained shear strength of these soils indicate that the clay strength will not control the external stability of the DSM section.

Reach S-2 (Clay): This reach is located in the south embankment area near the south spillway and the subsurface profile is represented by the findings of EB-22-CPT and EB-1. The findings of the CPT boring revealed that soils consisting of clay to silty clay were encountered to an elevation of approximately 962. Historic boring EB-1 indicated that "peat" was encountered. However, no particle size (or LOI) testing was conducted for this strata and the SPT N-values, ranging from 6 to 15, indicate that this layer has a density that is greater than what would be expected in a "peat" layer.

Reach S-3 (Sand): This reach is located in the northern portion of the south embankment area and the subsurface profile is represented by the findings of EB-23-SPT and EB-24-SPT. The findings of the SPT borings revealed that the proposed base will generally be situated in granular soils consisting of silty sand (SM) and poorly-graded sand with silt (SP-SM).

X. MAIN EMBANKMENT

Along the main embankment, a review of the borings resulted in three general subsurface profiles. In borings near Canova Drive, silt and silty clay were encountered near the anticipated depth of the gravity section base. In borings near the Kieffer's Marina, a more mixed subsurface profile was encountered with some borings encountering all sands while other borings encountered cohesive soils. Borings in the remaining portions of the main embankment alignment encountered mostly granular soils.

Reach M-1 (Sand): This reach is located in the main embankment near the left abutment area and the subsurface profile is represented by the findings of numerous borings; EB-2, and EB-3; EB-26-CPT and EB-28-CPT; EB-25, 27-1, and 27-2-SPT. The findings of the borings revealed primarily a granular (SP-SM and SP) profile. However, several of the borings drilled within the embankment section encountered sporadic soil layers classified as silt (ML), encountered near the base elevation of the proposed DSM section. Atterberg limit testing on these samples reveals that this layer is likely non-plastic.

Reach M-2 (Clay): This reach is located along the main embankment in the Canova Drive area and the subsurface profile is represented by the findings of numerous borings; EB-4, 5, and EB-9; EB-30-CPT; and EB-29-SPT. The findings of the borings revealed primarily a granular (SP-SM and SP) profile. However, several of the borings drilled within the embankment section encountered fine-grained silt (ML) and silty clay (CL-ML) layers near the base elevation of the proposed DSM section. It is assumed that this layer will behave in an undrained manner when a change in stress occurs.

Reach M-3 (Sand): This reach is located in the main embankment area and the subsurface profile is represented by the findings of EB-6, EB-8, EB-31-SPT, EB-32-CPT and EB-33-CPT. The findings of the borings revealed that the proposed base will

generally be situated in granular soils consisting of silty sand (SM), poorly-graded sand (SP), and poorly-graded sand with silt (SP-SM).

Reach K (Mixed Profile): This reach is located in the main embankment area near the Kieffer Marina, between the main reservoir and the Cat Swamp. The subsurface profile is represented by the findings of EB-34-CPT, EB-35-CPT, EB-35B-CPT, EB-26-SPT, EB-37-SPT, and EB-38-SPT. The findings of the borings revealed that the proposed base will generally be situated in a mixed soil profile. Sandy soils were generally encountered in the borings drilled along the southern bank, near the peninsula, while the borings drilled on land encountered lean clay (CL) and silty clay (CL-ML) soils. The cohesive soils exhibit properties consistent with glacial till soils with shear strengths significantly higher than other cohesive soils encountered on the project.

XI. NORTH EMBANKMENT

Primarily granular soils were encountered in the borings along the north embankment and a single reach was used in this area. A description of the north embankment reach is presented below.

Reach N (Sand): This reach is located in the north embankment area and the subsurface profile is represented by the findings of borings EB-38-SPT, EB-39-SPT, and EB-7. The findings of the borings revealed primarily a granular (SP-SM, SP, SC-SM) profile. Boring EB-38-SPT did encounter very stiff to hard silty clay (CL-ML) approximately 20 feet below the base of the DSM section. However, this layer was well below the base and did not affect the external stability assessment.

XII. FINAL DESIGN MATERIAL PROPERTIES

The shear strength properties were estimated using the results of SPT borings, CPT soundings, shear strength test results, pore pressure dissipation (DDP) tests, classification test results, correlations with SPT N-values and CPT metrics, and engineering judgment. The selected material properties on Table 3 were used in the evaluations of the critical sections and the proposed berm and DSM structure.

Table 3 – Assumed Material Properties

Stratum	Total Unit Weight (pcf)	c (psf)	ϕ (deg.)	c' (psf)	ϕ' (deg.)	Hydraulic Conductivity, k_h		k_v/k_h
						(ft/s)	(cm/s)	
Dam Fill (SP/SM)	125	0	32	0	32	1.0E-04	3.0E-03	0.5
Berm Fill (SP/SM)	120	0	30	0	30	3.3E-05	1.0E-03	1
CL-ML	125	Varies ¹	0	50	28	3.3E-08	1.0E-06	0.5
ML non-plastic	124	0	32	0	32	3.3E-08	1.0E-06	0.5
SM - Fnd.	125	0	32	0	32	3.3E-05	1.0E-03	0.5
SP I - Fnd.	125	0	34	0	34	6.6E-04	2.0E-02	1
SP II - Fnd.	128	0	36	0	36	6.6E-04	2.0E-02	1
SP III - Fnd.	130	0	40	0	40	6.6E-04	2.0E-02	1

1 – varied by Reach.

XIII. FINAL DESIGN ANALYSES

A number of analyses were performed to determine the configuration and stability of the new DSM overtopping structure. These analyses included seepage analyses to replicate the existing conditions and to determine exit gradients following an overtopping event (when the pool is at the top of the dam, the embankment has washed away, and the foundation soils are eroded to the ultimate scour elevation of 965.5). Global stability of the DSM overtopping structure was evaluated using limit-equilibrium methods assuming an overtopping event had occurred and the failure occurred along a surface beneath the DSM structure. Sliding and overturning of the DSM structure were evaluated using the requirements and guidance in EM 1110-2-2200, Gravity Dam Design assuming post-overtopping conditions. Finally, stability of the construction berm was also evaluated.

More detailed information regarding these evaluations and the analyses results are presented in the sections below. However, prior to performing the final design analyses, the subsurface conditions were reviewed to determine the locations of the cross-sections to be analyzed. The results of that review are discussed in the following section.

XIV. SELECTED ANALYTICAL CROSS SECTIONS

Several analyses were conducted for the project. These analyses consisted of evaluating the existing conditions, the proposed berm, and the proposed DSM gravity section. In general, the analyses sought to focus on the most critical conditions, which would control the design and proportions of the proposed DSM sections. Table 4 provides a general summary of the features evaluated along with the subsurface conditions that were

assumed in each analysis. Additional details regarding the evaluations and assumed subsurface conditions are presented in the following sections.

Table 4 – General Summary of Features Evaluated

Feature Evaluated	General Subsurface Soils ¹				Analyses Conducted		
	Clay - 1A	Clay - 1B	Clay - K	Sand - 1C	Global Stability	FE Seepage	External Stability
Existing Conditions	X	X	0	0	X	X	NA
Prop. Berm	X	X	0	0	X	0	NA
DSM Section	X	X	0	X	X	X	X
DSM Section	0	0	X	0	0	0	X

1 – Generalized subsurface profile / soil types considered in evaluations.

XV. TOPOGRAPHY/BATHYMETRY

For the design efforts, only a single embankment section was assumed. This section was located just north of Canova Drive and represents the maximum embankment height. Furthermore, this section contains the most critical upstream topography since the depth to the lake bottom is deepest in this area. The location of this critical cross section is shown in Figure 7.

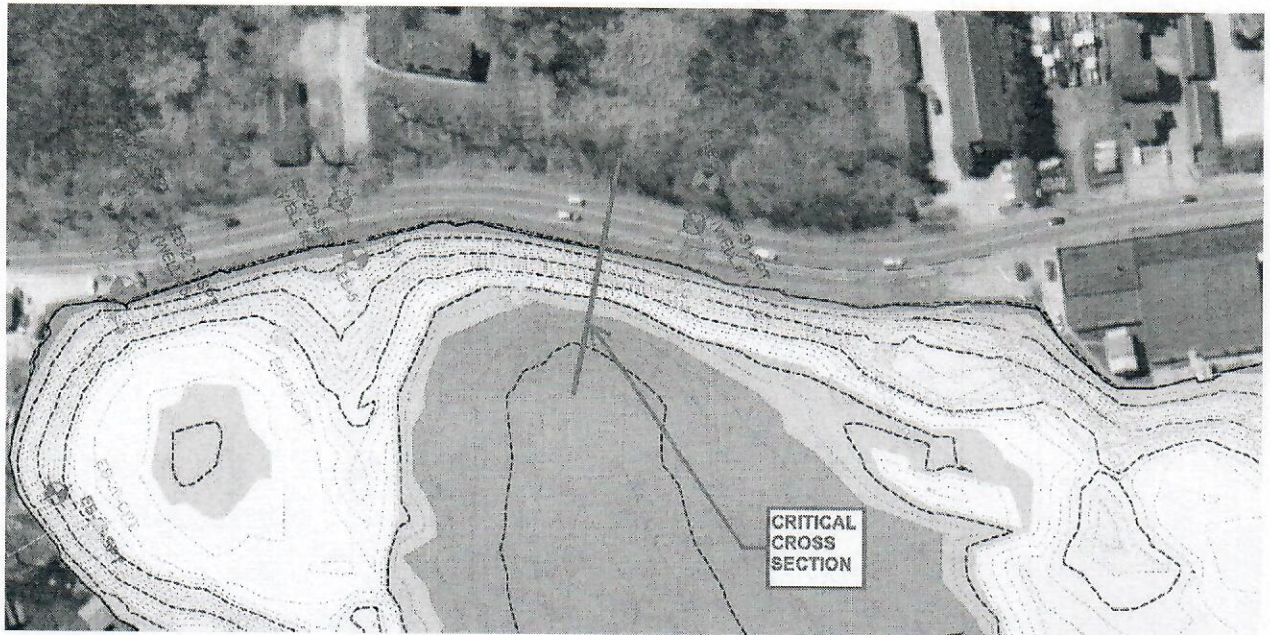


Figure 7 - Critical Cross Section Location

XVI. SUBSURFACE CONDITIONS

As the subsurface conditions in the various reaches were reviewed, it became clear there were three generalized subsurface conditions along the alignment of all three embankments: 1) a more cohesive profile overlying granular soils (mostly in the area of Canova Drive), 2) a mixed profile of sands and stiff clays encountered near the marina, and 3) a subsurface profile consisting entirely of granular soils. Combining these three subsurface profiles with the critical analytical section at the maximum embankment height allowed the conditions anywhere along any the embankments to be conservatively modeled in the analyses.

XVII. ANALYSES

A. Initial Analyses

The developed cross-sections used for the existing conditions evaluations, were modified to add the construction berm, DSM gravity section, and the DSM cutoff wall for the design analyses. In addition, the post-overtopping conditions were used in the design analyses assuming the embankment downstream of the DSM has washed away and the foundation soils scoured to an elevation of 965.5.

Initial stability analyses were performed to determine the trial depths and widths of the DSM gravity sections. The configuration of the gravity section and the cutoff wall are shown in Figure 8 below.

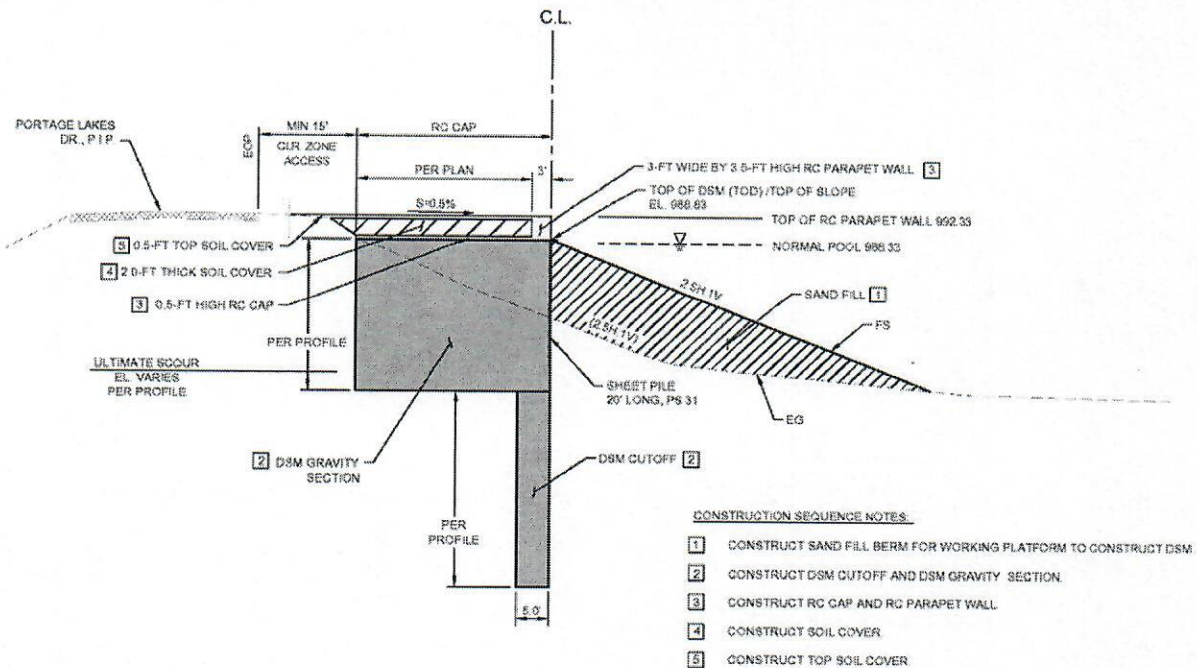


Figure 8 – Typical DSM (soil-cement) Cross-Section

These initial analyses indicated that the DSM gravity sections could not be founded in weak cohesive soils and must extend into an underlying granular layer. Therefore, the analyses assumed that the DSM gravity sections were founded in granular layers below any weak cohesive layers.

B. Seepage and Uplift Analyses

Seepage was evaluated using the computer program Slide V 7.013, developed by Rocscience. Slide is a 2D limit equilibrium slope stability program. However, in addition to conducting slope stability analyses, Slide includes a finite element groundwater seepage analysis program.

The Slide program is capable of conducting both transient-state and steady-state evaluations. However, for this work, only steady-state evaluations were conducted. The steady-state phreatic surface was computed using the Slide groundwater analysis program.

In general, the maximum vertical hydraulic gradient at the ground surface (exit gradient) was identified in the analysis. This maximum gradient was compared to the critical gradient, which is a function of the unit weight of the soil. As per EM 1110-2-1901 Seepage Analysis and Control for Dams, typical factors of safety (against critical gradient) range from 2.5 to 5.0. When considering the required factor of safety, Tetra Tech considered the nature of the structure and the frequency of the loading on the structure. Typically, the DSM gravity section will be buried within the embankment on the upstream side, this is the normal condition and is not critical from a stability standpoint. The critical loading case for the DSM gravity section is one where an overtopping event scours the downstream embankment. This is a rather infrequent event. As such, Tetra Tech adopted a factor of safety of 3.0 against critical gradient.

If elevated exit gradients were computed, an alternate seepage analysis was conducted. This alternate analysis sought to determine if there was a potential for foundation soil beneath the confining layer to be transported to the exit location. This piping can occur when the upward seepage forces are high. Evaluations of the seepage pathway around the proposed DSM gravity structure were made in accordance with Lane and Bligh. In general, the length of the seepage pathway relative to the net head across the structure was compared to a weighted creep coefficient (or ratio). The creep ratio was based on the soil type and its susceptibility to piping. The granular foundation soils are characterized as fine sand and silty sand.

The results of the analyses indicate that the exit gradient is expected to be 0.23 where an all sand profile is encountered. This provides a minimum factor of safety of 4.3, which is above the adopted minimum value of 3.0. This assumes that a seepage cutoff is installed to elevation 935. In general, the tip of the cutoff is situated approximately 30 feet below the base of the DSM gravity section.

C. Global Stability Analyses

Embankment and foundation stability for the overtopping structure were evaluated in accordance with USACE guidelines and EM 1110-2-1902 Slope Stability. Slope stability was evaluated using limit-equilibrium analyses. These limit equilibrium analyses were performed using the computer program Slide V 7.013, developed by Rocscience. The

program is capable of performing slope stability evaluations using many methods, however, for these evaluations Spencer's method of analysis was used. For each case, where a subsurface layer was characterized by effective stress shear strength parameters (cohesionless soil), the groundwater conditions assumed were based on the steady-state phreatic surface, which was computed using Slide's finite element groundwater program.

The steady-state seepage evaluations were conducted assuming the upstream pool level for that particular load case; either normal pool or PMF pool. The tailwater was taken to be the elevation at the downstream toe (~el. 968), and for PMF conditions the tailwater is taken to be elevation 969.4.

Where global stability was assessed for the proposed DSM gravity section following an overtopping event, the portions of the seepage cutoff were conservatively assumed to have the shear strength of the surrounding soil. However, in order to properly model the impact of the cutoff on the hydrostatic pressures within the foundation, the hydraulic conductivity of the seepage cutoff was the same as that used for the DSM material.

The results of the global stability evaluations demonstrated that the DSM gravity section met the required minimum safety factor of 1.5.

D. External Stability (Overturning and Sliding)

External stability of the proposed DSM gravity section was conducted in general accordance with EM 1110-2-2200, Gravity Dam Design. Evaluations were conducted assuming the full PMF headwater on the upstream side, coupled with the ultimate scour elevation on the downstream side.

A seepage cutoff element was assumed in the analyses in order to reduce the exit gradients to acceptable values. The results of the seepage analyses, discussed in previous sections, were used to determine the uplift pressures acting on the base of the DSM gravity section. Since the seepage analyses were performed assuming two different subsurface conditions (with a confining layer and without a confining layer), the most critical (highest) uplift pressures of the two conditions were used in the external stability calculations.

The foundation depth for the gravity section and its width were based on the results of these analyses. If the minimum sliding factor of safety of 1.7 wasn't met or if entire base was not in compression, the width, foundation depth, or both were increased until suitable results were calculated.

E. Additional Design Analyses

Two additional design analyses were performed: 1) strain compatibility at the interface between the DSM gravity section and the DSM cutoff wall, and 2) an internal stress evaluation of the individual DSM columns within the gravity section. Both of these analyses are described in more detail below.

Strain Compatibility

Another issue evaluated was the strain incompatibility between the stiff DSM gravity section and cutoff wall and the foundation soils. Following an overtopping event (and

assuming the embankment is washed away and the foundation is scoured to a depth of 5 feet), vertical and lateral loads on both the gravity section and the cutoff wall will result in a strain incompatibility between the DSM materials and the surrounding soils. Because the DSM cutoff wall and gravity section are stiffer than the foundation soils, this strain incompatibility could result in horizontal cracking in the cutoff wall near the base of the gravity section.

Tetra Tech used 2 Plaxis to evaluate the anticipated stresses and deformations in the cutoff wall and the gravity section. Based on the results of the Plaxis analyses, tension does not occur in the cutoff wall (although the compressive stress does decrease).

Internal Stress Evaluation for the Gravity Section

Since the DSM gravity section will be constructed using discrete DSM columns, internal stress analyses were required to confirm that the uplift pressures beneath the section don't overstress the interfaces between the individual DSM columns. Typically, the allowable shear stress between individual DSM columns is taken as approximately 60% of the design unconfined strength of the soil-cement mix.

These evaluations were performed for each cross-section selected for analyses. However, the results of the analyses indicated shear stresses well below the allowable shear stress of 60 psi between the individual DSM columns.

XVIII. ANALYSES CONCLUSIONS

The evaluations discussed previously were used to develop the DSM cross sections for each reach. In general, the gravity section was essentially square, with the width about 10% greater than the height. The seepage analyses, uplift calculations, evaluations of both internal and external stability, as well as sliding and overturning analyses, indicate that the soil-cement gravity section will be stable following an overtopping event that washes away most of the material in front of the soil-cement structure. Due to the long duration of the overtopping event (nearly 8 hours), a reinforced concrete cap and parapet wall will be constructed on top of the soil-cement sections. The parapet wall will be the actual top of dam and the concrete cap will be covered with 2' of soil cover, then topsoil and seeded. A cross-section of the proposed soil-cement overtopping structure is shown on the following page.

