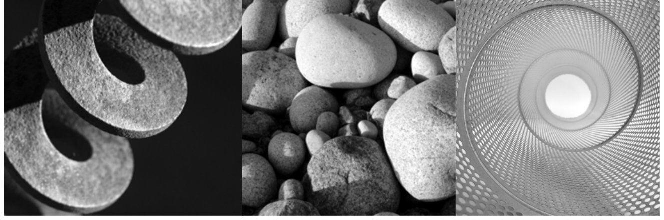

GEI Consultants

Consulting
Engineers and
Scientists

Upper Black Creek Reservoir

Spillway Reconstruction Project

Feasibility Study/Design Report

Dam ID: 360127

Filing No: C-XXXX

Submitted to:
Blue Lake Reservoir Company
Summit Trust
Vail, CO 81657

Submitted by:
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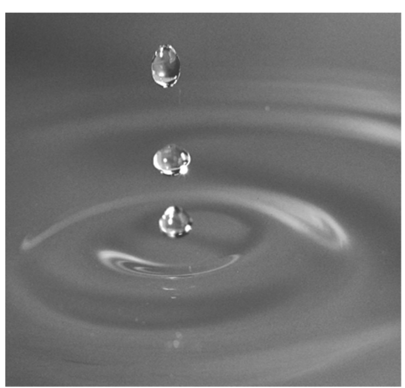
April 2020
Project 1801834



Chad Masching, PE
Project Manager

Paul Eggers, PE
Reviewer

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Executive Summary

Several deficiencies have been identified with the existing spillway at the Upper Black Creek Reservoir (UBCR), including inadequate spillway sizing to safely pass the inflow design flood (IDF) and deterioration of the spillway chute slab. Following the results of previous work, including a hydrology study and an evaluation of the concrete spillway structure, the Colorado Office of the State Engineer (SEO) ordered a storage restriction on the reservoir until improvements to the spillway could be made.

In December 2017, GEI Consultants, Inc. (GEI) drilled 4 borings at UBCR. One boring was advanced through the left abutment of the spillway and one advanced upstream of the concrete spillway apron. Two core holes were drilled in the middle third of the spillway chute concrete slab. The borings were advanced to characterize the foundation in terms of depth to bedrock, soil type, and to evaluate material strength and competency.

In February 2019, GEI completed a hydrology study to develop and evaluate three rainfall events to determine the critical IDF to be used to bring the spillway back into compliance with SEO Regulations. The three rainfall events evaluated were the 2-hour Local Storm, 6-hour Local Storm, and 48-hour Mid-Latitude Cyclone (MLC) General Storm. The model results indicated that the 6-hour Local Storm resulted in the highest water surface elevation in the UBCR with less than the required foot residual freeboard. Thus, the 6-hour local storm is the IDF for the UBCR Project and was used for evaluation for the alternatives considered for the project.

This further analysis performed on the UBCR Dam was used to develop a recommended preferred alternative for providing a spillway which meets the flood routing requirements of the SEO *Rules and Regulations for Dam Safety and Construction*. As part of the Feasibility Study, three alternatives to bring the UBCR back into compliance with SEO Regulations were considered. These alternatives included: 1) repairing the existing spillway; 2) replacing the existing spillway without a dam raise; and 3) replacing the existing spillway with a dam raise. GEI recommended replacing the spillway without raising the dam as this option provides a structure with a long life expectancy that meets SEO criteria and does not put additional loading on the dam during large flood events.

The proposed design for the replacement spillway is similar to that of the existing spillway with a concrete chute, an approach channel, concrete weir, and stilling basin. The replacement spillway is designed as a concrete overflow structure and maintains the normal reservoir surface elevation (El.) 8748.0. The spillway chute is 38 feet wide at the spillway crest and then contract to a width of 32 feet at the stilling basin. The spillway walls vary in height, with 13-foot-high walls in the stilling basin and 9.5-foot-high chute walls. The stilling basin is designed to dissipate much of the hydraulic energy from the drop in elevation from the spillway crest to the toe of the structure. The channel downstream of the spillway structure will be armored with 25 feet of riprap to provide protection against scour and undermining of the concrete structure.

1. Introduction

This Feasibility Study and Design Summary Report summarize the analyses that have been performed on the Upper Black Creek Reservoir (UBCR) Dam to develop recommendations for a preferred alternative for providing a spillway which meets the flood routing requirements of the Colorado Office of the State Engineer, Dam Safety Branch (SEO) *Rules and Regulations for Dam Safety and Construction* (referred to herein as current SEO Regulations) (SEO, 2020). The owner of the reservoir is receiving a matching grant through the Colorado Water Conservation Board to perform this work. This report provides the technical evaluations conducted by the design team, including the disciplines of civil, geotechnical, and structural engineering, hydrology and hydraulics, and construction. Project permitting for construction was also performed.

1.1 Project Overview and Objectives

Black Lake and the UBCR are located approximately 24 miles northwest of Silverthorne, in Summit County, Colorado. The reservoirs are owned and operated by the Blue Lake Reservoir Company (Owner) and are used for domestic water consumption, wildlife habitat, and recreation purposes. The dams are located in series on Black Creek, which is a tributary to the Blue River. Pertinent data for UBCR is provided in **Table 1.**

Table 1.
Upper Black Creek Reservoir Dam Pertinent Information

Design Parameter	UBCR Dam
Storage at Dam Crest (ac-ft)	655.0
Storage at spillway (ac-ft)	428.0
Dam Crest El. (ft)	8,755.5
Natural Streambed El. (ft)	8,719
Dam Height (ft)	29.0
Spillway Type	Fixed Crest (concrete)
Spillway Location	Right Abutment
Spillway Crest El. (ft)	8,748.0
Normal Pool El. (ft)	8,748.0
Spillway Width (ft)	28.0
Freeboard at Normal Pool (ft)	7.5
SEO Jurisdictional Dam (Y/N)	Y
SEO Size Classification	Significant

Following the results of previous work, including a hydrology study and an evaluation of the condition of the concrete spillway structure, the SEO ordered a storage restriction on the reservoir until improvements to the spillway could be made. In addition, the SEO recommended that the hydrology study be updated following the formalization of the

Colorado-New Mexico Regional Extreme Precipitation Study and recommended that a geotechnical investigation be performed to assist in the assessment of the spillway. GEI completed a geotechnical exploration at UBCR on December 28, 2017. On February 16, 2018, a memorandum summarizing the results of the exploration and associated laboratory testing program was finalized. The recommended hydrology report and plans to bring the project back into compliance with the SEO Regulations was completed by GEI in February 2019. This report documents the critical Inflow Design Flood (IDF) for UBCR and recommends a preferred alternative for providing a spillway which meets the flood routing requirements of the SEO based on evaluations of previous work and work performed by GEI.

1.2 Overview of Previous Work

In 2011, the Owner's previous engineer, Resource Engineering, Inc., prepared and submitted a hydrology study to the SEO performed to determine the adequacy of the existing spillway to meet the SEO's 2007 *Rules and Regulations for Dam Safety and Dam Construction* (referred to herein as previous SEO Regulations). The hydrology study was reviewed by the SEO and generated comments that required additional analysis. However, the Owner terminated the contract with Resource Engineering before the comments could be addressed, and they remain unresolved.

In August 2017, the Owner hired a contractor, Restruction Corporation, to conduct a field investigation and analysis of the existing spillway and provide recommendations for repairs of previously identified concrete deterioration. The exploration was conducted in August 2017. It was reported that during concrete coring on the lower portion of the spillway slab, pressurized muddy water was encountered in addition to an apparent 2.75-inch-deep void beneath the slab, indicating a build-up of hydrostatic pressure under the slab. The presence of voids under the spillway combined with uplift pressure presented a hazard for internal erosion of the spillway foundation and the potential for movement of the slab during spillway operation due to the net weight of the concrete being reduced and no anchorage of the spillway slab to the foundation rock. Additionally, the water under the slab could lead to freeze/thaw heave of the structure. The SEO met with Restruction and the Owner's dam tender in November 2017 to discuss the observed spillway condition and the results of the concrete evaluation. Following this meeting, the SEO recommended that a geotechnical investigation be performed to assist in the assessment of the spillway. Restruction contracted with GEI to perform the geotechnical study aimed to aid in evaluating the spillway subsurface conditions, in addition to the results of the concrete evaluation of the spillway.

In January 2018, based on the information in the 2011 hydrology study prepared by Resource Engineering and subsequent comments generated by the SEO dam safety engineer at the time, the SEO performed a hydrologic and hydraulic analysis to determine the safe reservoir storage level to mitigate the dam safety hazards present at the site. The 2018 SEO analysis performed the following to address the 2011 SEO comments:

- updated the flood routing methodology;
- modeled time steps; unit hydrographs; loss rates; storm events; rainfall depths; rainfall durations; temporal rainfall distributions; and,
- developed spillway weir coefficients and spillway rating curves.

It was determined from the modeling results estimated by the SEO that the spillway for the UBCR could not safely pass the IDF, resulting in approximately 0.6 feet of overtopping the earthen embankment. From these results, in addition to the results of the concrete evaluation of the spillway, a reservoir restriction to elevation (El.) 8739.0 was set by the SEO until improvements to the spillway could be made. The reservoir restriction is 9 feet below the spillway crest El. 8748.0. Following this analysis, the SEO recommended that the hydrology study be updated following the formalization of the Colorado-New Mexico Regional Extreme Precipitation Study and current SEO Regulations. If the new regulations and design tools would have significantly reduced the storm routing requirements and the spillway could be shown to pass the required storm inflow, options to rehabilitate the current spillway could have been considered. This hydrology study was included in GEI's Scope of Work.

1.3 Description of Facilities

Upper Black Creek Dam is a 650-foot-long earth fill embankment dam. The embankment has an irregular L-shape configuration with the long axis running approximately east-went, with a reported nominal crest El. 8755.5, crest width of 12 feet, and a structural height of about 29 feet. The existing spillway is a 28-foot-wide concrete control structure located on the right abutment of the dam with a crest El. 8748.0. The existing capacity is approximately 1,440 cubic feet per second (cfs) with 1 foot of residual freeboard. The dam has an outlet works conduit with an upstream control gate for regulating the reservoir water level and providing required downstream releases. The reservoir is currently on a storage restriction at 9 feet below the spillway crest.

1.4 Design Team

The team responsible for performing the engineering analyses and design includes project personnel from GEI. **Table 2** provides the project personnel and their roles and responsibilities on the Project.

Table 2.
Design Team

Individual	Project Role
Chad Masching, PE	Project Manager; Engineer of Record
Margaret Provencher, PE	Structural Engineer
Cassidy Diebold, EI	Project Engineer
Paul Eggers, PE, PMP	Project Reviewer
Paul Drew, PE (WI)	Hydraulics and Hydrology
Gillian Williams, PE	Geotechnical Engineer

1.5 Related Documents

This report references additional reports that document certain aspects and decision points of the Project in greater detail. Many of the documents are provided as appendices to this report. The reader should refer to the companion documents for more information regarding Project elements or design recommendations. Reports and Technical Memoranda that were instrumental in design of the Project but are not part of this report are provided in **Table 3**.

Table 3.
Other Reports

Title	Author	Status	Date Issued
Upper Black Creek Spillway Geotechnical Investigation	GEI	Final	February 16, 2018
Upper Black Creek Reservoir Reconstruction Project Hydrology Report	GEI	Final	July 2019

2. Site Conditions

2.1 Geology

UBCR is situated in a valley on the eastern side of the Gore Mountains in the Southern Rocky Mountains of Colorado. The Gore Mountains are a prominent northwest-southeast trending range with peak elevations between 12,500 and 13,500 feet. The range is fault bounded and is composed of old and resistant Precambrian basement rocks that have been uplifted during the Laramide orogeny. On the east side of the Gore Range, the lower flanks contain a sequence of Cretaceous sedimentary units deposited prior to the Laramide uplift, including the Pierre Shale and the Dakota Formation. The UBCR is situated between the upper peaks and the lower sedimentary units where the bedrock is composed of the Paleoproterozoic gneiss or granite, but these units are covered by a thick sequence of surficial glacial and landslide deposits.

The landslide deposits are mostly present within the impoundment area, and the glacial deposits are present at the embankment. The landslide features are likely glacial deposits that became unstable due to steep slopes. The glacial deposits are Pinedale in age, and are composed of non-sorted and non-stratified, matrix supported cobbles and boulders. The matrix is mostly poorly sorted sand but also contains some silt and clay. The thickness of the glacial deposits varies significantly; the unit can be as much as 100 feet thick in places.

2.2 Seismic Setting

UBCR is located along the northernmost reaches of the Rio Grande rift. The tectonic conditions that resulted in the uplift of the Gore Range and the Southern Rocky Mountains have been debated for several years because this high elevation region is far from subduction zones and seismic areas that typically result in mountain building. Although many questions remain outstanding regarding the mechanism of uplift of the Gore Range, the faults that bound the range are considered active and are the primary source of the seismic hazards at UBCR. The Gore fault is located on the west side of the range and the Blue River fault (also known as the Frontal fault) is located on the east side of the range. Several other associated faults have been mapped in the area and the Williams Range Thrust Fault is approximately 8 miles to the northeast on the opposite side of the valley from the Gore Range and UBCR.

UBCR is located between the Blue River fault and a fault splay to the northeast, approximately 1-mile from both mapped fault traces. An excerpt from the *Geologic Map of the Eastern Half of the Vail 30' x60' Quadrangle, Eagle, Summit, and Grand Counties, Colorado* (USGS, 2011) is shown in **Figure 1** and demonstrates the proximity to the faults.

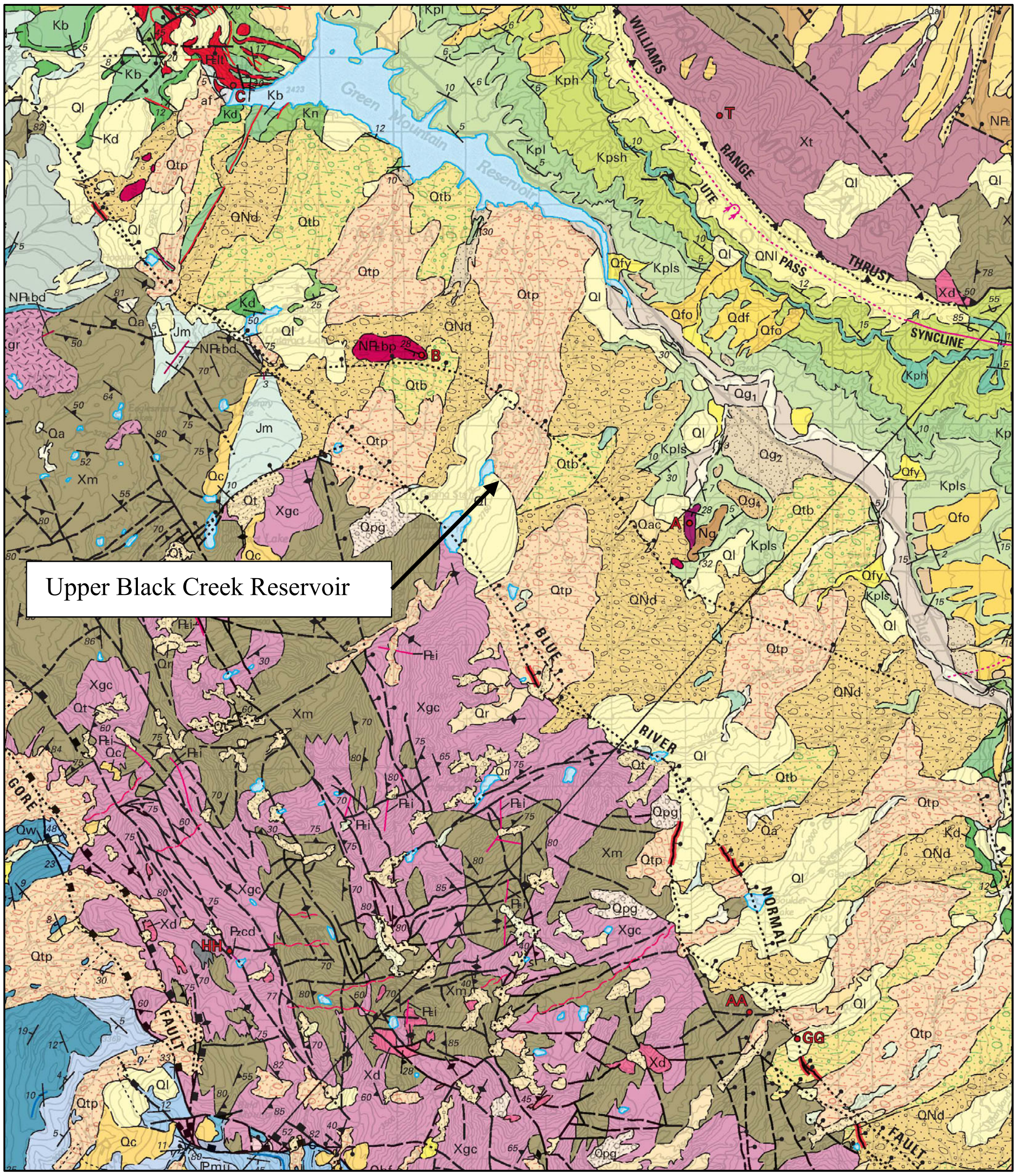

Figure 1. Excerpt of USGS geologic map showing position of UBCR in relation to the Blue River Normal Fault and the fault splay to the northeast.

2.1.1 *Design Seismic Accelerations*

Seismic loads used in the design of the replacement spillway were generated using the online USGS Unified Hazard tool and hazard curves developed for the National Seismic Hazard Mapping Project. An earthquake with a 1/10,000 Annual Exceedance Probability was selected as the design earthquake, which corresponds with a peak ground motion of 0.497g. Output from the Unified Hazard Tool is contained in **Appendix C.1**, which shows the hazard curves, uniform response spectra, and deaggregated data.

2.2 Hydrology

The UBCR is approximately 0.5 miles downstream from Black Lake; Black Creek conveys the discharge from Black Lake into the reservoir. Black Lake has a drainage area that is approximately 14.3 square miles. Black Lake is impounded by Black Lake Dam, which is a fixed crest rock crib located at the east end of the lake that also acts as the spillway. UBCR has a total drainage area of approximately 15.2 square miles and is impounded by the UBCR Dam. According to the current hazard categories provided by the SEO, UBCR Dam is classified as a Significant Hazard dam (SEO, 2020). The dam also should classify with a Hydrologic Hazard as "Significant" based on the fact that no life loss potential is expected to occur during a failure of the dam. The inflow design storm (IDF) for a Significant Hydraulic Hazard Dam is the 0.1% Annual Exceedence Probability (AEP) or the 1 in 1000 year event (SEO, 2020). Previous spillway sizing studies were based on the previous SEO Rules, requiring the spillway to be sized for 45 percent of the Probably Maximum Precipitation (PMP). GEI submitted a hydrology report to the SEO in July 2019, and the report was accepted in September 2019.

GEI developed and evaluated three rainfall events (2-hour Local Storm, 6-hour Local Storm, and 48-hour MLC General Storm) to determine the critical IDF for the UBCR Project. All storms evaluated were developed in accordance with the current SEO Regulations (SEO, 2020). The evaluation was accomplished by identifying the properties of key components necessary to developing the IDF for UBCR, including: basin delineation; design rainfall; rainfall loss rates; baseflow; unit hydrographs; channel routing; and reservoir routing.

The total watershed for the UBCR was modeled with two sub-basins, Upper Black Creek Basin and Lower Black Creek Basin. The basin boundaries were delineated and physical basin parameters were calculated using ESRI GIS software and 10-meter resolution United States Geological Survey (USGS) Digital Elevation Models (DEMs). GEI utilized the Colorado-New Mexico Regional Extreme Precipitation Study Precipitation Frequency (PF) tool to develop rainfall data for the contributing watershed. The SEO provided GEI with three different 1,000-year design storm rainfall depths and temporal rainfall distributions to assist in evaluating the IDF for the Project.

After review of the UBCR watersheds' unit hydrographs provided by the SEO, GEI observed that the total runoff volumes were consistently less than the recommended ratio of 1.0 and developed two new unit hydrographs using the United States Bureau of Reclamation (USBR) synthetic unit hydrograph method in accordance with the SEO Regulations. Two USBR Rocky Mountain region synthetic unit hydrographs were developed and included a general-storm unit hydrograph and a local-storm (thunderstorm) unit hydrograph. These hydrographs can be viewed in **Appendix B**.

The U.S. Army Corps of Engineers (USACE) HEC-HMS Version 4.2.1 computer model was used to estimate the IDF inflows and outflow hydrograph at Black Lake and UBCR. A 1-

minute time step was selected to model the local-storm and a 5-minute time step was selected to model the general storm IDF. The selected time steps meet standard criteria for adequately defining the peak of the unit hydrograph and the IDF. The various parameters selected for the IDF modeling were generally considered appropriate for the site-specific conditions. GEI developed and evaluated three rainfall events to determine the critical IDF for the Project. All storms evaluated were developed in accordance with the current SEO Regulations. The modeling results for the storms evaluated as part of the study are summarized in **Table 4** to **Table 8**.

Table 4.
Upper Black Creek Sub-Basin Runoff Results

Parameter of Modeling Result	2-hour Local Storm	6-hour MEC Local Storm	48-hour MLC General Storm
Storm Depth (in)	1.84	2.39	5.30
Rainfall Duration (hr)	2	6	48
Losses (in)	1.05	1.33	3.31
Storm Runoff (in)	0.79	1.06	1.99
Storm Runoff Percent (%)	43	44	38
Storm Runoff Volume (ac-ft)	600	810	1,520
Peak Discharge (cfs)	4,910	5,810	930

Table 5.
Lower Black Creek Sub-Basin Runoff Results

Parameter of Modeling Result	2-hour Local Storm	6-hour Local Storm	48-hour MLC General Storm
Storm Depth (in)	1.84	2.39	5.30
Rainfall Duration (hr)	2	6	48
Losses (in)	1.72	2.10	5.12
Storm Runoff (in)	0.12	0.29	0.18
Storm Runoff Percent (%)	6	12	3
Storm Runoff Volume (ac-ft)	5.6	13.9	8.7
Peak Discharge (cfs)	115	315	10

Table 6.
2-Hour Local Storm Reservoir Routing Results

Parameter	Black Lake	UBCR
Initial Water Surface El. (ft)	8,895.2	8,748.0
Peak Inflow (cfs)	4,910	1,415
Storm Runoff Volume (ac-ft)	600	585
Peak Outflow (cfs)	1,415	1,065
Peak Water Surface El. (ft)	8,899.1	8,753.3
Freeboard (ft)	-	2.2

Table 7.
6-Hour Local Storm Reservoir Routing Results

Parameter	Black Lake	UBCR
Initial Water Surface El. (ft)	8,895.2	8,748.0
Peak Inflow (cfs)	5,810	1,940
Storm Runoff Volume (ac-ft)	810	800
Peak Outflow (cfs)	1,935	1,480
Peak Water Surface El. (ft)	8,900.0	8,754.6
Freeboard (ft)	-	0.9

Table 8.
48-Hour General Storm Reservoir Routing Results

Parameter	Black Lake	UBCR
Initial Water Surface El. (ft)	8,895.2	8,748.0
Peak Inflow (cfs)	930	800
Storm Runoff Volume (ac-ft)	1,520	1,525
Peak Outflow (cfs)	795	780
Peak Water Surface El. (ft)	8,897.8	8,752.3
Freeboard (ft)	-	3.2

SEO Regulations require the UBCR to retain a 1-foot residual freeboard during the IDF storm. As shown in **Table 6**, the model results indicate that the 6-hour local storm results in the highest water surface elevation in the UBCR, resulting in less than the required one foot of residual freeboard. Therefore, the 6-hour local storm was selected as the IDF. Based on these results, GEI recommended that the UBCR spillway be reconstructed with the spillway discharge capacity to safely pass the SEO required IDF (1,000-year, 6-hour Local Storm) while providing a minimum of 1.0-foot of freeboard.

2.3 Subsurface and Groundwater Conditions

2.3.1 *Subsurface Conditions*

Subsurface conditions at the UBCR site were developed from results of the 2017 geotechnical exploration conducted by GEI and the associated laboratory testing program. The exploration program included 4 locations. Two borings were advanced using hollow stem augers and rotary drilling methods at the left spillway abutment (B-1) and upstream of the concrete spillway apron (B-2). Two core holes were drilled in the middle third of the spillway chute concrete slab, advancing to a depth of about 2 feet below the bottom of the slab. GEI used a hand auger to collect soil samples at these two slab locations. Bedrock was not encountered in any of the boreholes to the termination depths.

Borehole B-1 at the left spillway abutment was drilled to a depth of 12 feet before hitting refusal. The material was predominantly granular fill with 15 to 43 percent non- to medium plastic fines, classifying as SC, GC, and SM based on the Unified Soil Classification System (USCS). Cobbles were identified within the soil matrix. Large cobbles were also observed along the edge of the spillway and downstream of the stilling basin. Based on Standard Penetration Test (SPT) blow counts performed during the sampling, the clayey sand was judged to be medium dense. However, the upper 4 feet of fill had zones with lower density clayey sand.

The borehole drilled upstream of the spillway recovered foundation soils consisting of a combination of medium dense to dense, clayey and silty sand (SC, SM) with a large proportion of cobbles. Cobbles up to 6 inches in diameter were observed in the auger cuttings during drilling. Similar material was extracted from the two hand auger boreholes within the spillway chute, except more gravel was observed within the soil matrix beneath the spillway chute. Samples retrieved at the two locations consisted of silty sands with gravels to clayey sands with gravels, with cobbles up to 4 inches also found. Zones of clean sand were observed at these locations, which could indicate piping of fine clay and silt particles during the prolonged periods of seepage below the concrete slab. It should be assumed that cobbles exist throughout the spillway backfill and native foundation soils.

2.3.2 *Groundwater Conditions*

Groundwater was encountered while drilling the borehole upstream from the spillway concrete apron during the GEI geotechnical investigation. Water level depth was reported to be 16.0 feet below the ground surface. No groundwater was encountered at the boring near the left abutment of the spillway. However, this hole was only advanced to a depth of 12 feet below the ground surface before auger refusal was encountered.

The groundwater level measurement represents conditions at the time and location of the investigation. Fluctuations in groundwater levels should be expected seasonally and annually due to variations in precipitation, evaporation, and ground surface runoff.

According to the United States Department of Agriculture (USDA) Custom Soil Resource Report prepared for the area of Upper Black Creek, the five types of soil delineated on the soil map for the area all report a depth to water table of more than 80 inches.

2.4 Site Survey

GEI performed field mapping on July 19, 2018, of the Upper Black Creek Dam, reservoir and surrounding area using Unmanned Aerial System (UAS) flights that captured geo-oriented photography. The geo-oriented photographs were used to generate photogrammetric models that enabled the formation of dam face orthoplanes, topographic contours, and elevation models of the existing conditions. All UAS flights were performed in accordance with Federal Aviation Administration (FAA) regulations by a licensed remote pilot. Multiple automated and manual flights were necessary to obtain the required photographs to generate necessary data for analyses.

A total of 20 ground control points (GCPs), visually discernable marks on the ground made with marking paint were established at the time of the aerial survey. The GCPs assisted with rectifying the photogrammetric models and provided reference benchmarks that assisted with scaling, positioning, and orienting the models.

A total of 5 separate flights were made with the UAS. DroneDeploy application software was used for 3 of the flights to automatically perform pre-planned flight grids over the area. Two manual flights were conducted to capture oblique imagery of the dam, spillway, and surrounding area upstream and downstream of the dam. The manual flights assisted with three-dimensional (3D) modelling and enabled access to areas difficult to image from a nadir perspective (directly below the UAS). The automated flights paths occurred at elevations between 150 and 250 feet above ground level, and the manual flights included lower elevation flights to obtain photos to be used to assist with wetland delineation. The average ground sampling distance (GSD) of the 3D model is 1.73 cm. The GSD is the distance between two successive pixel centers measured on the ground surface.

After the site visit, GEI uploaded 1,148 photos covering an area of approximately 44 acres of the dam and surrounding area into Pix4D software for the generation of photogrammetric models. Pix4D uses matching keypoints in overlapping photographs to tie topographic information together and develop a three-dimensional (3D) mosaic by using geo-oriented photographs. The photogrammetric 3D model was used to generate orthomosaic images in addition to elevation contours, and elevation models which were exported for use in AutoCAD Civil3D. These elevation models were used to develop representative site topography.

Site elevations were adjusted based on a spillway crest El. 8748.0 so that previous studies and documentation would reference the same vertical elevation datum. A bathymetric survey was completed by Resource Engineering in 2007. This survey was also completed with a referenced "zero" elevation at the spillway crest. GEI merged bathymetric contour data with the UAV contour data to create a composite site contour map. The UAV survey was completed at a lower water level than the bathymetric survey, resulting in overlapping contours between the two surfaces. GEI manually adjusted the contours within the overlapping area so that contour lines did not cross, with a greater level of accuracy assumed for the elevation data from the UAV survey.

Three control points were set in the vicinity of the dam to facilitate layout of the work. These include a chiseled and painted "X" on a large boulder to the west of Black Creek Road near the dam crest, rebar at the outlet works valve house, and rebar to the south of the spillway.

3. Design Criteria

3.1 General

UBCR Dam is a jurisdictional dam (CO Dam ID 360127) subject to regulatory authority of the SEO. Design of the Project must conform to applicable SEO statutes pertaining to dams and appurtenant structures. The primary requirements that will govern the design of the Project are the current SEO Regulations. Additional design guidance was employed in the design, including:

- Reclamation Design Standards No. 14: Appurtenant Structures for Dams (Spillway and Outlet Works); and
- Corps Engineer Manuals (EM), Engineer Regulations (ER), and Engineer Technical Letters (ETL)

3.1.1 *Applicable Codes and Standards*

In addition to the SEO statutes and the design standards listed in Section 3.1, the following codes, standards, and specifications are included as part of the overall Project design criteria. The applicable version of each document was the latest edition in force at the time the Project design was originally authorized, unless noted otherwise. References to specific codes and standards will be included in the applicable specifications provided within the construction drawings.

The civil and structural design, engineering, materials, equipment, and construction will conform to the applicable specified codes and standards of the following organizations:

- ACI American Concrete Institute
- AISC American Institute of Steel Construction
- ANSI American National Standards Institute
- ASCE American Society of Civil Engineers
- ASTM ASTM International
- AWWA American Water Works Association
- ICC International Code Council

3.2 Hazard Classification

SEO Rule 4.13 states the “Hazard Classification” falls into one of four categories based on the hazard potential derived from an evaluation of sunny day failure of the dam. The four hazard categories include:

- High Hazard: a dam for which loss of human life is expected to result from failure of the dam.
- Significant Hazard: A dam for which significant damage, but no life loss is expected to result from failure of the dam.
- Low Hazard: A dam for which neither life loss nor significant damage as defined for a Significant Hazard dam are expected to result from failure of the dam.
- No Public Hazard (NPH): A dam for which minimal damage, with no life loss, is expected to result from failure of the dam.

The SEO classifies the UBCR Dam as a "Significant Hazard" structure.

3.3 Hydrologic Hazard

The spillway sizing is driven by the "Hydrologic Hazard" of the dam, which is defined in Rule 4.15 by the consequences of dam failure due to an overtopping event. The Hydrologic Hazards are ranked as follows:

- Extreme: Life loss potential of 1 or more.
- High: Life Loss potential of less than 1.
- Significant: No life loss potential but significant damage is expected to occur.
- Low: No life loss potential or significant damage is expected to occur.

The UBCR Dam was judged to be a Significant Hydrologic Hazard structure, requiring IDF to be the 0.1% AEP, per Rule 7.2.1.

3.4 Seismic Design Criteria

Performance criteria of Upper Black Creek Dam and Reservoir is in general conformance with rule 7.6 of the SEO *Rules and Regulations for Dam Safety and Dam Construction*.

The seismic design criteria for the spillway follows the guidelines established in the US Army Corps of Engineers manual EM 1110-2-2104 - *Strength Design for Reinforced Concrete Hydraulic Structures* for strength and serviceability criteria and EM 1110-2-2100 – *Stability Analysis of Concrete Structures* for stability criteria. The coefficient of dynamic earth pressure was developed using *Seismic Earth Pressures on Retaining Structures with Cohesive Backfills* (Agusti, 2013).

3.5 Operational Criteria

The spillway is designed to convey all inflows, up to the IDF with at least 1-foot of freeboard. Operational design criteria for the spillway are summarized below:

- Spillway must operate without the need to adjust gates or valves
- The spillway must have an underdrain system that is accessible for cleaning and closed-circuit television (CCTV) inspection when necessary

3.6 Material Properties

Materials that will be used or need to be considered for the Project include embankment fill, filter sand, drain gravel, PVC piping, concrete, reinforcing steel, sheet pile, and riprap.

3.6.1 *Soil*

For project design, material properties were generally developed through laboratory testing of collected samples, empirical correlations, and engineering judgement.

3.6.1.1 Strength Properties

Material properties used for the slope stability analyses include the unit weight, drained shear strength (φ', c'), and undrained shear strength (φ, c). Materials modeled in the analyses include the silty/clayey sand foundation soils, embankment fill, filter materials, and concrete.

Embankment fill was assumed to consist of stockpiled and re-worked silty/clayey sand excavated during construction. The embankment fill will be placed and compacted in lifts. The unit weight for the embankment fill and silty/clayey sand were estimated to correlate with mixed-grained, medium dense to dense sand using published correlations. Drained shear strength parameters for the silty/clayey sand were based on correlations between N-values and friction angle. The embankment fill was assumed to have the same strength properties as the silty/clayey sand. The silty/clayey sand and embankment fill consist of predominately sand and have low to non-plastic fines; therefore, we have assumed the undrained strengths are the same as the drained strengths.

Filter material properties (filter sand and drain gravel) were assumed based on our experience on other projects. The concrete spillway was modeled with a high strength to prevent slip surface failures through the concrete. Error! Reference source not found.**9** summarizes assumed material strength parameters used for design. Material properties are included in **Appendix C.2.**

Table 9.
Soil Fill Material Strength Parameters for Design

Materials	Unit Weight (pcf)	Drained Shear Strength		Undrained Shear Strength	
		Friction Angle ϕ' (degrees)	Cohesion c' (psf)	Friction Angle ϕ (degrees)	Cohesion c (psf)
Embankment Fill	130	35	0	35	0
Silty Clayey Sand	130	35	0	35	0
Filter Material	125	32	0	32	0
Concrete	150	0	10,000	0	10,000

3.6.1.2 Hydraulic Conductivity

Material properties used for the seepage analyses include the horizontal hydraulic conductivity (Kx), the vertical hydraulic conductivity (Ky), and the anisotropy ratio (the ratio of Kx to Ky). Materials modeled in the seepage analyses include the silty/clayey sand foundation soil, embankment fill, filter materials, concrete, and the sheet pile. Hydraulic conductivities and anisotropy ratios for the embankment fill and silty/clayey sand were estimated based on industry accepted published correlations for similar materials and engineering judgment.

Hydraulic conductivity for the sheet pile was calculated assuming the sheet pile has water sealing joints. From manufacturer data, the permeability of the joints ranges from 3.8×10^{-8} to 1×10^{-10} cm/sec. We assumed a sheet pile panel width of about 2 ft and calculated the equivalent hydraulic conductivity based on a modeled sheet pile thickness of 0.375-inch. The hydraulic conductivity of the concrete was assumed based on new concrete with waterstop joints. Error! Reference source not found.**10** summarizes assumed material hydraulic conductivity parameters used for design. Material properties are included in **Appendix C.2.**

Table 10.
Soil Material Hydraulic Conductivity Parameters for Design

Material Type	Horizontal Hydraulic Conductivity, K_x (cm/sec)	Horizontal Hydraulic Conductivity, K_x (ft/sec)	Anisotropy Ratio (K_x/K_y)
Silty/Clayey Sand	1.00×10^{-5}	3.28×10^{-7}	10
Embankment Fill	1.00×10^{-5}	3.28×10^{-7}	4
Filter Material	1.00×10^{-2}	3.28×10^{-4}	4
Sheet Pile	4.75×10^{-10}	1.56×10^{-11}	1
Concrete	1.00×10^{-7}	3.28×10^{-9}	1

3.6.2 *Concrete*

The concrete mix specified for the spillway was designed for several conditions beyond structural loading requirements. Design conditions including weather (freeze-thaw, hot weather concreting, cold weather concreting) and hydraulic flow conditions (velocity and duration) were considered.

The recommended concrete design will have a 4,500 pounds per square inch (psi) minimum compressive strength, a maximum water/cement ratio of 0.45, and a unit weight of 150 pounds per cubic foot (pcf). The concrete properties are based on empirical relationships and equations presented in ACI 318.

3.6.3 *Other Material Properties*

Other materials that will be used or need to be considered for the Project include reinforcing steel and riprap.

3.6.3.1 Reinforcing Steel Properties

All reinforcing steel will be standard ASTM A615 Grade 60 reinforcement, with minimum yield stress of 60,000 psi and minimum tensile stress of 90,000 psi.

3.6.3.2 Riprap Properties

The riprap used on the project will be specified to meet the requirements of Colorado Department of Transportation Standard Specifications for Road and Bridge Construction, Section 506: Riprap.

3.6.3.3 Sheet Pile Properties

Steel sheet pile sections were selected based on drivability. Steel sheet piles shall conform to ASTM A328 and shall have minimum yield strength of 50,000 psi. Sheet pile joints will be treated to increase the water-tightness of the joint. The sheet piling has been included as an additional protective measure against seepage under the spillway structure. However, sheet piles do have a finite life, which can vary due to corrosivity of the soil and groundwater among other things. The service life for the sheet piling is assumed to be 50 years. Replacement of the sheet pile barrier may be required after this point, depending on the performance of the structure.

4. Feasibility Alternatives

GEI considered 3 alternatives for modifications to the dam and spillway to meet the SEO flood routing criteria and reduce overall risk of dam or spillway failure.

4.1 Alternative 1: Repair Existing Spillway

This alternative consists of selective demolition of the existing spillway slab, installing underdrains below the concrete slab, and repairing the existing concrete as necessary. **Table 11** summarizes the advantages and disadvantages of this spillway alternative.

Table 11.
Comparison of Advantages and Disadvantages for Alternative 1

Advantages	Disadvantages
• Excavations can be minimized by utilizing the existing spillway walls to form the structure. • The overall cost of repairing the structure may be less than a full replacement.	• The existing spillway is not wide enough to route the IDF with 1-foot of freeboard. A dam raise would be required to meet freeboard criteria. • The cost for repairing an existing structure would be a high percentage of the cost to replace the structure. • Life expectancy for the repairs should be assumed to be 10 to 20 years. • A dam raise would likely require additional geotechnical investigation and analysis which could result in potential modifications to the dam. • There is additional risk of additional construction requirements due to unknowns with the existing spillway.

Alternative 1 was not developed because a repaired spillway could not route the IDF and meet SEO freeboard criteria without raising the dam. A significant dam raise will require additional geotechnical exploration on the dam and stability analysis of the raised embankment.

4.2 Alternative 2: Replace Existing Spillway (No dam raise)

This alternative consists of demolishing the existing spillway structure and constructing a new reinforced concrete spillway with a 38-foot-wide crest. The spillway chute would converge down to 32 feet wide. A new underdrain system would be constructed under the slab and adjacent to the spillway walls. A sheet pile cutoff would be constructed on the upstream side of the structure to reduce the potential for under-seepage. **Table 12** summarizes the advantages and disadvantages of this spillway alternative.

Table 12.
Comparison of Advantages and Disadvantages for Alternative 2

Advantages	Disadvantages
• Spillway modifications can be made with only nominal crest grading on the dam to maintain the design crest elevation (El. 8755.5). • The entire spillway would be reconstructed with a design life of 50 years or greater. • Underdrain provisions and "state-of-the-practice" construction would provide for safe spillway operation at all design flows. • This alternative was presented to the US Army Corps of Engineers for permitting and has been accepted as a Nation Wide Permit No. 3.	• This project cost is potentially the highest cost of all three alternatives. • The sheet-pile cutoff is susceptible to corrosion over the life of the structure and may require replacement. A 50-year life span is estimated for the cutoff.

Alternative 2 was fully developed. This alternative meets SEO design criteria and provides a robust structure with low maintenance and a long life.

4.3 Alternative 3: Replace Existing Spillway (1.0 foot dam raise)

This alternative consists of demolishing the existing spillway structure and constructing a new reinforced concrete spillway with a 32-foot-wide crest. The bottom width of the stilling basin would still be required to be 32 feet for energy dissipation. A new underdrain system would be constructed under the slab and adjacent to the spillway walls. A sheet pile cutoff would be constructed on the upstream side of the structure to reduce the potential for under-seepage. The dam crest would require a nominal 1.0 foot raise above the existing design crest elevation to achieve the required flood routing capacity. **Table 13** summarizes the advantages and disadvantages of this spillway alternative.

Table 13.
Comparison of Advantages and Disadvantages for Alternative 3

Advantages	Disadvantages
• The entire spillway would be reconstructed with a design life of 50 years or greater. • The reduction in spillway crest width compared to Alternative 2 would result in approximately 22 cubic yards less concrete. • Underdrain provisions and "state-of-the-practice" construction would provide for safe spillway operation at all design flows.	• This project cost is similar to Alternative 2. • An additional geotechnical investigation • The sheet-pile cutoff is susceptible to corrosion over the life of the structure and may require replacement. A 50-year life span is estimated for the cutoff. • A dam raise would likely require an additional geotechnical investigation and additional stability analysis for the which could result in potential modifications to the dam.

Pricing for Alternative 3 was developed as a scaled factor of Alternative 2. This alternative would meet SEO design criteria for spillway routing and provides a robust structure with low maintenance and a long life. However, there is a risk in modifying the existing dam.

5. Design of Spillway

5.1 General Description

The existing 28-foot-wide spillway at UBCR is insufficient to safely pass both the previous and current SEO required IDF while also providing a minimum of 1.0-foot of freeboard. The spillway flood routing criteria could be met with spillway improvements with the existing configurations along with a minor dam crest raise. However, based on the results of the Hydrology Study (GEI, 2019), GEI developed a new spillway configuration which would allow the spillway at UBCR to safely pass the IDF for a Significant Hazard dam classification. Constructing a new replacement spillway at UBCR was determined to be the most economic approach to accomplish this.

The designed configuration of the replacement spillway is similar to that of the existing spillway, with a concrete chute, an approach channel, concrete weir, and stilling basin. The invert elevation of the spillway is at the existing normal pool El. 8748.0. The concrete chute of the replacement spillway contracts in width along its length; it is 38 feet wide on the upstream side of the spillway, with a 15:1 contraction along the inclined portion of the training walls, resulting in a 32-foot-wide concrete chute on the downstream side. The designed training walls vary in height along the length of the spillway, with the maximum height being 13 feet and the minimum height being 9.5 feet. A typical plan and profile view of the replacement spillway is shown in UBCR Spillway Replacement Drawing 10.

5.2 Hydraulic Analysis and Design

5.2.1 *Outlet Works*

The existing low-level outlet works will remain unchanged during the spillway reconstruction project. The low-level outlet works consists of a 24-inch-diameter pipe at invert El. 8720.0 and is used to control the normal pool at El. 8748.0. In 2018, the SEO determined that the discharge capacity of the low-level outlet works at normal pool was approximately 65 cfs.

5.2.2 *Spillway Structure*

GEI developed a proposed spillway configuration to safely pass the significant hazard 6-hour local-storm IDF while providing a minimum of 1.0-foot of freeboard below the dam crest El. 8755.5. The proposed spillway structure will be an uncontrolled 38-foot wide broad crested concrete weir at invert El. 8748.0 that discharges through a (2H:1V) spillway chute and United States Bureau of Reclamation (USBR) Type III stilling basin. The rectangular spillway chute width transitions from 38 feet wide at the crest to 32 feet wide at the stilling basin using a maximum 1:15 (horizontal to longitudinal) wall flare as recommended in the

USACE Engineering Manual EM 1110-2-1601 (USACE, 1994). The left and right spillway training walls retain the embankment fill. The spillway walls vary in height from a minimum of 8 feet at the weir, 9.5 feet in the spillway chute, and 13 feet in the stilling basin. The following sub-sections summarize the hydraulic calculations and modeling used to determine the spillway structure geometry.

5.2.2.1 Spillway Discharge Rating Curve

The proposed spillway discharge capacity was evaluated using the USACE Hydrologic Engineering Center – River Analysis System (HEC-RAS) Version 5.0.7. The HEC-RAS model was developed to estimate the reservoir elevations for various discharges through the spillway. The spillway geometry was modeled using a series of rectangular cross sections with a bottom width ranging from 38- to 50-feet, an upstream invert El. 8748.0 and stilling basin floor El. 8724.0. Cross sections were placed along the spillway chute at 1 foot intervals in order to model the transition from subcritical to supercritical flow. Several cross sections were placed within the stilling basin in order to estimate where the hydraulic jump would occur. A Manning's n value of 0.05 was used in the stilling basin to account for the additional energy loss due to the rows of chute blocks and baffle piers that exist in the stilling basin but were not added into the HEC-RAS cross sections. In all other cross sections within the spillway the Manning's n-values were set to 0.03.

The spillway discharge coefficient was determined using the procedures described in the United States Geologic Survey (USGS) *Circular 397 Discharge Characteristics of Broad Crested Weirs* (USGS, 1957) assuming an upstream and downstream slope equal to 2H:1V. The computed spillway discharge rating curve for the proposed spillway is provided in **Figure B.3** in **Appendix B**. A summary of HEC-RAS Model setup and results is provided in **Appendix B**.

The spillway rating curve was then input into HEC-HMS described in Section xx.0 as the primary spillway to determine final routing results. The significant hazard 6-hour local-storm IDF results in a peak inflow into UBCR of 2,160 cfs, a peak water surface El. 8,754.4 , and a total peak discharge of 1,895 cfs. During this flood, the residual freeboard below the dam crest is approximately 1.1 feet. Modeling results for the 6-hour local storm are summarized in Table 1. The proposed IDF hydrographs are included in **Figure B.4** in **Appendix B**.

Table 14.
6-hour Local Storm Reservoir Routing Results

Parameter	Black Lake	UBCR
Initial Water Surface El. (ft)	8,890.2	8,748.2
Peak Inflow (cfs)	6,990	2,160
Peak Outflow (cfs)	2,070	1,895
Peak Water Surface El. (ft)	8,895.2	8,754.4
Freeboard (ft)	-	1.1

5.2.2.2 Spillway Training Walls Freeboard

Flow rates ranging from 100 to 2,000 cfs were routed through the HEC-RAS model to check the freeboard below the spillway chute and stilling basin training walls for various discharges up to the IDF. The freeboard check accounts for wave action, air bulking, splash and spray above the HEC-RAS calculated water surface elevations. We developed a spreadsheet tool to estimate the required minimum freeboard using the USBR Design Standard No. 14 *Appurtenant Structures for Dams (Spillways and Outlet Works)*. **Figure B.5** in **Appendix B** illustrates that the IDF discharge of approximately 1,900 cfs results in a minimum freeboard of approximately 2.4 feet.

5.2.2.3 Stilling Basin Hydraulic Analysis

Flow rates ranging from 100 to 2,000 cfs were also routed through the HEC-RAS model to determine the discharge, flow depths and velocities within the stilling basin and downstream tailwater. The maximum depth, velocity and Froude number in the spillway chute upstream of the stilling basin were 1.8 ft, 34.0 ft/s and 5.6 respectively. These values were used as input into a spreadsheet tool based on the methods described in the USBR *Hydraulic Design of Stilling Basins and Energy Dissipators – EM No. 25* (USBR 1984) to determine the stilling basin geometry and analyze performance.

The selected stilling basin structure is a horizontal basin style Type III basin (USBR, 1978). Type III basins are shallow concrete basins with chute blocks at the interface of the spillway chute and stilling basin floor, followed by baffle piers and sill at the downstream end. These basins perform well for velocities up to 50 ft/sec and Froude numbers above 4.0. A summary of the stilling basin design parameters is provided in Table 2. A summary of stilling basin design calculations is provided in **Appendix B**.

Table 15.
Summary of Stilling Basin Design Parameters

Stilling Basin Parameters	
Maximum Froude #,Fr_1	5.6
Maximum Upstream Depth, D_1 (ft)	1.8
Maximum Upstream Velocity, V_1 (ft/sec)	34.0
Maximum Downstream Depth, D_2 (ft)	10.5
Ratio L/D_2 (Figure 12 EM-25)	2.4
Basin Length, L (ft)	26.0

To prevent retention of stagnant water within the basin, several slots will be formed in the end sill to allow water to drain following a rainfall event. Flow velocity at the end of the stilling basin is estimated to be about 8 ft/sec. Protection against undermining of the stilling basin during the IDF is provided by a riprap apron and concrete cutoff wall beneath the end sill which is keyed into the foundation layer. We used the methods outlined in the USACE Engineering Manual EM 1110-2-1601 (USACE, 1994) to evaluate the riprap size required to protect the stilling basin against undermining. Based on Plates B-29 and B-30, Stone Stability, Velocity vs. Stone Diameter for High Turbulence Stilling basins, riprap with a D_{50} of 1.0 feet would be expected to resist erosive forces from velocities between 8 and 10 ft/s. The riprap apron extends the full width of the stilling basin and 25feet downstream. The riprap will be sloped from the end of the dentated sill at El. 8724.0 and slope down to match existing grade at approximate El. 8723.5. A 2-foot-thick layer of CDOT Moderately Heavy riprap (D_{50} = 12 inches, D_{max} = 24 inches) will be placed over riprap bedding.

5.3 Geotechnical Analysis and Design

5.3.1 *Seepage and Stability Analyses*

5.3.1.1 Seepage Analysis

A two-dimensional steady-state seepage analysis was performed to model the phreatic surface along the proposed spillway. The seepage analysis was performed using the finite element program SEEP/W, GeoStudio 2019 by Geo-Slope, International.

The seepage analysis was performed along the spillway centerline and includes the proposed sheet pile cutoff and three drainage pipes located at about 10-foot vertical intervals beneath the spillway. The sheet pile was modeled extending to about El. 8713.

The steady-state seepage loading condition represents the long-term stability of the spillway under normal reservoir pool steady-state seepage conditions. The reservoir was modeled at El. 8478, and the downstream head elevation was modeled at El. 8724, which is the downstream ground surface elevation.

5.3.1.2 Stability Analyses

Stability analyses were performed using SLOPE/W, GeoStudio 2019 by Geo-Slope, International. The Spencer method, which satisfies both force and moment equilibrium, was used in the analyses.

The spillway stability was analyzed for the steady-state seepage condition with normal reservoir water surface El. 8748. Rapid drawdown was not considered because the slope is relatively flat upstream of the spillway control structure. A pseudo-static analysis was also performed to estimate the yield acceleration. The yield acceleration was evaluated from a slope stability analysis using the spillway geometry and the selected shear strengths of the foundation and fill materials. The yield acceleration is the horizontal acceleration at which the critical failure mass through the embankment has a factor of safety of 1.0. The yield acceleration was utilized in the seismic deformation analysis discussed in Section 5.3.2.

5.3.1.3 Results

Seepage and stability results are presented in **Appendix C.3**. The steady-state stability analysis indicates a calculated factor of safety of 1.9, which is greater than the minimum factor of safety of 1.5 required by the SEO.

The yield acceleration was calculated to be 0.235g, which is less than the PGA of 0.5g for the 10,000-year return interval; therefore, a deformation analysis was performed as discussed in Section 5.3.2.

5.3.2 *Seismic Deformations*

The seismic deformation of the spillway was estimated using the empirical method developed by Makdisi & Seed (1978). This method is commonly used for simplified analyses of seismic deformations. The seismic deformation analysis calculations and supporting information are presented in **Appendix C.4.**

The analysis is based on the Newmark sliding block analogy, and the input parameters include the earthquake magnitude, firm rock peak ground acceleration (PGA), PGA at the base of the spillway, PGA at the crest of the spillway, yield acceleration, and the ratio of the height of the failure mass and the embankment height. Permanent deformation generally

Based on the calculations, seismic deformation of the spillway was considered to be negligible.

5.3.3 *Filter Compatibility*

The USDA, National Engineering Handbook, Chapter 26 methodology was used for the filter compatibility analysis to select the materials for the filter diaphragms. The USDA method involves developing a set of minimum and maximum control points which are calculated based upon the gradation of the base soil. These control points represent limits for the gradations of possible filter materials. A material is considered adequate for use in filtering the base soil if the filter's gradation is located within the band developed between the maximum and minimum control points.

The goal of the filter compatibility analyses was to develop a gradation band that meets the USDA filter requirements, is coarse enough to reduce the amount of filter layers required between the foundation and filter sand and can be efficiently produced from nearby suppliers. Required maximum and minimum filter control points were developed using the USDA guidance. A gradation band for the filter material was developed using the coarse (maximum) particle control points as a guide for the coarse side of the filter band, and criteria for prevention of gap grading for the fine side of the filter band. This allows for the development of the coarsest gradation band that meets filtering criteria so the amount of filter layers can be minimized. Once gradation limits were established, filter material compatible with the gradation limits were selected for design.

The gradation band for the filter sand developed using the USDA methodology was compared to ASTM C-33 filter sand and was determined to be acceptable. Filter compatibility calculations and gradation curves are included in **Appendix C.5**.

5.3.4 *Underdrain System*

An underdrain system will be installed in three locations below the spillway to reduce uplift pressures on the spillway slab and capture any seepage through the foundation. The underdrain system will be installed adjacent to each of the three keyed transverse construction joints. The underdrain system is designed as a two-stage filter system consisting of a 6-inch diameter slotted PVC pipe, bedded in a 6-inch layer of drainage gravel and surrounded by a 6-inch layer of filter sand. Lateral spacing of the underdrain pipes is designed to be spaced approximately 20 feet apart.

In addition to the spillway slab underdrains, drains will be constructed above the spillway footings to reduce external hydrostatic loading on the spillway walls.

Filter compatibility was evaluated between the native soil and filter sand and between the filter sand and drainage gravel. Based on filter compatibility evaluations, ASTM C33 fine aggregate (concrete sand) is recommended for use as the filter sand and No. 89 coarse aggregate (pea gravel) is recommended to be used as the drainage gravel.

The underdrain system plan and details are shown on UBCR Spillway Replacement Drawing 10 and 12, respectively. Filter compatibility calculations are contained in **Appendix C.5**.

5.4 Structural Analysis and Modeling

The replacement spillway at UBCR was separated into two components for design: the concrete slab and the training walls. Both components were designed for two controlling locations: the stilling basin, which is the location of the tallest wall section of the spillway, and the inclined portion of the spillway, which is the location of the shortest wall section of the spillway.

Reinforced concrete elements were designed in accordance with the requirements of the American Concrete Institute's *Building Code Requirements for Structural Concrete* [ACI 318-14] and *Code Requirements for Environmental Engineering Concrete Structures* [ACI 350-06]. Additional guidance was provided from the US Army Corps of Engineers' *Strength Design of Reinforced Concrete Hydraulic Structures* (USACE, 2003) and *Engineering and Design: Retaining and Flood Walls* (USACE, 1989). Structural stability loading conditions were analyzed in accordance with the American Society of Civil Engineer's *Minimum Design Loads for Buildings and Other Structures* [ASCE 7-10], US Army Corps of Engineers' *Strength Design of Reinforced Concrete Hydraulic Structures* (USACE, 2003) Table E-1, and G.C. Agusti and Nicholas Sitar's *Seismic Earth Pressures on Retaining Structures in Cohesive Soils* (UCB GT 12-02, 2013).

Structural calculations for the concrete slab and training walls of the replacement spillway are provided in **Appendix C**.

5.4.1 *Design of Spillway Slab*

The spillway concrete slab was modeled as a simple beam assumed to be pinned at a distance of 5 feet. It was assumed that the minimal unsupported length of the slab is 5 feet, which would allow for some erosion under the spillway slab. Shear and moment were checked across the slab to determine the maximum shear and moment load on the slab. Temperature and shrinkage steel controlled the design, and #7 bars at 12 inch spacing were determined to be sufficient for both controlling locations along the spillway.

Thickness of the slab was designed to be 1 foot at the inclined portion and 2 feet at the stilling basin portion of the spillway. The thickness was increased at the stilling basin to provide more support for the design loading conditions on the spillway training wall at that location.

5.4.2 *Design of Spillway Training Walls*

The spillway walls were designed as cantilevered retaining walls with soil and seismic loads. The strength/capacity checks were performed using the one-foot strip. Two load cases were analyzed for the spillway wall: a usual condition and an extreme condition. USACE and ACI strength/service load combinations were applied to develop the loading conditions. The spillway wall stem was analyzed as a cantilever beam with fixity provided at the footing. It was assumed that the groundwater table is below the wall section for both loading conditions. For Load Case 1, or the usual condition, the cantilever beam is subject to lateral forces due to at-rest earth forces from fill and construction live loads. For Load Case 2, or the extreme condition, the cantilever beam is subject to lateral forces due to seismic inertial forces due to self-weight and dynamic fill loads. Diagrams of the load cases are available in the structural calculations presented in **Appendix C**. Refer to UBCR Replacement Spillway Drawings 10 and 12 for a plan and profile view of the spillway walls and details of the walls, respectively.

5.4.2.1 Global Stability

A conventional stability analysis of the proposed spillway walls was performed for both load cases at both controlling locations analyzed in the design. Both overturning and sliding stability were considered, but it was assumed that the spillway was constrained from transverse sliding due to the connection with the slab and wall on the opposite side.

5.4.2.2 Strength and Serviceability Design

The spillway walls were designed for strength and serviceability using the loads, load cases, and load combination guidelines developed as per ASCE 7-10, USACE EM 1110-2-2104, ACI 318-14, and ACI 350-06.

5.4.2.2.1 Inclined Portion of the Spillway Wall

Based on the analysis, the spillway wall along the inclined portion will have a footing thickness of 2 feet and will extend 3.5 feet into the spillway chute, where it ties into the 1-foot-thick concrete chute slab. The heel will be 3 feet long, for a total footing length of 8 feet. This section of the training wall will be 9.5 feet tall, and support fill material beside the channel with 1 foot gap between the top of the wall and the top of the fill. The spillway wall along the inclined portion is 1 foot thick at the top of the wall, and 1.5 feet thick at the bottom of the wall.

The spillway wall and footing will be reinforced for the controlling shear and moment produced from the load cases that were analyzed. The spillway walls will be reinforced vertically with #7 bars at 12 inch spacing and horizontally with #5 bars at 12 inch spacing, with 3 inch clear cover on all sides. The footing will be reinforced with #9 bars at 12 inch spacing on both faces perpendicular to the spillway wall, and #7 bars at 12 inch spacing on the top and bottom face parallel to the spillway wall. A 3 inch cover will exist on the top of the footing, and 4 inch cover on the bottom.

5.4.2.2.2 Stilling Basin Portion of the Spillway Wall

Based on the analysis, the spillway wall along the stilling basin portion will have a footing thickness of 2 feet and will extend 3.5 feet into the spillway chute, where it ties into the 2-foot-thick concrete chute slab. The heel will be 3 feet long, for a total footing length of 8 feet. This section of the training wall will be 13 feet tall, and support fill material beside the channel with a 5 foot gap between the top of the wall and the top of the fill. The spillway wall along the stilling basin is 1 foot thick at the top of the wall, and 1.5 feet thick at the bottom of the wall.

The spillway wall and footing will be reinforced for the controlling shear and moment produced from the load cases that were analyzed. The spillway walls will be reinforced vertically with #7 bars at 12-inch-spacing and horizontally with #5 bars at 12 inch spacing, with 3 inch clear cover on all sides. The footing will be reinforced with #9 bars at 12 inch spacing on both faces perpendicular to the spillway wall, and #7 bars at 12 inch spacing on the top and bottom face parallel to the spillway wall. A 3 inch cover will exist on the top of the footing, and 4 inch cover on the bottom.

5.5 Outlet Channel Protection

Protection against erosion at the outlet and undermining of the spillway is provided by a stilling basin and installation of a riprap channel downstream of the stilling basin. The 26-foot-long stilling basin is equipped with chute blocks, baffle piers, and a sloped endsill. These features reduce the energy of the flow discharging from the outlet of the spillway and allow water to exit into the outlet channel at a reduced velocity. Protection against erosion of the outlet channel from water exiting the stilling basin is provided by a 2-foot-thick layer of CDOT Type M riprap placed on a 6-inch-thick layer of bedding stone. This riprap channel continues at El. 8724.0 for approximately 25 feet downstream of the stilling basin and, along with the stilling basin, extends the entire width of the concrete chute of the spillway at the downstream end.

6. Opinion of Probable Construction Costs (OPCC)

This project OPCC is meant to assist Blue Lake Reservoir Company in the assessment of project costs. The The current developed expected construction cost is approximately $1,030,000 with an additional $150,000 reserved for construction contingencies. reflective of a competitive bid procurement process. Table 16 summarizes the estimated quantities and the breakdown of construction cost items.

Table 16.
Engineer's Estimate of Construction Cost – Alternative 2

No.	Construction Item		Estimated Quantity	Units	Unit Cost (Bid Price)	Estimated Total Cost
1	**Site Work**					
		Reclamation and Revegatation	1	LS	$ 5,000.00	$ 5,000
	Subtotal					**$ 5,000**
2	**Water Control**					
		Dewatering and Unwatering	1	LS	$ 20,000.00	$ 20,000
		Cofferdam	1	LS	$ 10,000.00	$ 10,000
	Subtotal					**$ 30,000**
3	**Erosion and Sediment Control**					
		Silt Fence	1,635	LF	$ 10.00	$ 16,350
	Subtotal					**$ 16,350**
4	**Spillway Construction**					
		Existing Spillway Concrete Demolition	280	CY	$ 100.00	$ 28,000
		Excavation	1	LS	$ 40,000.00	$ 40,000
		Backfill	1	LS	$ 25,000.00	$ 25,000
		Spillway Concrete	490	CY	$ 1,100.00	$ 539,000
		Sheet Pile	2,300	SF	$ 60.00	$ 138,000
		Drain Pipe (6" Slotted PVC Pipe)	450	LF	$ 80.00	$ 36,000
		Filter and Drain Aggregate	100	CY	$ 150.00	$ 15,000
		Riprap	100	CY	$ 140.00	$ 14,000
		Riprap Bedding	25	CY	$ 140.00	$ 3,500
		Road Base	35	CY	$ 120.00	$ 4,200
	Subtotal					**$ 842,700**
BASE CONSTRUCTION SUBTOTAL (BCS)						**$ 894,100**
		Mobilization and Demobilization @ 10% BCS				$ 89,400
		Construction Engineering @ 5% BCS				$ 44,700
ESTIMATED TOTAL CONSTRUCTION COST						**$ 1,028,200**
		Contingency Allowance @ 15% of Total Construction Cost				**$ 147,500**

The major cost items for this work include the cost for structural concrete and the sheet pile wall. Unit prices for structural concrete can vary from about $800 to $1,500 per cubic yard depending on the bidding environment. Additionally, the sheet pile upstream cutoff is a high dollar item. An upstream cutoff is highly recommended for the spillway based on the observed higher permeability sandy layers below the spillway and the potential that a preferred seepage pathway in the ground below the spillway has already been established.

As mentioned in Section 4, Alternative 3 included constructing a narrower spillway channel coupled with a dam crest raise. We estimated that the crest width could be decreased to 32 feet with a 1 foot crest raise while maintaining the required freeboard during flood routing. Instead of having the tapered side walls of the spillway chute, this concept would maintain a fixed 32 foot width along the entire spillway. The dam crest would be raised with 1 foot of aggregate base course over a 20 foot crest width. The cost for this alternative would be $10,000 to $30,000 less than the proposed Alternative 2. However, additional geotechnical studies would likely be required which may negate any cost savings.

7. Permitting

GEI supported the Blue Lake Reservoir Company with steps towards an acquisition of a Nationwide Permit (NWP) from the U.S. Army Corps of Engineers, Sacramento District, Grand Junction Regulatory Office (USACE) for the spillway rehabilitation of the spillway. GEI's work included efforts to move the project forward via a Maintenance Exemption, and while this was not approved by the USACE due to the extent of repairs necessary, materials used to propose this alternative permitting strategy were repurposed for submission of a Pre-Construction Notification (PCN) to the USACE for completion of work under a NWP 3(a): *Maintenance - repair, rehabilitation, or replacement of previously authorized, currently serviceable structures or fills*. While the permit has not yet been finalized by the USACE, it is anticipated to be issued pending review by the Office of Archaeology and Historic Preservation's (OAHP) State Historic Preservation Officer (SHPO) for cultural resources. Additionally, GEI was were informed that the USACE will not require consultation with U.S. Fish and Wildlife Service for depletions. We expect that a final permit will be issued in May 2020.

The following tasks were completed in order to support the permit application for SPK-2020-00022:

- Desktop evaluation of sensitive species potentially impacted as a result of the project.
- Desktop evaluation of aquatic resources within the project footprint and calculation of both temporary and permanent impacts.
- Assessment of potential depletions and cultural resources impacted as a result of the project.
- Formulation of final permit report and PCN.
- Preparation of supplemental materials to support consultation with the SHPO.
- General coordination and communication with the USACE to facilitate acquisition of the final permit (pending).

Please note that upon receipt of the permit verification for SPK-2020-00022, a Compliance Certificate will be attached. This Compliance Certificate will need to be completed and returned to the USACE once all work is finished.

8. Conclusions

The existing spillway at UBCR is inadequate to pass the IDF required by the SEO Regulations and is presently showing serious signs of concrete deterioration. It is recommended that the existing spillway be reconstructed to provide a discharge capacity to safely pass the SEO required IDF while also providing a minimum 1.0-foot of freeboard. This is considered the most economic approach with bringing the reservoir back into compliance with SEO regulations.

The proposed design for the replacement spillway is similar to that of the existing spillway with a concrete chute, an approach channel, concrete weir, and stilling basin. The existing spillway is 28 feet wide at the upstream side, while the replacement spillway is designed to be 38 feet wide at the spillway crest and then contract to a width of 32 feet at the stilling basin. Underdrains will be installed adjacent to the construction joints along the spillway slab to assist with proper drainage of seepage through the foundation. The current spillway has no drains installed, which likely aided in the deterioration of the existing spillway slab. A riprap channel will be installed downstream of the stilling basin. This proposed design for the replacement spillway will have the capability of safely passing the IDF at UBCR and will bring the structure back into compliance with SEO Regulations.

9. References

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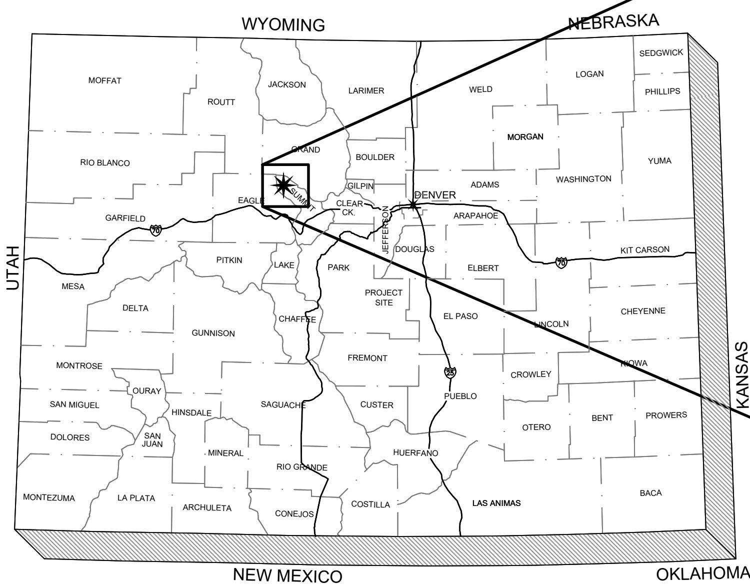
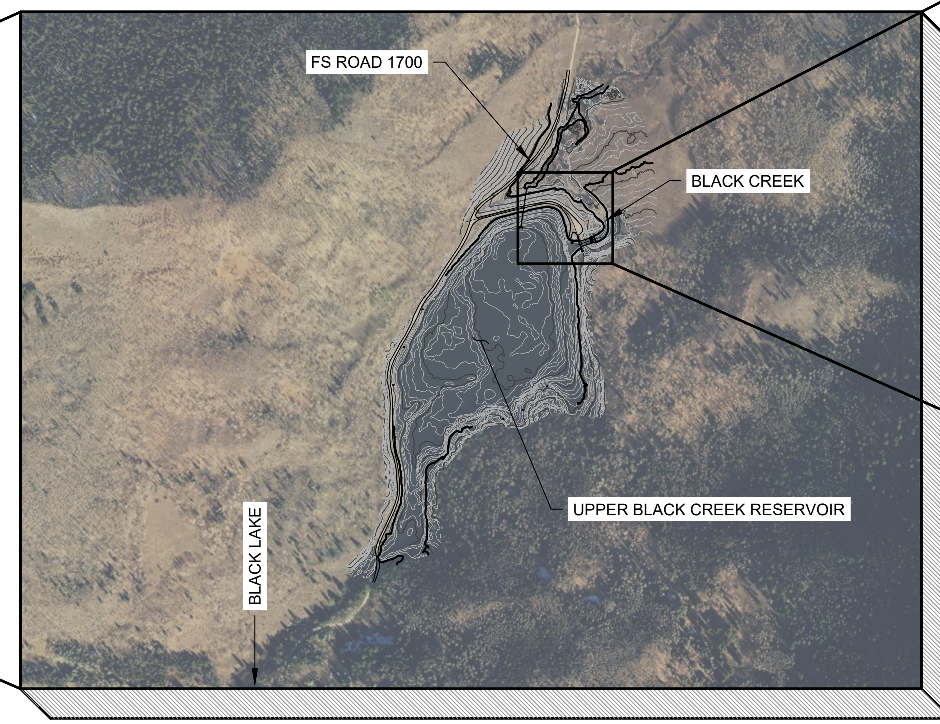
Appendix A Upper Black Creek Spillway Replacement Design Drawings

UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

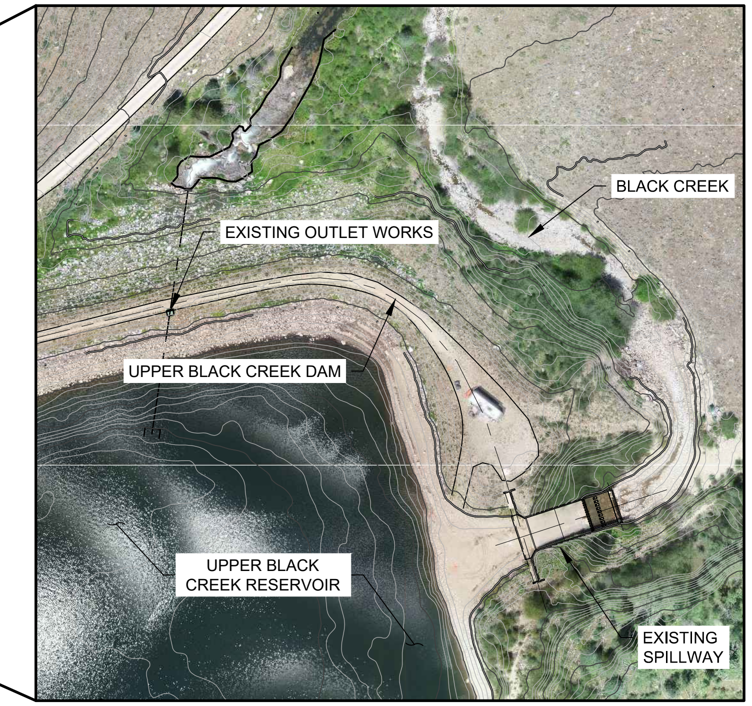
BLUE LAKE RESERVOIR COMPANY
DAM ID# 360127
SUMMIT COUNTY, COLORADO
WATER DIVISION 5, WATER DISTRICT 36



PROJECT LOCATION MAP
(NOT TO SCALE)



SITE LOCATION MAP
(NOT TO SCALE)



SITE VICINITY MAP
(NOT TO SCALE)

I HEREBY CERTIFY THAT THESE PLANS FOR THE CONSTRUCTION OF THE ***UPPER BLACK CREEK DAM SPILLWAY RECONSTRUCTION*** PROJECT WERE PREPARED BY ME (OR UNDER MY DIRECT SUPERVISION).

CHAD M. MASCHING, P.E. COLORADO P.E. #***39806***
REGISTERED ENGINEER
GEI CONSULTANTS, INC.

APPROVED ON THE ________DAY OF______________20____

STATE ENGINEER

WILLIAM T. MCCORMICK III, CO P.E. #29127
CHIEF - DAM SAFETY BRANCH

THESE PLANS MAY NOT BE MATERIALLY CHANGED, EXCEPT WITH THE PRIOR WRITTEN CONSENT OF THE STATE ENGINEER.

AS-CONSTRUCTED CERTIFICATION

THESE PLANS REPRESENT THE "AS-CONSTRUCTED" CONDITIONS FOR THE ***UPPER BLACK CREEK DAM SPILLWAY RECONSTRUCTION*** PROJECT TO THE BEST OF OUR KNOWLEDGE AND JUDGMENT, BASED IN PART ON INFORMATION FURNISHED BY OTHERS AS OF

(SIGNATURE)

GENERAL PROJECT INFORMATION

Blue Lake Reservoir Company
OWNER

CONTRACTOR

GEI Consultants Inc., Denver, Colorado
ENGINEER

CONSTRUCTION STARTED

CONSTRUCTION COMPLETED

PREPARED FOR

BLUE LAKE
RESERVOIR COMPANY
SUMMIT COUNTY • COLORADO

PREPARED BY

GEI Consultants
DENVER • COLORADO

SUMMIT TRUST
1999 SUNBURST DRIVE
VAIL, CO 81657

GEI CONSULTANTS, INC.
4601 DTC BOULEVARD
SUITE 900
DENVER, CO 80237

NOTES:
1. SEE DWG 2 FOR DRAWING LIST.

C-XXXX

DWG. NO.
1

SHEET NO.
1 of 13

P:\1801834 - Upper Black Creek\00_CAD\Design\Sheets\ 01 - Cover Sheet.dwg Apr 2020

DRAWING LIST

ABBREVIATIONS

@	=	AT
AC-FT	=	ACRE-FEET
APP	=	APPROVED, APPROVAL
APPROX.	=	APPROXIMATE
AWWA	=	AMERICAN WATER WORKS ASSOCIATION
BA	=	BORROW AREA
B.O.	=	BOTTOM OF
BOT	=	BOTTOM
CJ	=	CONSTRUCTION JOINT
CL, ℄	=	CENTERLINE
CLR	=	CLEAR
CNTR, CTR	=	CENTER
CONT	=	CONTINUOUS
CP	=	CONTROL POINT
CRJ	=	CONTRACTION JOINT
CTR(D)	=	CENTER(ED)
DIA, Ø	=	DIAMETER
DWG, DWGS	=	DRAWING OR DRAWINGS
EF	=	EACH FACE
EL., ELEV	=	ELEVATION
EQ SPC	=	EQUALLY SPACED OR EQUAL SPACES
EW	=	EACH WAY
FT	=	FEET OR FOOT
GALV	=	GALVANIZED
H, HORZ	=	HORIZONTAL
H:V	=	HORIZONTAL:VERTICAL
I.E.	=	INVERT ELEVATION
IF	=	INSIDE FACE
INV	=	INVERT
MAX	=	MAXIMUM
MIN	=	MINIMUM
NO	=	NUMBER
NTS	=	NOT TO SCALE
NWS	=	NORMAL WATER SURFACE
OPT	=	OPTION
OW	=	OUTLET WORKS
PVC	=	POLYVINYL CHLORIDE
REQ'D	=	REQUIRED
SEO	=	STATE ENGINEERS OFFICE
SIM	=	SIMILAR
J.	=	STATION
T.O.	=	TOP OF
TYP	=	TYPICAL
UNO	=	UNLESS NOTED OTHERWISE
V, VERT	=	VERTICAL
VIF	=	VERIFY IN FIELD
WSE	=	WATER SURFACE ELEVATION
W/	=	WITH
CRJ	=	CONTRACTION JOINT
OHWM	=	ORDINARY HIGH WATER MARK

GENERAL NOTES

1. GENERAL:
   - 1.1. THE TOPOGRAPHIC BASE MAP WAS GENERATED BY GEI CONSULTANTS USING A COMBINATION OF UNMANNED AERIAL VEHICLE SURVEY FLOWN BY GEI IN JULY 2018 AND 2007 BATHYMETRIC DATA PROVIDED BY RESOURCE ENGINEERING, INC. BATHYMETRIC DATA WAS RECORDED RELATIVE TO THE EXISTING SPILLWAY CREST.
   - 1.2. HORIZONTAL DATUM: NAD 83 (2014), COLORADO STATE PLANE COORDINATE SYSTEM, CENTRAL ZONE, U.S. SURVEY FEET.
   - 1.3. VERTICAL DATUM: NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)
   - 1.4. THESE NOTES SUPPLEMENT THE SPECIFICATIONS, WHICH SHALL BE REFERENCED FOR ADDITIONAL REQUIREMENTS.
   - 1.5. THE CONTRACTOR SHALL VERIFY LOCATIONS OF ALL UTILITIES PRIOR TO COMMENCING WORK. REPRESENTATIVES FROM THE RESPECTIVE UTILITY COMPANIES MUST BE NOTIFIED IN ACCORDANCE WITH STATE LAW. CALL UNDERGROUND SERVICE ALERT TOLL FREE 1-800-922-1987 TWO WORKING DAYS BEFORE YOU DIG.
   - 1.6. PLANS AND SPECIFICATIONS MAY NOT BE MATERIALLY CHANGED, EXCEPT WITH THE PRIOR WRITTEN CONSENT OF THE STATE ENGINEER.

2. EXISTING UNDERGROUND OR EXPOSED STRUCTURES AND UTILITIES:
   - 2.1. CONTRACT DOCUMENTS HAVE BEEN PREPARED USING AVAILABLE DRAWINGS AND SITE OBSERVATION AND ARE NOT WARRANTED TO BE EXACT.
   - 2.2. DURING CONSTRUCTION, THE CONTRACTOR MAY ENCOUNTER EXISTING CONDITIONS WHICH ARE NOT NOW KNOWN OR ARE AT VARIANCE WITH PROJECT DOCUMENTATION. CONTRACTOR SHALL NOTIFY THE ENGINEER OF ALL CONDITIONS NOT PER THE CONTRACT DOCUMENTS. EXAMPLES INCLUDE:
     - SIZES, DIMENSIONS, OR ELEVATIONS OTHER THAN THOSE SHOWN
     - DAMAGE OR DETERIORATION TO MATERIALS AND COMPONENTS
     - CONDITIONS OF INSTABILITY OR LACK OF SUPPORT
     - ITEMS NOTED AS EXISTING ON THE DRAWINGS BUT NOT FOUND IN THE FIELD
   - 2.3. PREPARE DIMENSIONAL DRAWINGS OF ALL DISCOVERED ITEMS.
   - 2.4. CONTRACTOR SHALL FIELD VERIFY ALL EXISTING STRUCTURAL AND UTILITIES CONDITIONS PRIOR TO SUBMITTING AS-BUILT DRAWINGS.
   - 2.5. CONTRACTOR SHALL MAKE ALLOWANCE FOR THE RESOLUTION OF SUCH DISCOVERIES IN THE CONSTRUCTION SCHEDULE.

3. USE OF DRAWINGS:
   - 3.1. DO NOT SCALE DRAWINGS.
   - 3.2. WHERE DISCREPANCIES OCCUR BETWEEN PLANS, DETAILS, GENERAL NOTES AND SPECIFICATIONS, THE MORE STRINGENT REQUIREMENTS SHALL GOVERN. DETAILS ON DRAWINGS TAKE PRECEDENCE OVER GENERAL NOTES AND TYPICAL DETAILS. DETAILS NOTED TYPICAL APPLY TO ALL SIMILAR CONDITIONS. WHERE NO SPECIFIC DETAILS ARE SHOWN, CONSTRUCTION SHALL CONFORM TO SIMILAR WORK ELSEWHERE ON THE PROJECT.

LEGEND

RIPRAP

RIPRAP BEDDING

NATIVE SEEDING

EXISTING CONCRETE

COARSE AGGREGATE

EARTH

STABILIZED CONSTRUCTION ENTRANCE

ROCKCUT

5405 — CONTOUR

? — BEDROCK

SILT FENCE

CONTRACTOR WORK LIMITS

CUT/FILL SLOPE

STAGING AREA BOUNDARY

TREELINE

DETAIL 1/8 — TYPICAL DRAIN

DETAIL TITLE. THE NUMBER "1" REFERS TO THE DETAIL DESIGNATION. THE NUMBER "8" REFERS TO THE DRAWING NUMBER WHERE THE DETAIL IS CALLED OUT.

SECTION A/8 — STA. 10+00

SECTION TITLE. THE LETTER "A" REFERS TO THE SECTION DESIGNATION. THE NUMBER "8" REFERS TO THE DRAWING NUMBER WHERE THE SECTION IS CALLED OUT.

A/8

SECTION LOCATION. THE LETTER "A" REFERS TO THE SECTION DESIGNATION. THE NUMBER "8" REFERS TO THE DRAWING NUMBER WHERE THE SECTION IS SHOWN.

1/8

DETAIL LOCATION. THE NUMBER "1" REFERS TO THE DETAIL DESIGNATION. THE NUMBER "8" REFERS TO THE DRAWING NUMBER WHERE THE DETAIL IS SHOWN.

PLAN — DAM MODIFICATIONS

MAIN TITLE

0 30 60 SCALE, FEET

DRAWING SCALE IN UNITS SPECIFIED

A/7

ELEVATION LOCATION. THE LETTER "A" REFERS TO THE SECTION DESIGNATION. THE NUMBER "7" REFERS TO THE DRAWING NUMBER

C-XXXX

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.

NO.	DATE	ISSUE/REVISION	APP
0	04/27/2020	SEO REVIEW	CMM

NOT FOR CONSTRUCTION REVIEW SUBMITTAL

Designed:	E. BLOOM
Checked:	C. MASCHING
Drawn:	E. BLOOM
Approved By:	C. MASCHING
Approval Date:	04/27/2020

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303-662-0100

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Summit County, Colorado

GEI Project 1801834

UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

DRAWING LIST, GENERAL NOTES, LEGEND, AND ABBREVIATIONS

DWG. NO. 2

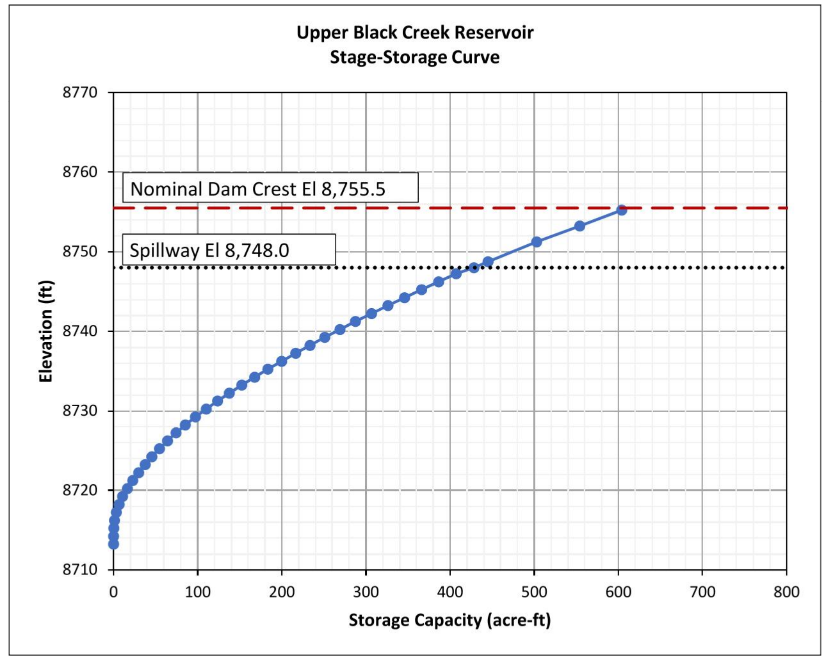
SHEET NO. 2 of 13

P:\1801834 - Upper Black Creek\00 CAD\Design\Sheets\ 02 - Drawing List, Notes, Legend.dwg Apr 26, 2020 cdiebold

RESERVOIR ELEVATION-STORAGE CAPACITY CURVE

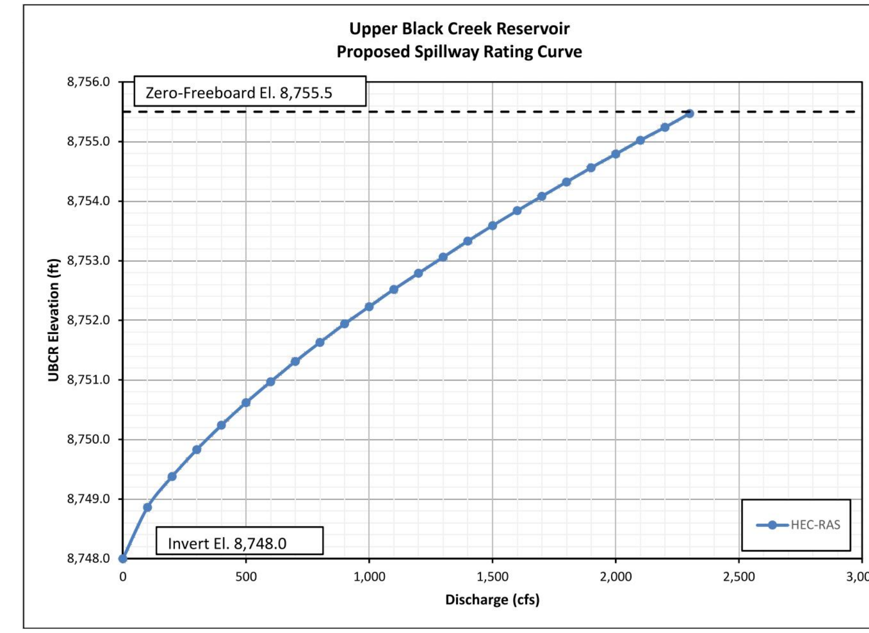
RESERVOIR ELEVATION-STORAGE CAPACITY	
ELEVATION (FT)	STORAGE (ACRE-FT)
8713.2	0.0
8714.2	0.0
8715.2	0.2
8716.2	1.2
8717.2	3.4
8718.2	6.5
8719.2	10.7
8720.2	16.3
8721.2	22.8
8722.2	29.8
8723.2	37.5
8724.2	45.6
8725.2	54.5
8726.2	64.1
8727.2	74.3
8728.2	85.4
8729.2	97.3
8730.2	110.1
8731.2	123.6
8732.2	137.7

RESERVOIR ELEVATION-STORAGE CAPACITY	
ELEVATION (FT)	STORAGE (ACRE-FT)
8733.2	152.4
8734.2	167.6
8735.2	183.4
8736.2	199.7
8737.2	216.4
8738.2	233.5
8739.2	251.0
8740.2	269.0
8741.2	287.5
8742.2	306.6
8743.2	326.1
8744.2	345.9
8745.2	366.1
8746.2	386.5
8747.2	407.2
8748.0	428.3
8748.7	445.0
8751.2	502.7
8753.2	554.0
8755.2	604.0

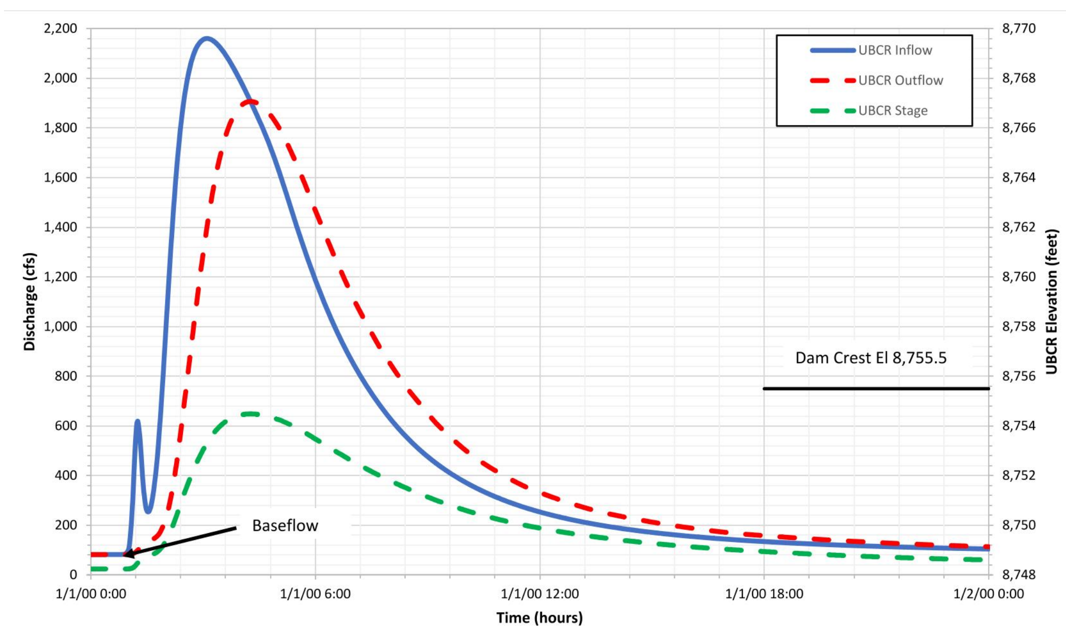


SPILLWAY DISCHARGE RATING CURVE

SPILLWAY DISCHARGE	
ELEVATION (FT)	DISCHARGE (CFS)
8748.0	0
8748.9	100
8749.4	200
8749.8	300
8750.2	400
8750.6	500
8751.0	600
8751.3	700
8751.6	800
8751.9	900
8752.2	1,000
8752.5	1,100
8752.8	1,200
8753.1	1,300
8753.3	1,400
8753.6	1,500
8753.8	1600
8754.1	1700
8754.3	1800
8754.6	1900
8754.8	2000
8755.0	2100
8755.2	2200
8755.5	2300



1,000-YEAR, 6-HOUR LOCAL STORM



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C-XXXX

Attention:

0 1"

If this scale bar does not measure 1" then drawing is not original scale.

NO.	DATE	ISSUE/REVISION	APP
0	04/27/2020	SEO REVIEW	CMM

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Designed: E. BLOOM
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Drawn: E. BLOOM
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Approval Date: 04/27/2020

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Summit County, Colorado

GEI Project 1801834

UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

HYDRAULIC AND GEOTECHNICAL INFORMATION

DWG. NO. 3

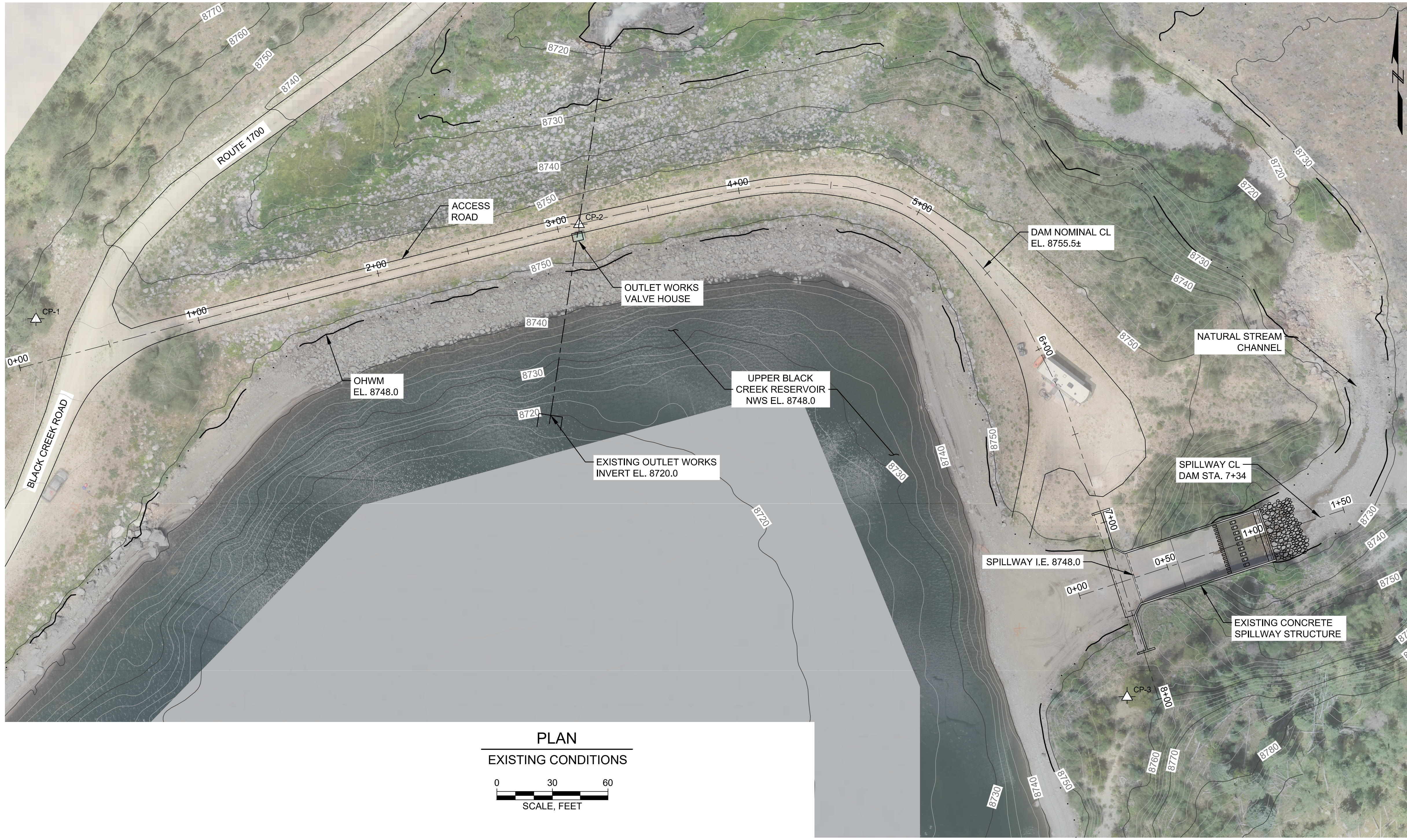
SHEET NO. 3 of 13

CONTROL POINT TABLE

POINT	NORTHING	EASTING	ELEVATION
CP-1	1719130.13	2785064.95	8756.00
CP-2	1719181.19	2785358.22	8755.50
CP-3	1718926.04	2785654.21	8759.00

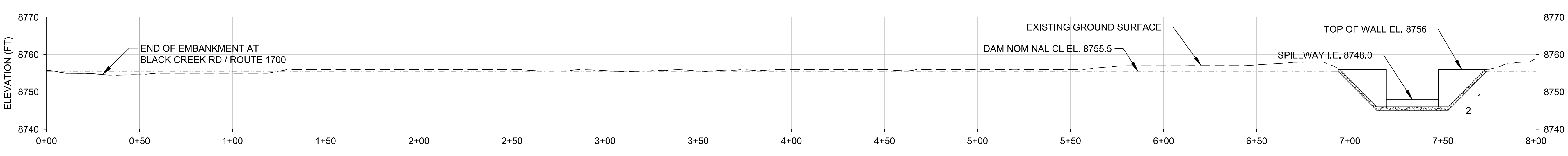
NOTES:

1. DAM AND OUTLET WORKS CENTERLINES AND STATIONING ARE APPROXIMATE. CONTRACTOR TO VERIFY.



PLAN
EXISTING CONDITIONS

0 30 60
SCALE, FEET



PROFILE
DAM CENTERLINE

HORIZONTAL SCALE, FEET 0 30 60
0 15 30 VERTICAL SCALE, FEET
NOTE: 2x VERTICAL EXAGGERATION

C-XXXX

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.

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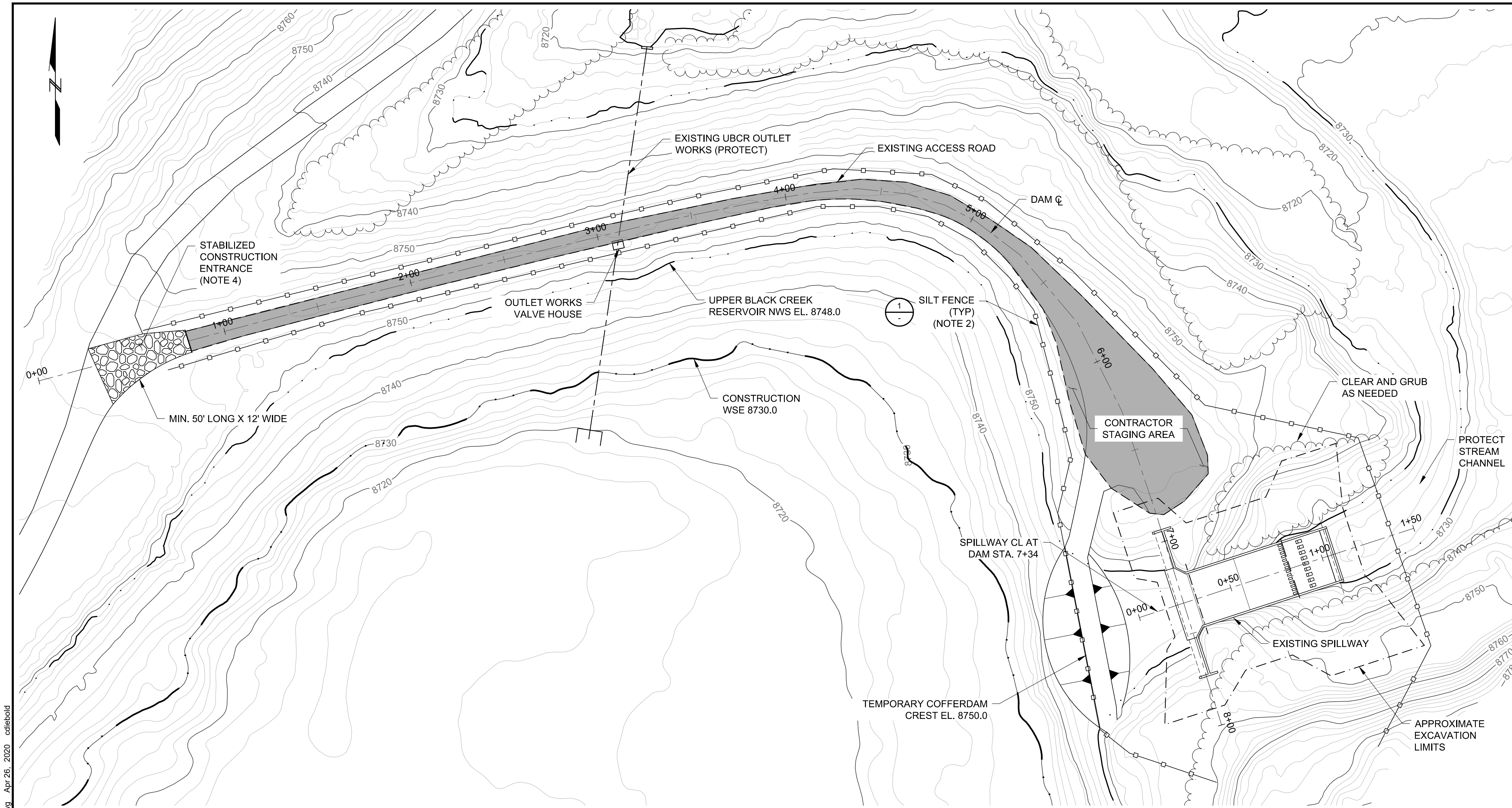
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

PLAN OF EXISTING CONDITIONS AND SURVEY CONTROL

DWG. NO. 4

SHEET NO. 4 of 13

P:\1801834 - Upper Black Creek\00 CAD\Design\Sheets\ 04 - Existing Conditions.dwg Apr 26, 2020 cdiebold

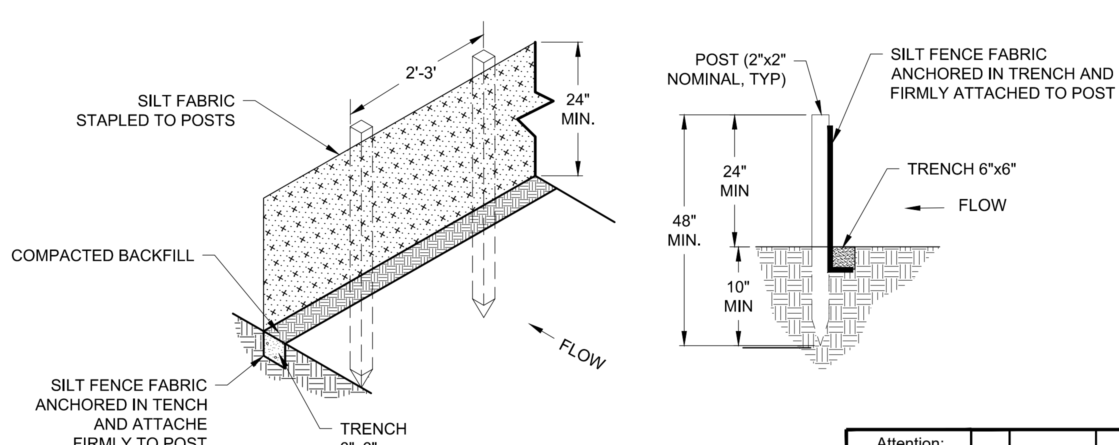


PLAN

CONSTRUCTION LIMITS

0 30 60

SCALE, FEET



DETAIL 1/-

SILT FENCE

NOTES:

1. CONTRACTOR TO DEVELOP AND SUBMIT FOR APPROVAL ALL STREAM DIVERSION AND FLOOD CONTROL MEASURES AS SPECIFIED.
2. DIRECT SURFACE FLOWS TOWARDS AREAS THAT HAVE SILT FENCE USING SMALL EARTHEN BERMS OR OTHER MEANS AS NECESSARY.
3. ALL DISTURBED AREAS REQUIRING SEEDING WILL BE RECLAIMED AND RESTORED IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
4. CONTRACTOR TO DEVELOP AND SUBMIT FOR APPROVAL STABILIZED CONSTRUCTION ENTRANCE METHODS AND DETAILS.
5. INSTALL, MAINTAIN AND REMOVE SOIL EROSION AND SEDIMENT CONTROL MEASURES (EROSION CONTROL MEASURES) IN ACCORDANCE WITH APPROVED EROSION PROTECTION AND SEDIMENT CONTROL PLAN AND WITH THE PROJECT SPECIFICATIONS.
6. COMPLY WITH APPLICABLE REQUIREMENTS OF PROJECT PERMITS.
7. EROSION CONTROL MEASURES MAY BE PHASED CONSISTENT WITH OVERALL CONSTRUCTION SEQUENCING. INCLUDE PHASING PROVISIONS IN EROSION PROTECTION AND SEDIMENT CONTROL PLAN. PHASING OF EROSION CONTROL MEASURES THAT ARE NOT DESCRIBED IN THE APPROVED EROSION PROTECTION AND SEDIMENT CONTROL PLAN WILL NOT BE ALLOWED.
8. AS A MINIMUM, INSTALL AND MAINTAIN EROSION CONTROL MEASURES AS SHOWN. INSTALL AND MAINTAIN ADDITIONAL EROSION CONTROL MEASURES AS NECESSARY TO COMPLY WITH PROJECT SPECIFICATIONS AND PERMIT REQUIREMENTS. LOCATIONS OF SPECIFIC EROSION CONTROL FEATURES SHOWN ON DRAWINGS ARE APPROXIMATE. FINAL LOCATIONS SHALL BE DETERMINED BY CONTRACTOR AND INCLUDED IN EROSION PROTECTION AND SEDIMENT CONTROL PLAN.
9. REMOVE ACCUMULATED SEDIMENT AS NECESSARY TO MAINTAIN PROPER FUNCTIONING OF EROSION CONTROL MEASURES. DISPOSE OF SEDIMENT IN ACCORDANCE WITH THE APPROVED EROSION PROTECTION AND SEDIMENT CONTROL PLAN.

C-XXXX

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.

NO.	DATE	ISSUE/REVISION	APP
0	04/27/2020	SEO REVIEW	CMM

NOT FOR CONSTRUCTION REVIEW SUBMITTAL

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Drawn:	E. BLOOM
Approved By:	C. MASCHING
Approval Date:	04/27/2020

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Summit County, Colorado

GEI Project 1801834

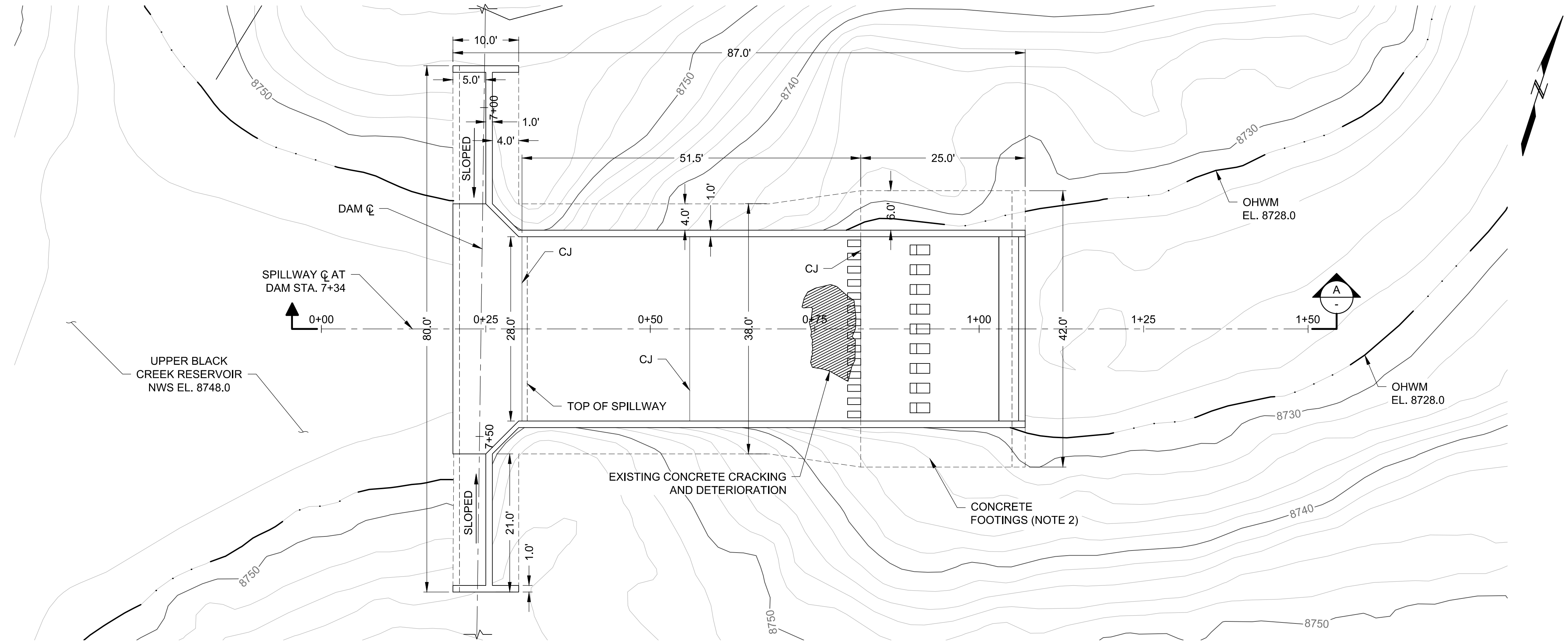
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

EROSION CONTROL AND LIMITS OF WORK

DWG. NO. 5

SHEET NO. 5 of 13

P:\1801834 - Upper Black Creek\00 CAD\Design\Sheets\ 05 - Erosion Control and Limits of Work.dwg Apr 26, 2020 cdiebold

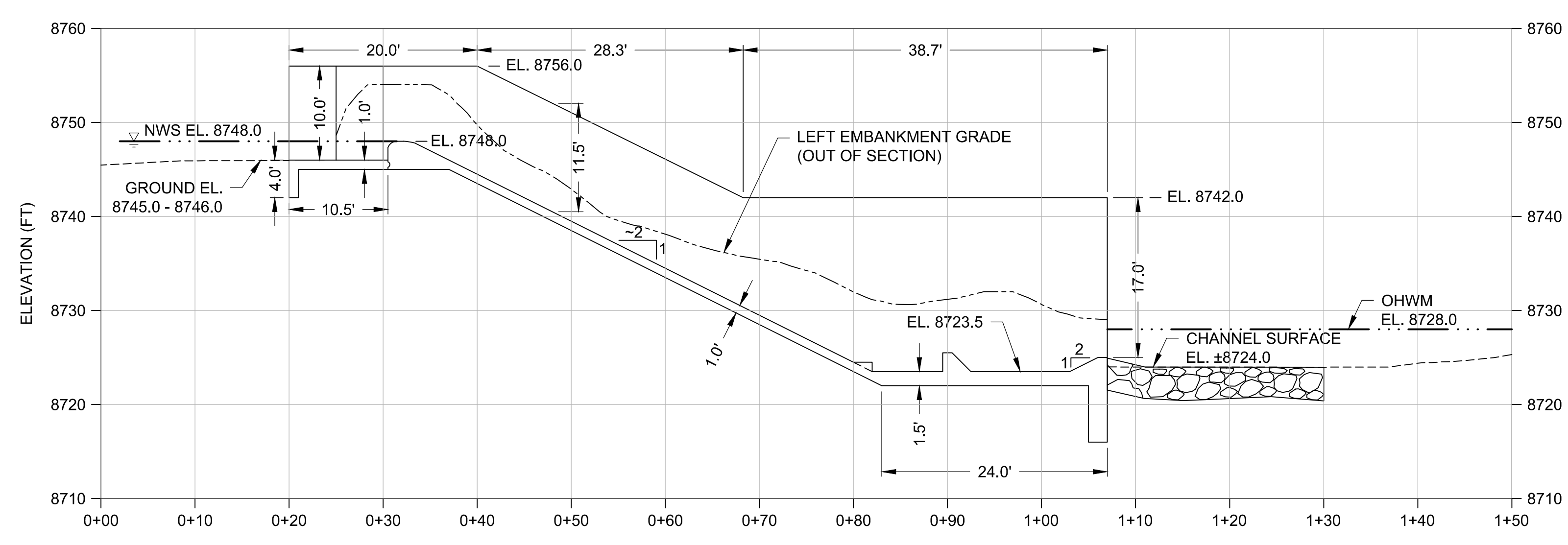


PLAN

EXISTING SPILLWAY

0 10 20

SCALE, FEET



SECTION (A)

EXISTING SPILLWAY

0 10 20

SCALE, FEET

NOTES:

1. ALL EXISTING STRUCTURE DIMENSIONS ARE APPROXIMATE AND SHOULD BE CONFIRMED IN THE FIELD BY THE CONTRACTOR.
2. CONCRETE WALL FOOTINGS ARE 1.5 FEET THICK. SEE AS-BUILT DRAWINGS FOR ANY ADDITIONAL DIMENSIONS NOT SHOWN.

P:\1801834 - Upper Black Creek\00 CAD\Design\Sheets\ 06 - Existing Plan and Section.dwg Apr 26, 2020 cdiebold

C-XXXX

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.

NO.	DATE	ISSUE/REVISION	APP
0	04/27/2020	SEO REVIEW	CMM

NOT FOR CONSTRUCTION REVIEW SUBMITTAL

Designed: E. BLOOM
Checked: C. MASCHING
Drawn: E. BLOOM
Approved By: C. MASCHING
Approval Date: 04/27/2020

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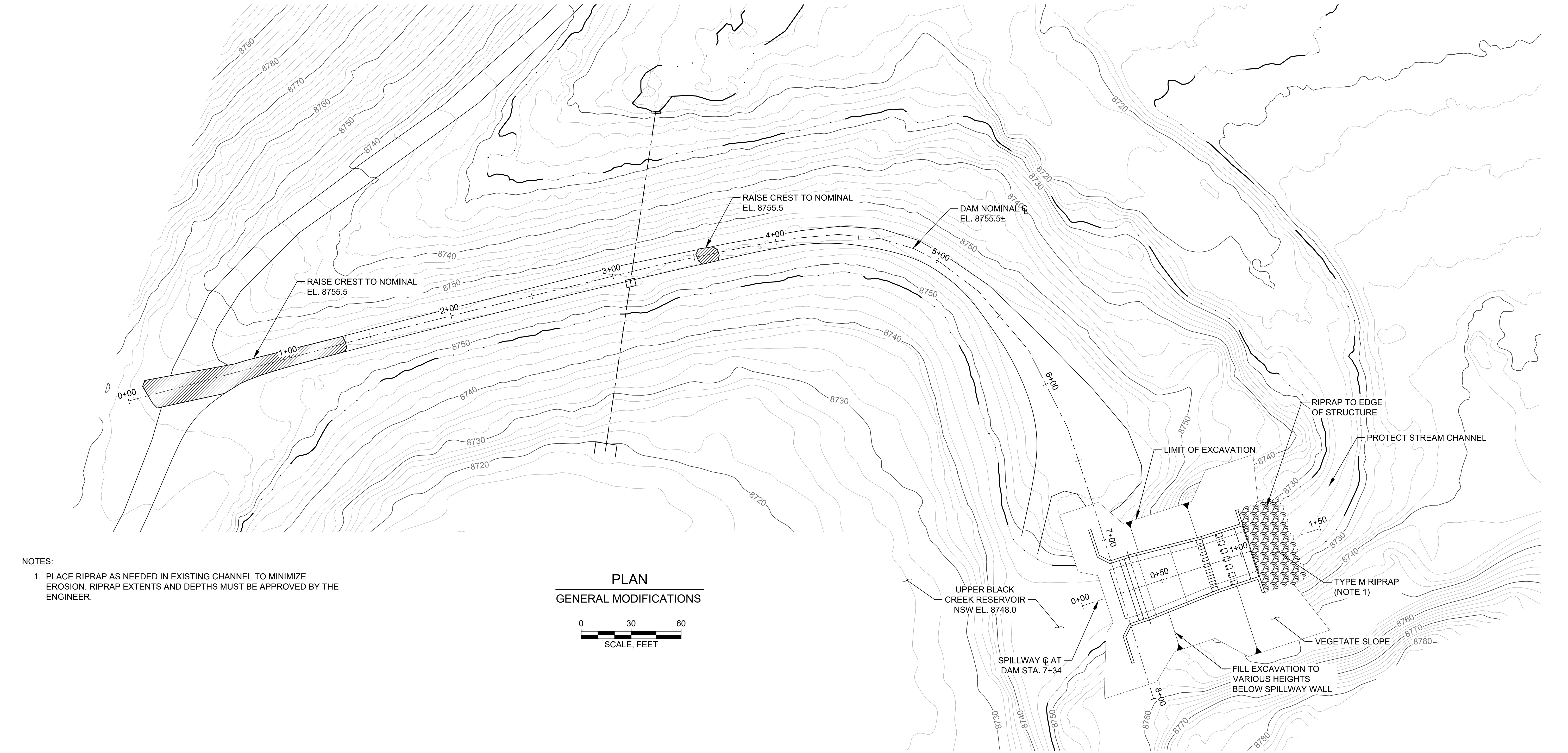
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

EXISTING SPILLWAY PLAN AND SECTION

DWG. NO. 6

SHEET NO. 6 of 13



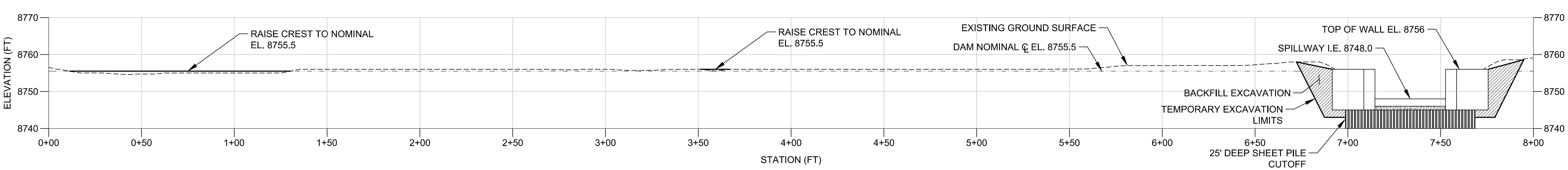
NOTES:

1. PLACE RIPRAP AS NEEDED IN EXISTING CHANNEL TO MINIMIZE EROSION. RIPRAP EXTENTS AND DEPTHS MUST BE APPROVED BY THE ENGINEER.

PLAN

GENERAL MODIFICATIONS

0 30 60
SCALE, FEET



PROFILE

EMBANKMENT CENTERLINE

HORIZONTAL SCALE, FEET
0 30 60
0 15 30
VERTICAL SCALE, FEET
NOTE: 2x VERTICAL EXAGGERATION

C-XXXX

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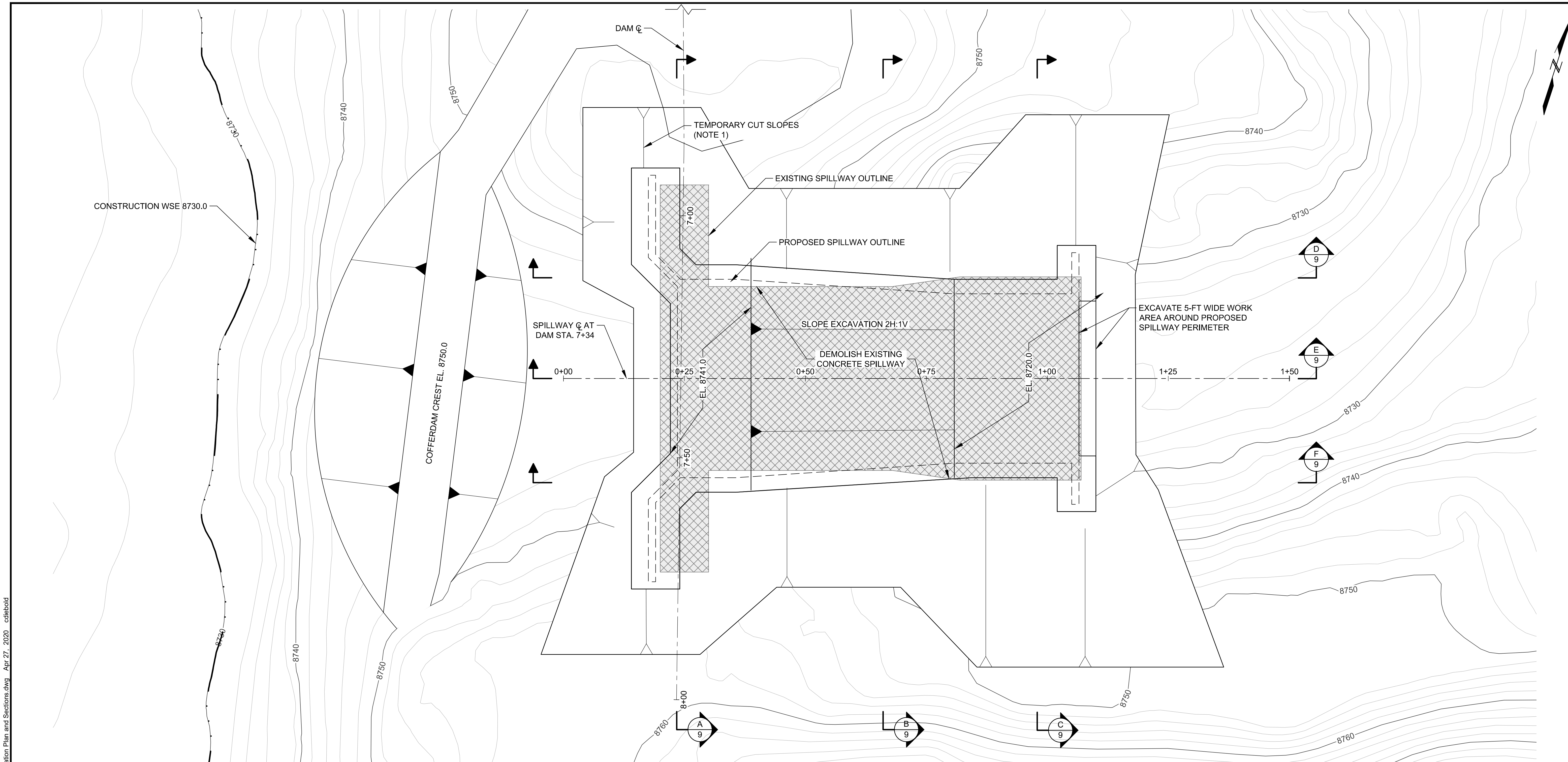
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

PLAN OF MODIFICATIONS

DWG. NO. 7

SHEET NO. 7 of 13

P:\1801834 - Upper Black Creek\00 CAD\Design\Sheets\ 07 - General Plan of Modifications.dwg Apr 26, 2020 cdiebold



PLAN

SPILLWAY DEMOLITION

0 10 20

SCALE, FEET

NOTES:

1. CONTRACTOR TO EXCAVATE SLOPES AS REQUIRED FOR STABILITY DURING CONSTRUCTION AND TO MEET OSHA SAFETY STANDARDS.

C-XXXX

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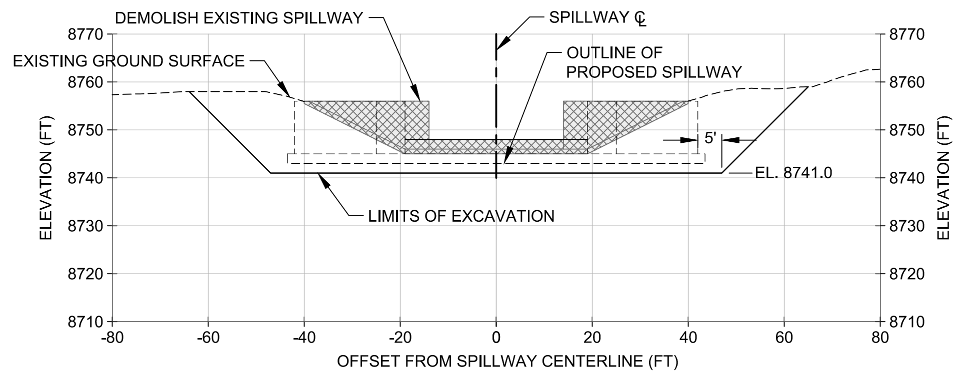
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

EXCAVATION AND DEMOLITION PLAN

DWG. NO. 8

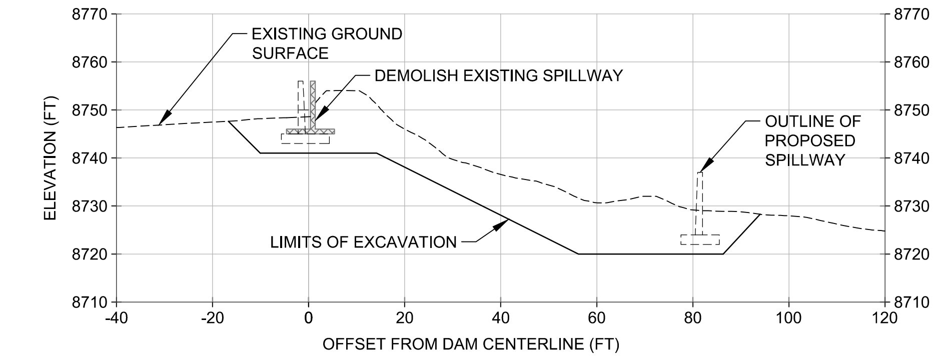
SHEET NO. 8 of 13

P:\1801834 - Upper Black Creek\00_CAD\Design\Sheets\ 08 09 - Excavation Plan and Sections.dwg Apr 27, 2020 cdiebold



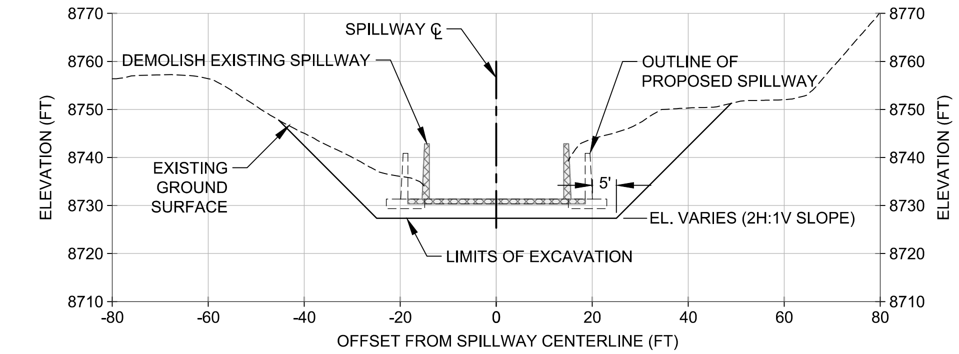
PROFILE (A/8)
SPILLWAY STA. 0+25

0 20 40
SCALE, FEET



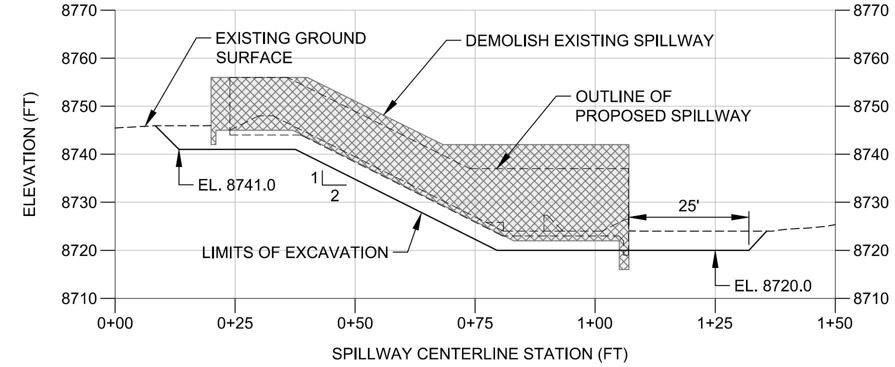
SECTION (D/8)
DAM STA. 7+10

0 20 40
SCALE, FEET



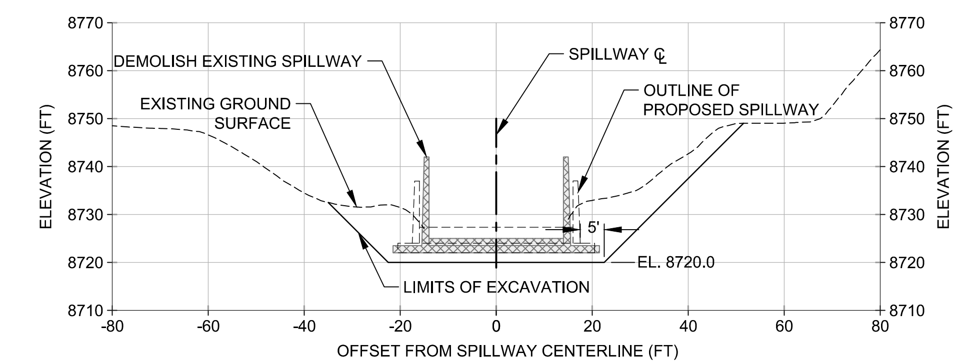
PROFILE (B/8)
SPILLWAY STA. 0+65

0 20 40
SCALE, FEET



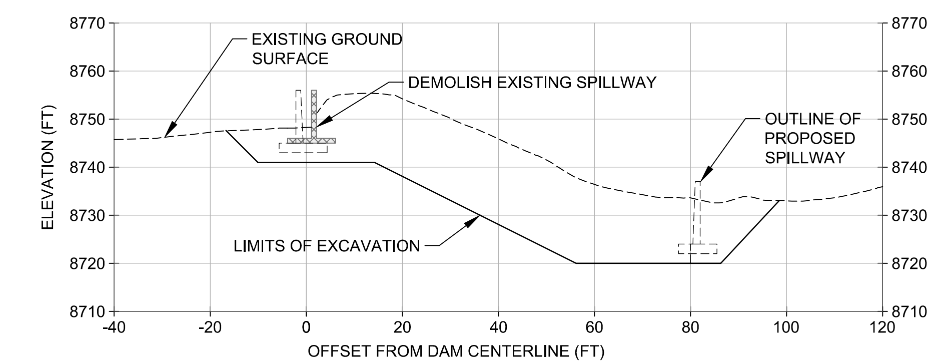
SECTION (E/8)
SPILLWAY CENTERLINE

0 20 40
SCALE, FEET



PROFILE (C/8)
SPILLWAY STA. 1+00

0 20 40
SCALE, FEET



SECTION (F/8)
DAM STA. 7+60

0 20 40
SCALE, FEET

C-XXXX

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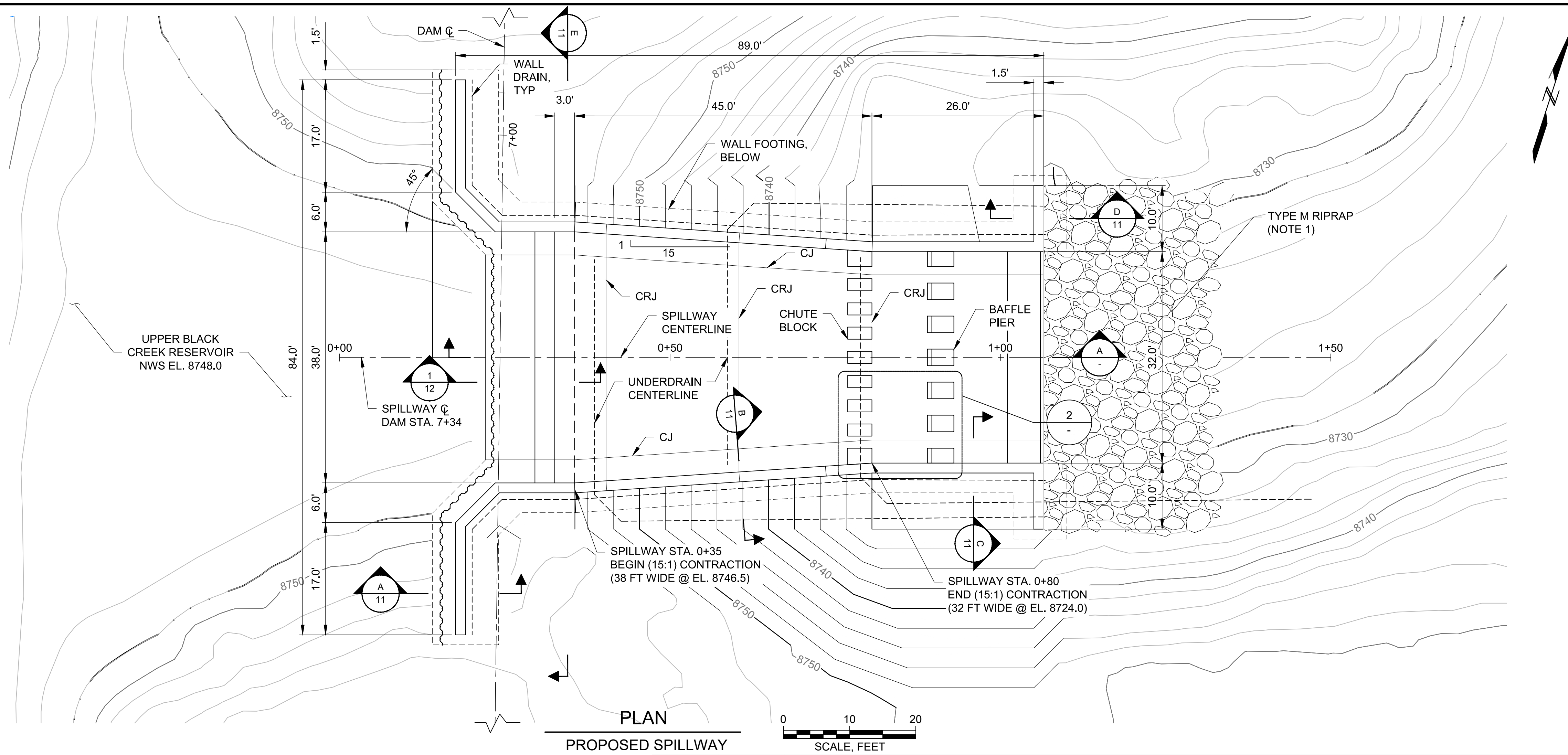
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

EXCAVATION SECTIONS

DWG. NO. 9

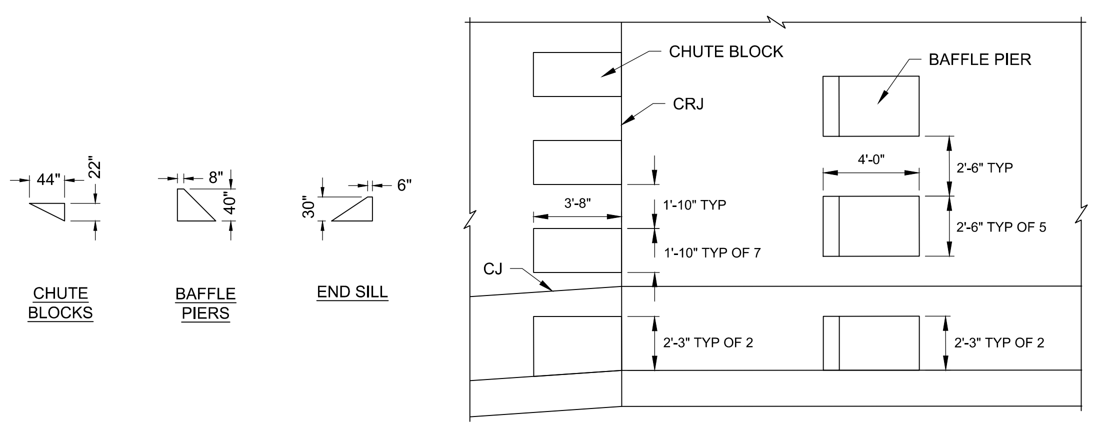
SHEET NO. 9 of 13



PLAN

PROPOSED SPILLWAY

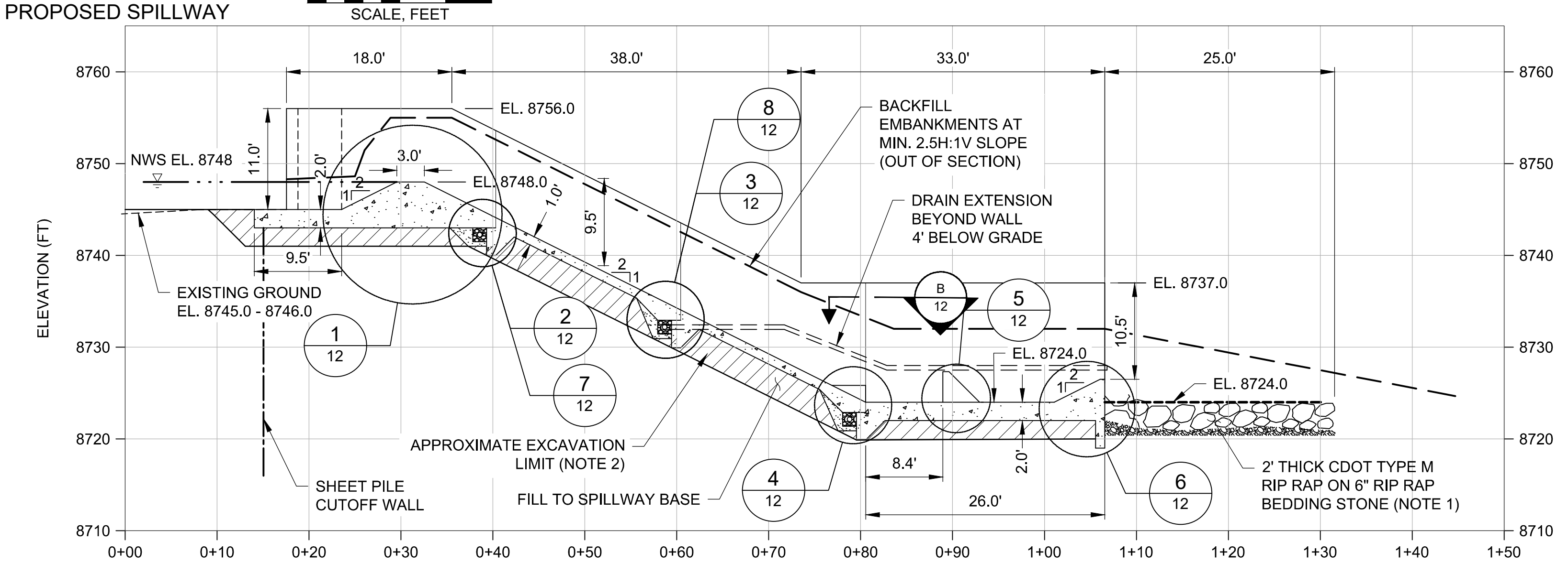
0 10 20 SCALE, FEET



DETAIL 2 -

CHUTE BLOCKS AND BAFFLE PIERS

0 4 8 SCALE, FEET



SECTION A -

PROPOSED SPILLWAY

0 10 20 SCALE, FEET

NOTES:

1. PLACE RIPRAP AS NEEDED IN EXISTING CHANNEL TO MINIMIZE EROSION. RIPRAP EXTENDS AND DEPTHS MUST BE APPROVED BY THE ENGINEER.
2. ENGINEER MAY ALLOW LESS EXCAVATION THAN SHOWN BASED ON FIELD CONDITIONS.

C-XXXX

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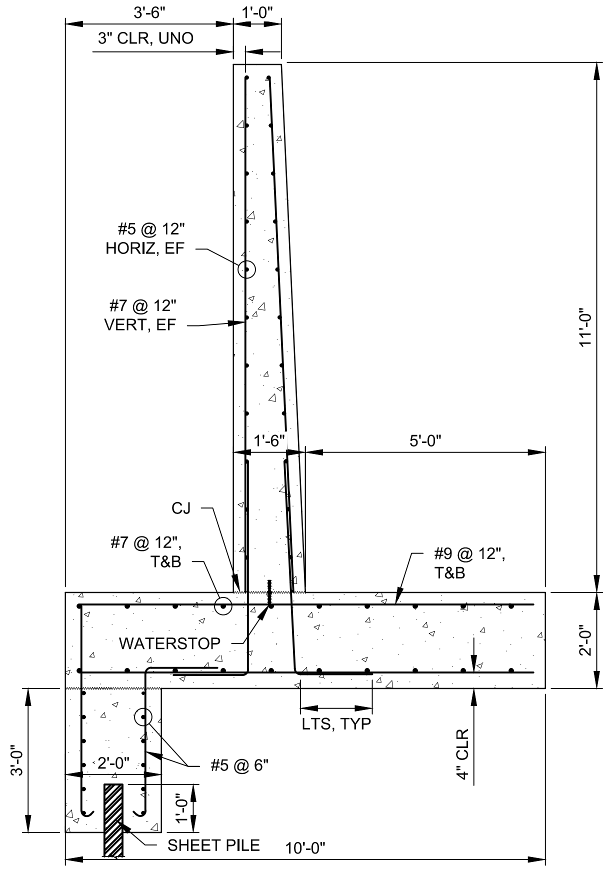
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

SPILLWAY PLAN AND SECTION

DWG. NO. 10

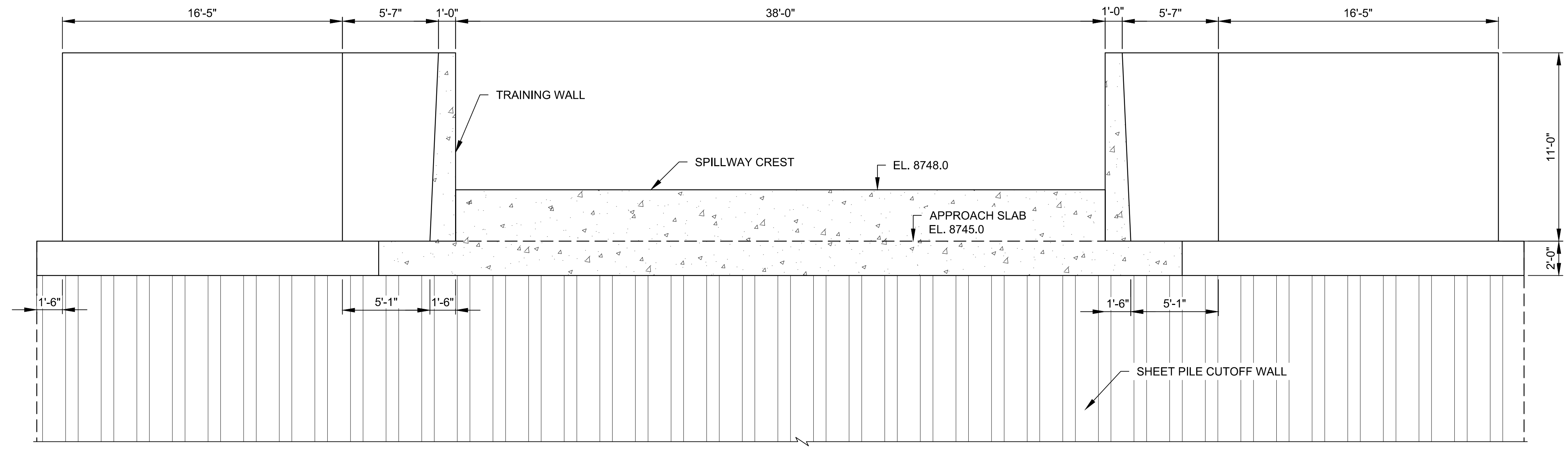
SHEET NO. 10 of 13

P:\1801834 - Upper Black Creek\00_CAD\Design\Sheets\ 10 - Spillway Plan and Section.dwg Apr 27, 2020 cdiebold



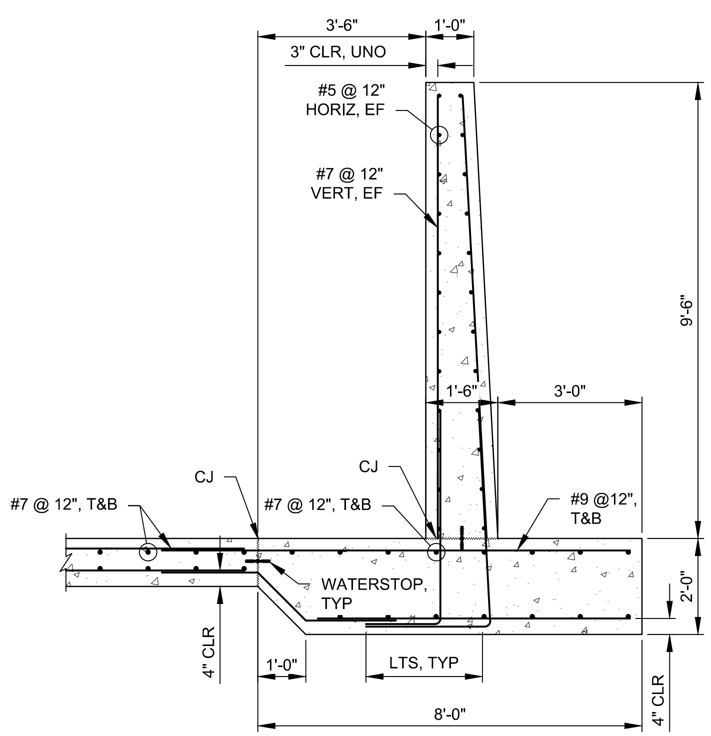
SECTION A / 10
UPSTREAM WINGWALL

0 2 4 SCALE, FEET



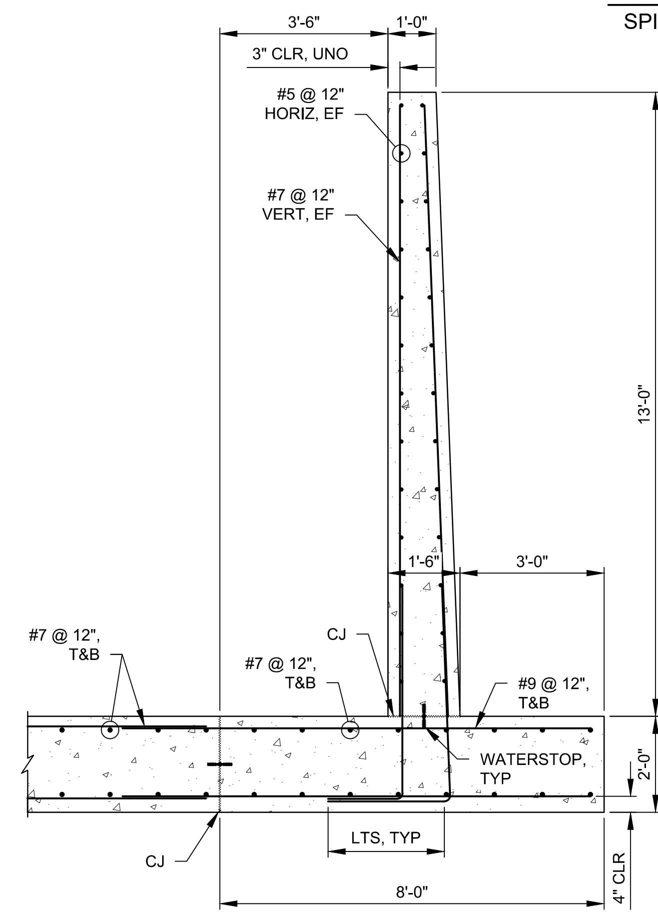
SECTION E / 10
SPILLWAY CREST

0 4 8 SCALE, FEET



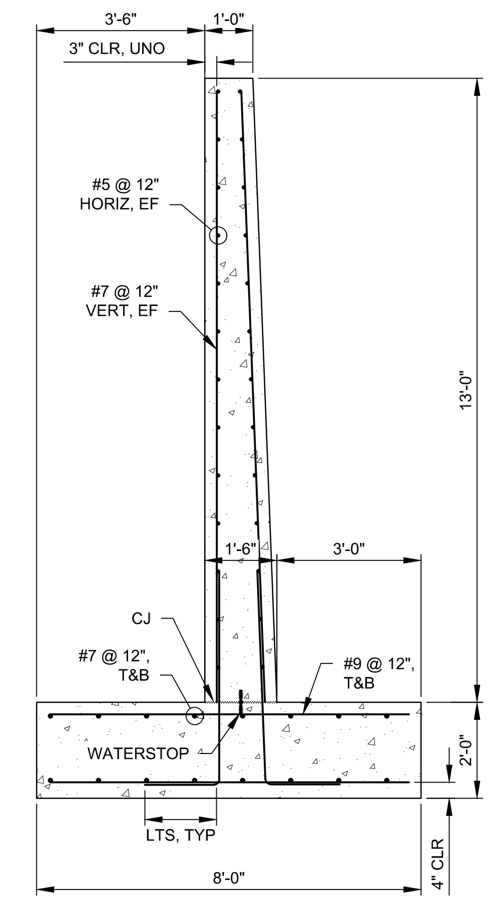
SECTION B / 10
TRAINING WALL

0 2 4 SCALE, FEET



SECTION C / 10
TRAINING WALL

0 2 4 SCALE, FEET



SECTION D / 10
DOWNSTREAM WINGWALL

0 2 4 SCALE, FEET

C-XXXX

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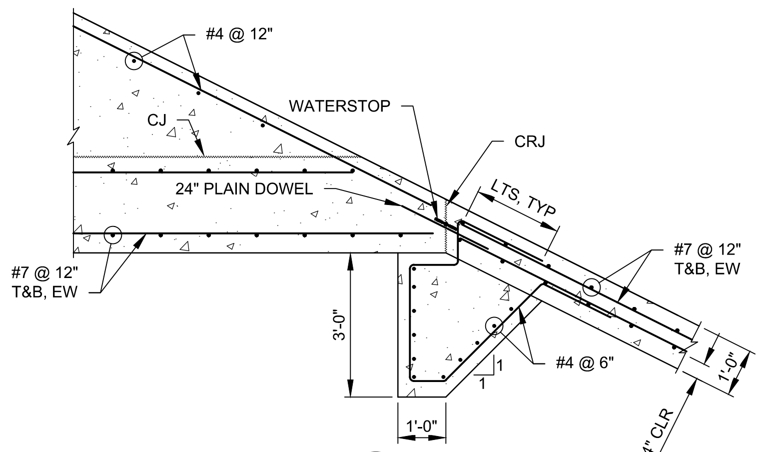
UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

SPILLWAY SECTIONS AND DETAILS (1 OF 2)

DWG. NO. 11

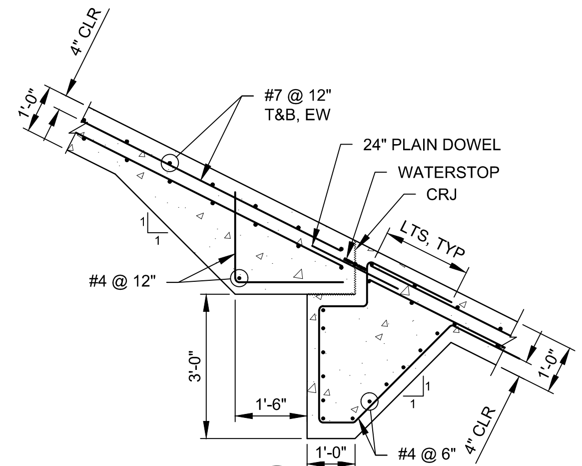
SHEET NO. 11 of 13

P:\1801834 - Upper Black Creek\00 CAD\Design\Sheets\ 11- Spillway Sections and Details (1 of 2).dwg Apr 27, 2020 cdiebold



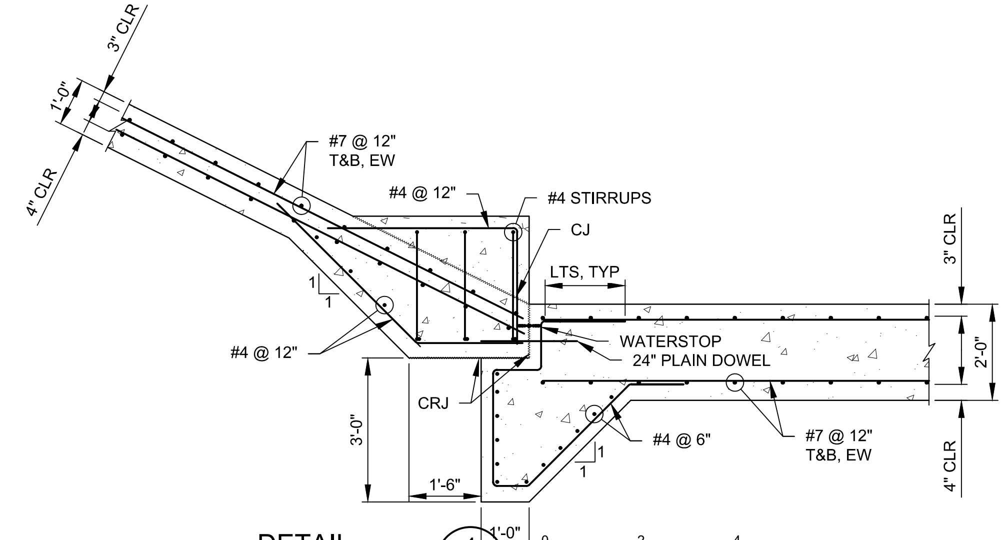
DETAIL 2 / 10
SPILLWAY JOINT

0 2 4 SCALE, FEET



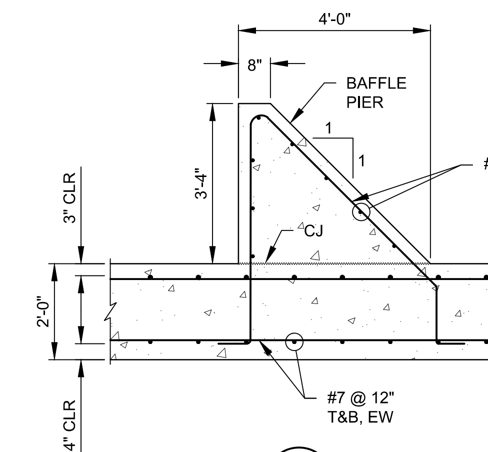
DETAIL 3 / 10
SPILLWAY JOINT

0 2 4 SCALE, FEET



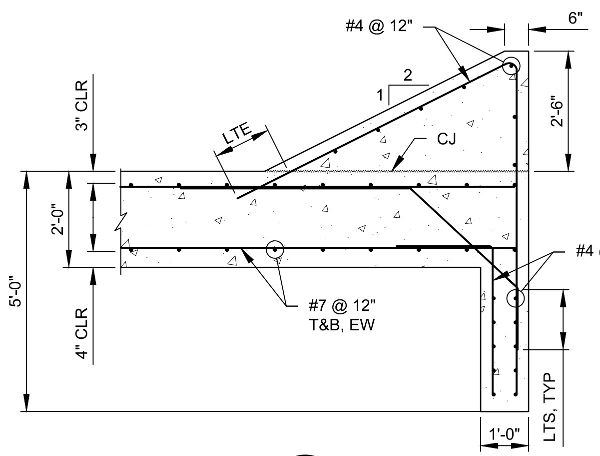
DETAIL 4 / 10
SPILLWAY JOINT

0 2 4 SCALE, FEET



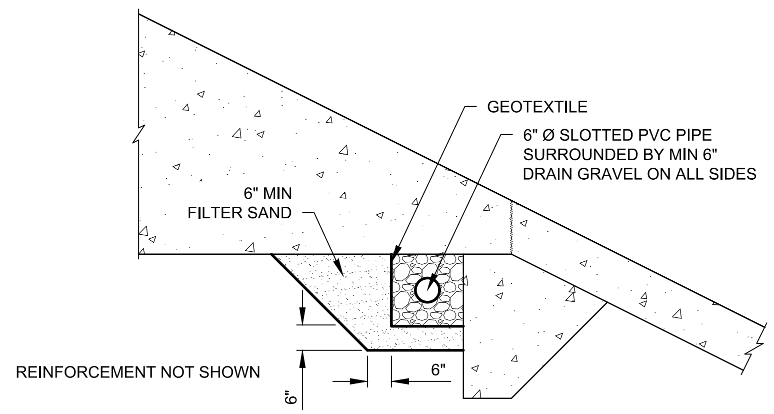
DETAIL 5 / 10
BAFFLE PIER

0 2 4 SCALE, FEET



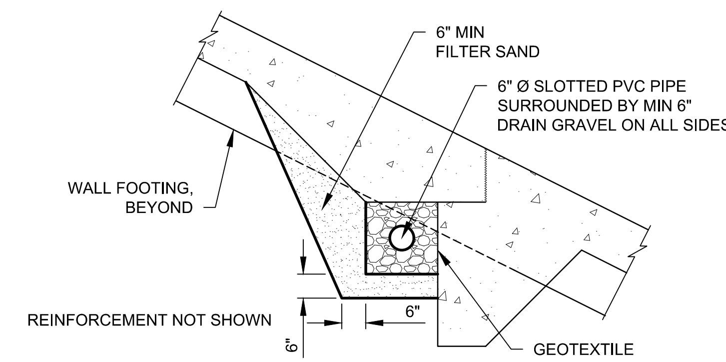
DETAIL 6 / 10
SPILLWAY OUTLET

0 2 4 SCALE, FEET



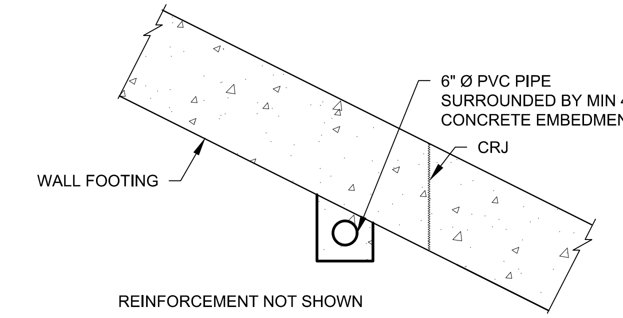
DETAIL 7 / 10
UNDERDRAIN

0 2 4 SCALE, FEET



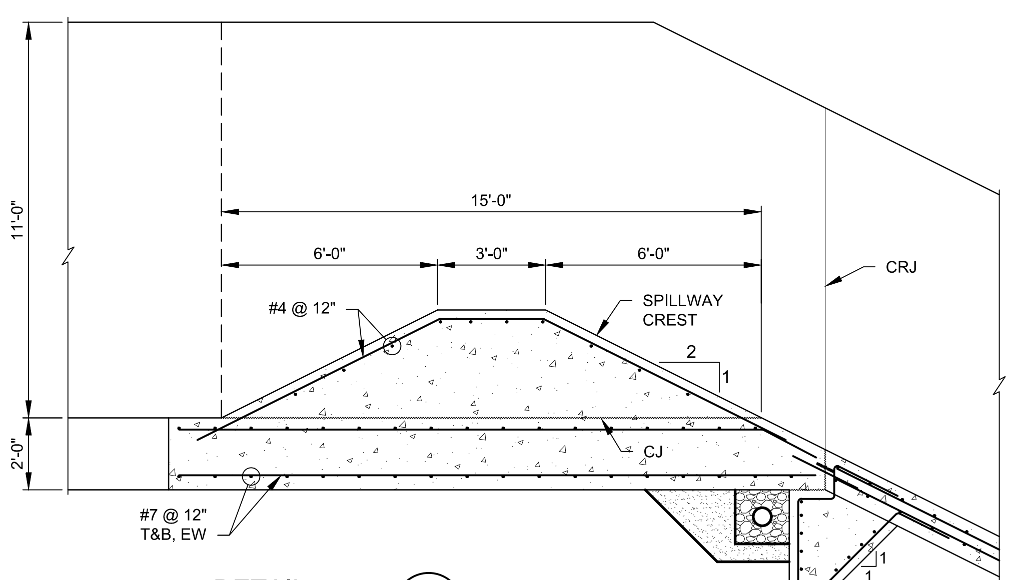
DETAIL 8 / 10
TYPICAL UNDERDRAIN

0 2 4 SCALE, FEET



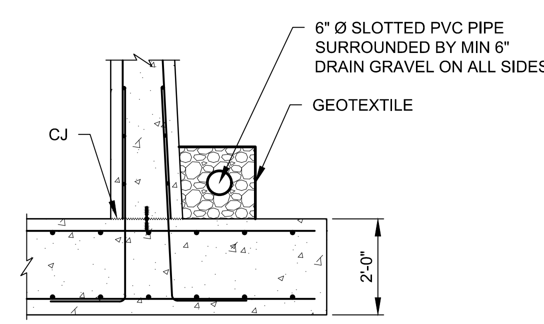
DETAIL 9 / 10
UNDERDRAIN AT WALL

0 2 4 SCALE, FEET



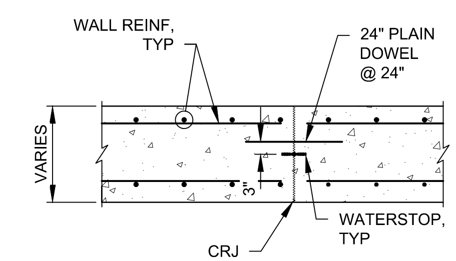
DETAIL 1 / 10
SPILLWAY CREST

0 2 4 SCALE, FEET



TYPICAL DETAIL A / 10
WALL DRAIN

0 2 4 SCALE, FEET



TYPICAL DETAIL B / 10
WALL CRJ JOINT

0 2 4 SCALE, FEET

C-XXXX

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UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

SPILLWAY SECTIONS AND DETAILS (2 OF 2)

DWG. NO. 12

SHEET NO. 12 of 13

CONCRETE NOTES

1. GENERAL

1.A. ALL WORK SHALL CONFORM TO ACI 301, LATEST EDITION, UNLESS NOTED OTHERWISE IN DRAWINGS OR PROJECT SPECIFICATIONS.

1.B. DETAILING SHALL FOLLOW THE RECOMMENDATIONS OF ACI 315 UNLESS OTHERWISE NO CHANGES SHALL BE MADE WITHOUT PRIOR APPROVAL.

1.C. DETAIL BARS IN ACCORDANCE WITH THE LATEST EDITIONS OF PUBLICATION SP-66: "ACI DETAILING MANUAL" WITH ADDED REQUIREMENTS OF THE PROJECT SPECIFICATION AND ACI 318: "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE."

1.D. BEFORE PLACING CONCRETE, CHECK ALL APPLICABLE DRAWINGS RELEASED AS SUITABLE FOR CONSTRUCTION INCLUDING MANUFACTURER'S DRAWINGS TO VERIFY THE PRESENCE OF ALL EMBEDDED MATERIAL REQUIRED IN THE PLACEMENT.

2. DIMENSIONS

2.A. DIMENSIONS ARE TO THE CENTERLINES OF THE BARS UNLESS OTHERWISE SHOWN. CLEAR COVER DIMENSIONS ARE MARKED "CLR". ALL DIMENSIONS TO A JOINT ARE TO THE CENTERLINE OF THE JOINT. BEAMS AND WALLS ARE CENTERED ON REFERENCED LINES UNLESS SHOWN OTHERWISE.

2.B. THICKNESSES SHOWN FOR WALLS AND SLABS ADJACENT TO UNDISTURBED SOIL OR ROCK ARE MINIMUM DIMENSIONS.

3. STRUCTURAL CONCRETE MIX REQUIREMENTS

3.A. SEE SPECIFICATIONS SECTION 03 30 00.

4. SLAB-ON-GRADE

4.A. TAKE PRECAUTIONS TO MINIMIZE SLAB CURLING. GRIND SLAB OR USE LEVELING COMPOUND IF FLOOR FLATNESS AND LEVELNESS VALUES ARE NOT ACCEPTABLE TO THE CONTRACTING OFFICER.

4.B. SEE SPECIFICATIONS SECTION 03 30 00 FOR FLOOR FLATNESS AND LEVELNESS REQUIREMENTS.

5. NON-SHRINK GROUT:

5.A. SEE SPECIFICATIONS SECTION 03 62 00.

6. REINFORCING MATERIALS

6.A. TYPICAL REINFORCING SHALL BE ASTM A615, GRADE 60.

6.B. FIELD BENT REINFORCING SHALL BE ASTM A706, GRADE 60.

7. REINFORCING FABRICATION

7.A. SPLICES:

- NO SPLICING OF REINFORCEMENT PERMITTED EXCEPT AS NOTED ON DRAWINGS. MAKE BARS CONTINUOUS AROUND CORNERS. WHERE PERMITTED, SPLICES MAY BE MADE BY CONTACT LAPS OR MECHANICAL CONNECTORS.
- SPLICES ARE TO BE MADE SO THAT GIVEN CLEAR DISTANCES TO THE FACE OF CONCRETE WILL BE MAINTAINED.
- SEE 'LAP SPLICE SCHEDULE' FOR LAP LENGTHS.

7.B. HOOK EMBEDMENT NOTES

- SIDE COVER IS 2 1/2 INCHES OR GREATER
- COVER BEYOND IS 2 INCHES OR GREATER
- IF SIDE COVER IS LESS THAN 2 1/2 INCHES, INCREASE LENGTHS BY 40%

8. PLACING REINFORCEMENT

8.A. PLACE REINFORCEMENT IN ACCORDANCE WITH APPROVED REINFORCEMENT SHOP DRAWINGS. IN THE EVENT OF A CONFLICT BETWEEN THESE DRAWINGS AND THE APPROVED SHOP DRAWINGS, THE APPROVED SHOP DRAWINGS SHALL GOVERN.

8.B. SEE ACI 318-11 7.5 AND ACI 301, SECTION 6.3 FOR REINFORCEMENT PLACING TOLERANCES AND ACI 117 FOR ADDITIONAL REQUIREMENTS.

8.C. THE FIRST AND LAST BARS IN SLABS AND WALLS, AND STIRRUPS IN BEAMS ARE TO START AND END AT A MAXIMUM OF ONE HALF THE ADJACENT BAR SPACING. ALL REINFORCING TO BE EQUALLY SPACED UNLESS OTHERWISE SHOWN ON THE DRAWINGS.

8.D. WHERE POSSIBLE, REINFORCEMENT SHALL BE PLACED TO MAINTAIN A CLEAR DISTANCE OF AT LEAST 1 INCH BETWEEN OTHER REINFORCEMENT, ANCHOR BOLTS, FORM TIES, OR OTHER EMBEDDED METALWORK. REINFORCEMENT PARALLEL TO ANCHOR BOLTS OR OTHER EMBEDDED METAL WORKS SHALL BE PLACED TO MAINTAIN A CLEAR DISTANCE OF AT LEAST 1-1/3 TIMES THE MAXIMUM SIZE AGGREGATE TO BE USED.

8.E. REINFORCEMENT PROTECTION:

- SEE 'CONCRETE COVER TABLE'
- SEE ACI 318-05 7.5 FOR REINFORCEMENT PLACING TOLERANCES AND ACI 117 FOR ADDITIONAL REQUIREMENTS

8.F. PROVIDE ACCESSORIES NECESSARY TO PROPERLY SUPPORT REINFORCING AT POSITIONS SHOWN ON PLANS. ALL REINFORCING, DOWELS, BOLTS, AND EMBEDDED PLATES SHALL BE SET AND TIED IN PLACE BEFORE THE CONCRETE IS POURED. "STABBING" INTO PREVIOUSLY PLACED CONCRETE IS NOT PERMITTED.

8.G. PROVIDE ADDITIONAL BARS OR STIRRUPS REQUIRED TO SECURE REINFORCING IN PLACE DURING CONCRETE PLACEMENT.

8.H. REINFORCEMENT MAY BE ADJUSTED IN THE FIELD TO CLEAR FORM TIES AND ANCHOR BARS. IN SUCH CASES, RELOCATION OF THE EMBEDDED MATERIALS MUST BE CONSIDERED. IN NO CASE SHOULD BARS BE BENT IN THE FIELD.

9. MISCELLANEOUS REINFORCING REQUIREMENTS:

9.A. MAKE ALL REINFORCING BAR BENDS IN THE FABRICATOR'S SHOP UNLESS NOTED.

9.B. NO WELDING OF REINFORCING PERMITTED UNLESS NOTED ON DRAWINGS. WHERE PERMITTED, PERFORM WELDING IN ACCORDANCE WITH AWS D1.4, LATEST EDITION.

9.C. PROVIDE ADDED REINFORCING TO TRIM ALL OPENINGS, NOTCHES, AND REENTRANT CORNERS AS NOTED IN TYPICAL DETAILS.

10. CONSTRUCTION/CONTROL JOINTS

10.A. SUBMIT DRAWINGS SHOWING CONSTRUCTION AND CONTROL JOINT LOCATIONS ALONG WITH THE SEQUENCE OF PLACEMENTS. CONSTRUCTION JOINT LOCATIONS AND CASTING SEQUENCE SHALL BE ARRANGED TO MINIMIZE THE EFFECTS OF ELASTIC AND LONG-TERM SHORTENING/SHRINKAGE. NO OTHER JOINTS SHALL BE INTRODUCED UNLESS APPROVED BY THE SOE BEFORE CONCRETE IS PLACED.

10.B. CONTROL JOINTS SHALL CONFORM TO DETAILS PROVIDED ON DRAWINGS. IMMEDIATELY BEFORE THE SECOND CONCRETE PLACEMENT, THE PROJECTING HALF OF THE DOWEL SHALL BE GREASED TO PREVENT BOND TO THE CONCRETE.

10.C. CONSTRUCTION JOINTS IN SLABS-ON-GRADE, AND STRUCTURAL SLABS SHALL BE LOCATED TO ACCOMMODATE THE MAXIMUM LENGTH AND AREA THE CONTRACTOR CAN REASONABLY POUR, FINISH, AND JOINT IN THE SAME DAY, BUT SHALL NOT EXCEED A LENGTH OF 150 FEET WITH A MAXIMUM AREA OF 15,000 SQUARE FEET UNLESS APPROVED BY THE ENGINEER.

10.D. SHEAR FRICTION JOINTS: WHERE CONSTRUCTION JOINTS ARE LABELED AS "ROUGHENED" ON THE DRAWINGS, THE ENTIRE JOINT SURFACE SHALL BE MECHANICALLY ROUGHENED TO A 1/4" AMPLITUDE AND THOROUGHLY CLEANED. EXPOSE THE COURSE AGGREGATE IN THE HARDENED CONCRETE AND REMOVE ALL LOOSE MATERIAL

11. FINISHING

11.A. SEE SPECIFICATIONS SECTION 03 33 00.

11.B. UNLESS OTHERWISE INDICATED, CHAMFER EDGES OF ALL PERMANENTLY EXPOSED CONCRETE SURFACES WITH A 45 DEGREE BEVEL, 3/4 INCH X 3/4 INCH. CHAMFER STRIP MAY NOT BE SHOWN ON THE DESIGN DRAWINGS.

12. MEP AND OTHER OPENINGS AND EMBEDMENTS:

12.A. PROVIDE SLEEVES AT OPENINGS (SUCH AS THOSE REQUIRED FOR PLUMBING AND ELECTRICAL PENETRATIONS) BEFORE PLACING CONCRETE. DO NOT CUT REINFORCING WHICH MAY CONFLICT. CORING OF CONCRETE IS NOT PERMITTED.

12.B. REFER TO TYPICAL DETAILS FOR SPACING LIMITS ON SLEEVES AND FOR REQUIREMENTS FOR EMBEDDED CONDUIT AND PIPE.

13. PRECAST CONCRETE:

13.A. THE STRUCTURAL DRAWINGS SHOW THE INTENT OF THE PRECAST CONCRETE FRAMING. REINFORCING SHOWN BUT NOT CALLED OUT IS CONCEPTUAL AND SHALL BE DESIGNED BY THE PRECAST MANUFACTURER. REINFORCING CALLED OUT ON DRAWINGS IS A MINIMUM FOR FINAL INPLACE CONDITIONS.

13.B. SIZE OF PRECAST MEMBERS SHALL NOT BE CHANGED UNLESS ACCEPTED BY THE CONTRACTING OFFICER.

13.C. PROVIDE RANDOM ORIENTED FIBER REINFORCED BEARING PADS UNLESS NOTED OTHERWISE. MINIMUM PAD ALLOWABLE COMPRESSION STRESS SHALL BE 1500 PSI.

13.D. PROVIDE MEMBER CONNECTIONS DETAILED ON THE DRAWINGS WITHOUT VARIATION. CONNECTIONS CONCEPTUALLY SHOWN OR NOT SHOWN SHALL BE DESIGNED BY THE PRECAST MANUFACTURER TO TRANSFER LOADS FROM ERECTION AND FINAL CONDITIONS.

MASONRY NOTES

1. DEFINITIONS

1.A. STRUCTURAL MASONRY IS DEFINED AS BEING EITHER LOAD BEARING AND/OR SERVING AS PART OF THE LATERAL LOAD RESISTING SYSTEM. STRUCTURAL MASONRY IS SHOWN ON THE STRUCTURAL PLANS AND DEFINED IN SCHEDULES AND DETAILS ON THE STRUCTURAL DRAWINGS.

2. DESIGN STRENGTH

2.A. DEVELOP 1800 PSI COMPRESSIVE STRENGTH (F'M) IN 28 DAYS.

2.B. STEEL REINFORCING:

- PRIMARY REINFORCING: ASTM A615, 60 KSI
- HORIZONTAL JOINT REINFORCING: ASTM A82, PREFABRICATED, LADDER TYPE

3. SPLICES

3.A. SEE MASONRY LAP SPLICE SCHEDULE FOR LAP LENGTHS.

4. INSTALLATION REQUIREMENTS

4.A. GROUT SOLID ALL CELLS CONTAINING REINFORCING, EMBEDDED ITEMS, AND ALL OTHER CELLS NOTED ON THE CONTRACT DOCUMENTS.

LAP SPLICE AND DEVELOPMENT LENGTH SCHEDULE (INCHES)

BAR SIZE (US)	BAR DIAMETER	F'c = 4500 PSI						
		COMP		TENSION				
		LCE	LCS	LDH	LTE TOP	LTE	LTS TOP	LTS
#3	0.375	8	12	6	17	13	23	17
#4	0.500	9	15	6	23	17	30	23
#5	0.625	11	18	8	29	22	38	29
#6	0.750	13	22	9	34	26	45	35
#7	0.875	15	26	11	50	39	66	51
#8	1.000	18	30	13	58	44	76	58
#9	1.128	20	33	14	65	50	85	66
#10	1.270	22	38	16	73	56	96	74
#11	1.410	25	42	18	82	63	107	82

SPLICE SCHEDULE NOTES

1. 'LCE' COMPRESSION EMBEDMENT LENGTH, 'LCS' = COMPRESSION LAP SPLICE LENGTH, 'LDH' = HOOK DEVELOPMENT LENGTH, 'LTE' = TENSION EMBEDMENT LENGTH, 'LTS' TENSION LAP SPLICE LENGTH.
2. 'TOP' BARS ARE HORIZONTAL BARS PLACED WITH MORE THAN 12 INCHES OF FRESH CONCRETE IS CAST BELOW THE BAR.
3. ALL SPLICES SHALL BE WIRED IN CONTACT AND STACKED VERTICALLY.
4. ALL SPLICES ARE 'LTS' UNLESS NOTED OTHERWISE.
5. SMALLER BAR LAP LENGTH SHALL BE USED WHEN SPLICING DIFFERENT SIZED BARS.
6. LAP LENGTHS SPECIFICALLY DETAILED ON DRAWINGS SHALL GOVERN IN LIEU OF LAP LENGTHS SCHEDULE.
7. SCHEDULE LAP LENGTHS ASSUMPTIONS:
   - CLEAR COVER IS GREATER THAN BAR DIAMETER, AND NOT LESS THAN 3/4".
   - CLEAR SPACING BETWEEN BARS IS GREATER THAN 2 BAR DIAMETERS.
   - IF EITHER CONDITION ABOVE IS NOT MET FOR A GIVERN BAR, INCREASE LENGTH BY 50%.
8. SPLICE LENGTHS NOTED BASED ON FY = 60,000 PSI. FOR OTHER YIELD STRENGTHS, MULTIPLY SPLICE LENGTHS NOTED BY FY/60,000.

STEEL NOTES

1. GENERAL

1.A. FABRICATION AND ERECTION OF STRUCTURAL STEEL SHALL CONFORM TO CURRENT AISC MANUAL OF STEEL CONSTRUCTION.

2. BOLTED CONNECTIONS

2.A. ANCHORS AND STRUCTURAL BOLTS SHALL BE STRUCTURAL STEEL, ASTM A 325, STRUCTURAL NUTS SHALL BE STRUCTURAL STEEL ASTM A563. ALL BOLTED STRUCTURAL CONNECTIONS SHALL CONFORM TO THE AISC SPECIFICATION FOR STRUCTURAL JOINTS. ALL STRUCTURAL BOLTED CONNECTIONS SHALL BE BEARING-TYPE CONNECTIONS.

3. ANCHOR BOLTS AND EMBEDDED THREADED RODS

3.A. ANCHOR BOLTS AND EMBEDDED THREADED RODS SHALL BE STRUCTURAL STEEL, ASTM F 1554, GRADE 55.

4. WELDING REQUIREMENTS:

4.A. WELDERS: HAVE IN POSSESSION CURRENT EVIDENCE OF PASSING THE APPROPRIATE AWS QUALIFICATION TESTS.

4.B. MINIMUM WELDS: AISC SPECIFICATION, NOT LESS THAN 3/16-INCH FILLET, CONTINUOUS UNLESS OTHERWISE NOTED.

4.C. WELD SIZES AND LENGTHS CALLED FOR ON THE DRAWINGS ARE THE NET EFFECTIVE REQUIRED. INCREASE WELD SIZE IF GAPS EXIST AT THE FAYING SURFACE.

4.D. WELD SIZES SHALL BE AS SHOWN UNLESS A GREATER SIZE IS REQUIRED BY ANSI/AISC 360-05 TABLES J2.3 AND J2.4.

4.E. ALL GROOVE WELDS SHALL BE COMPLETE PENETRATION UNLESS NOTED.

4.F. FIELD WELDING SYMBOLS INDICATE SUGGESTED CONSTRUCTION PROCEDURES.

4.G. WELDING ELECTRODES FOR PLAIN STRUCTURAL STEEL SHALL BE AWS SERIES E-70.WELDING ELECTRODES FOR GALVANIZED STEEL SHALL BE AWS SERIES E6010 OR E6011.

5. STRUCTURAL STEEL INSTALLATION:

5.A. ALL BOLTS USED IN CONNECTIONS SHALL BE INSTALLED SNUG TIGHT AS DEFINED BY AISC. UNLESS NOTED OTHERWISE.

C-XXXX

Attention:				
0 1"				
If this scale bar does not measure 1" then drawing is not original scale.	0	04/27/2020	SEO REVIEW	CMM
	NO.	DATE	ISSUE/REVISION	APP

NOT FOR CONSTRUCTION REVIEW SUBMITTAL

Designed: C. DIEBOLD
Checked: M. PROVENCHER
Drawn: C. DIEBOLD
Approved By: C. MASCHING
Approval Date: 04/27/2020

GEI Consultants
4601 DTC Boulevard
Denver, Colorado 80237
303-662-0100

Blue Lake Reservoir Company
Summit County, Colorado

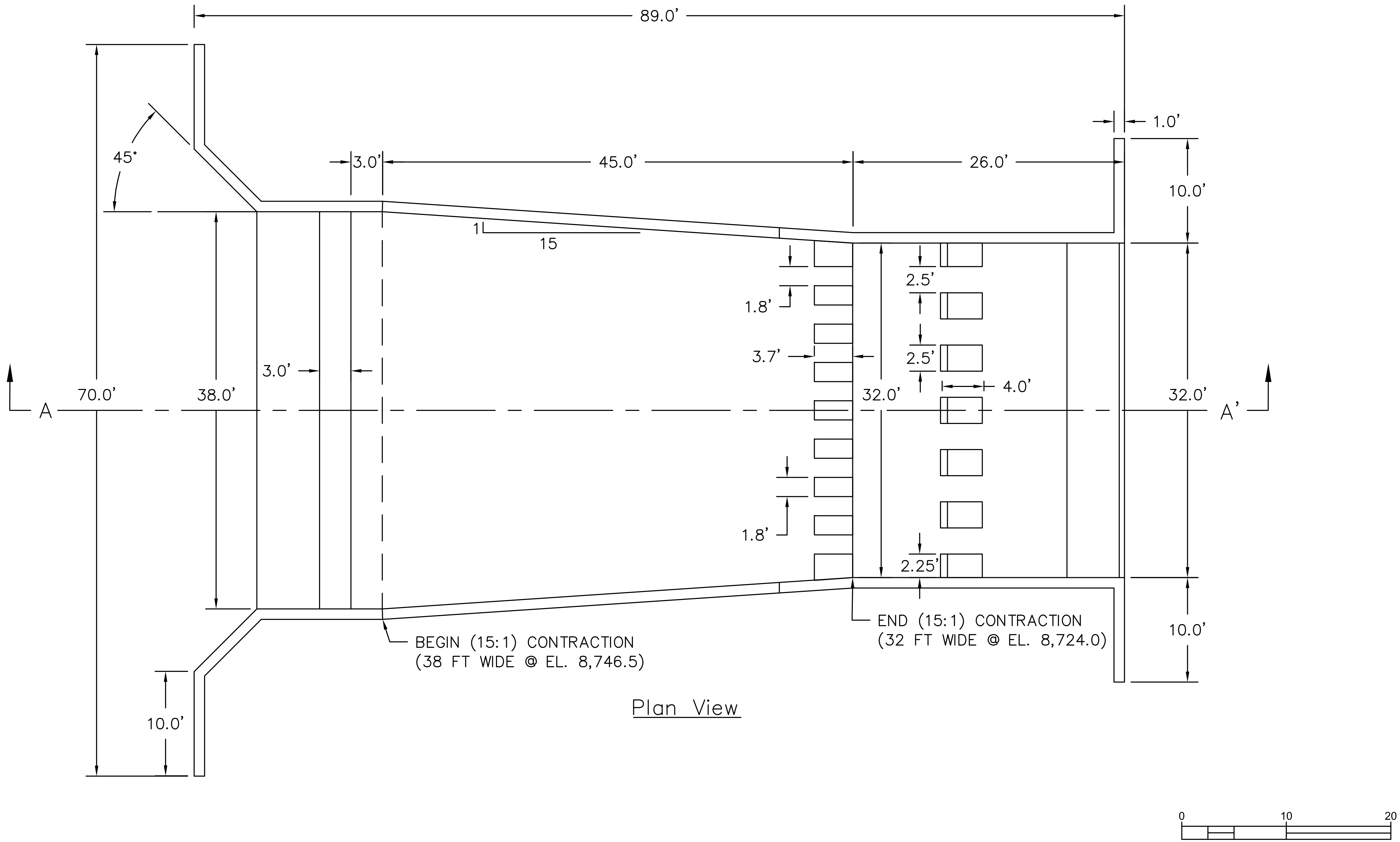
GEI Project 1801834

UPPER BLACK CREEK RESERVOIR SPILLWAY REPLACEMENT

STRUCTURAL NOTES AND DETAILS

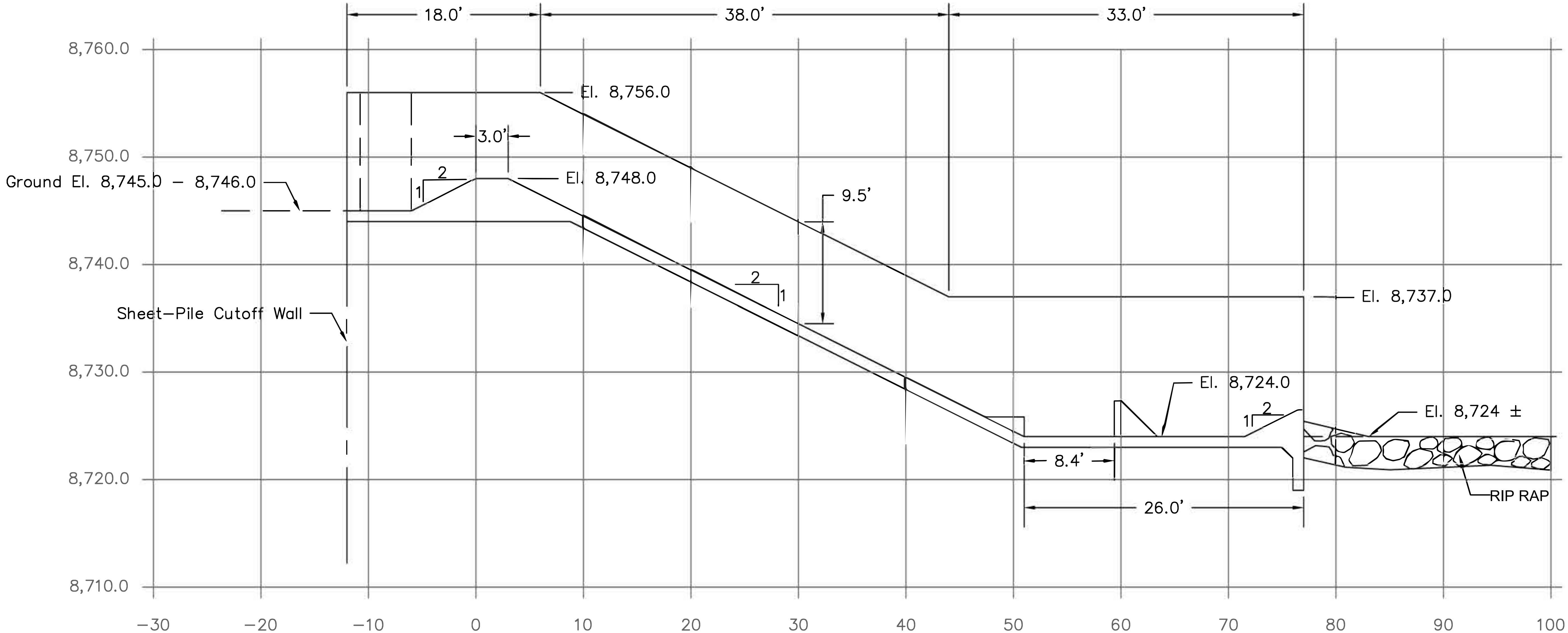
DWG. NO. 13

Appendix B Hydraulic Calculations

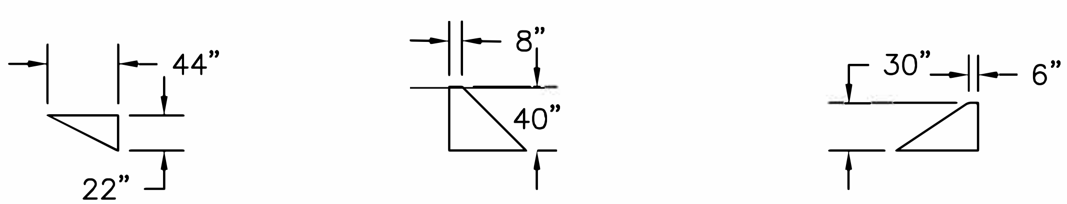


UPPER BLACK CREEK RESERVOIR SPILLWAY RECONSTRUCTION PROJECT	GEI Consultants	PROPOSED SPILLWAY PLAN VIEW
BLUE LAKE RESERVOIR COMPANY SUMMIT COUNTY, CO	Project 1801834	SEPTEMBER 2019 FIGURE B.1

L:\Working\Blue Lake Reservoir Company\CAD\Spillway_Chute



Section View A–A'

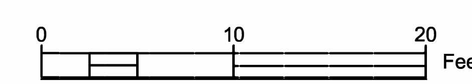


CHUTE BLOCKS

BAFFLE PIERS

END SILL

TYPE III STILLING BASIN DETAILS



UPPER BLACK CREEK RESERVOIR
SPILLWAY RECONSTRUCTION PROJECT

BLUE LAKE RESERVOIR COMPANY
SUMMIT COUNTY, CO

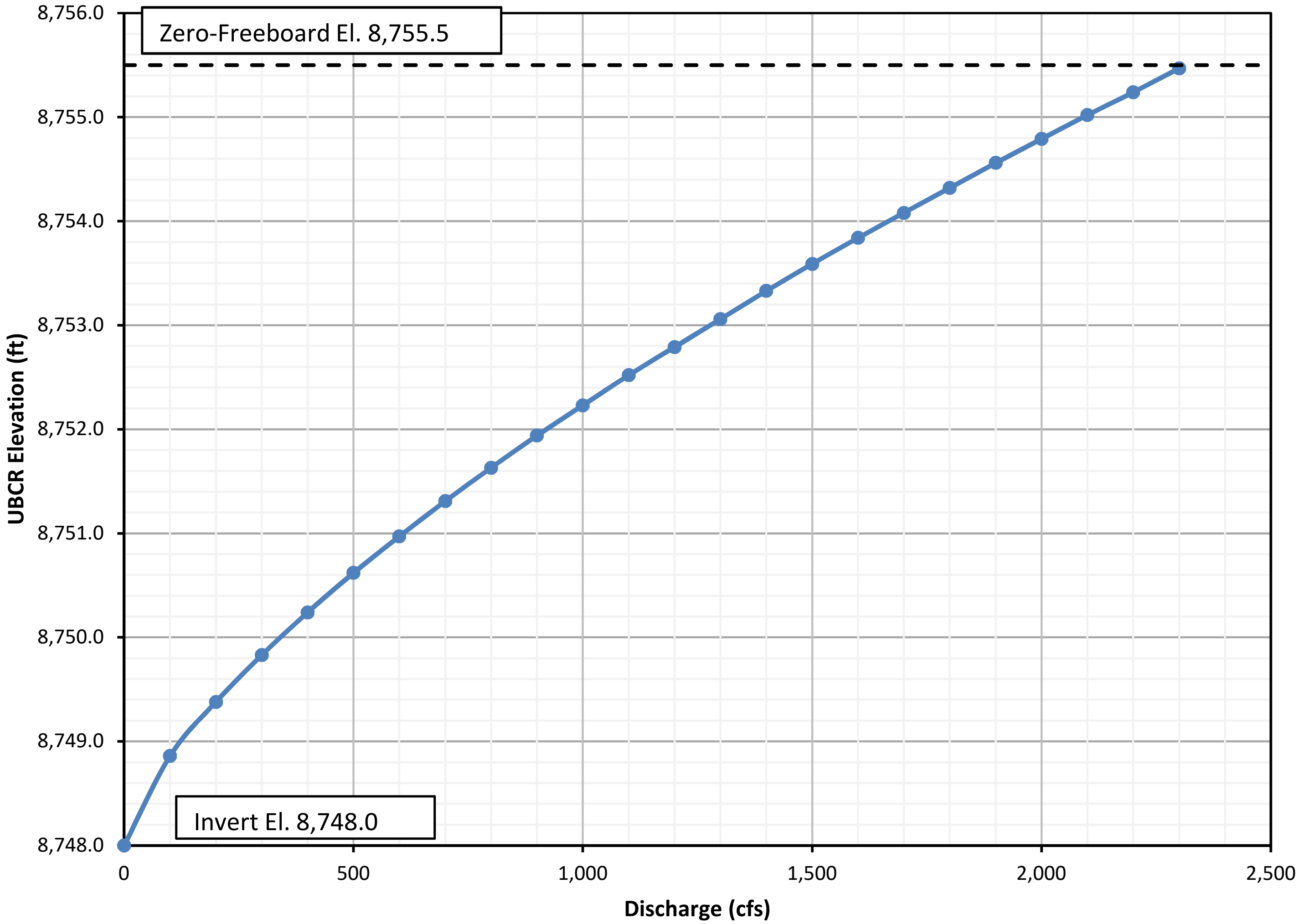
GEI Consultants

Project 1801834

PROPOSED SPILLWAY
SECTION VIEW & DETAILS

SEPTEMBER 2019 FIGURE B.2

L:\Working\Blue Lake Reservoir Company\CAD\Spillway_Chute



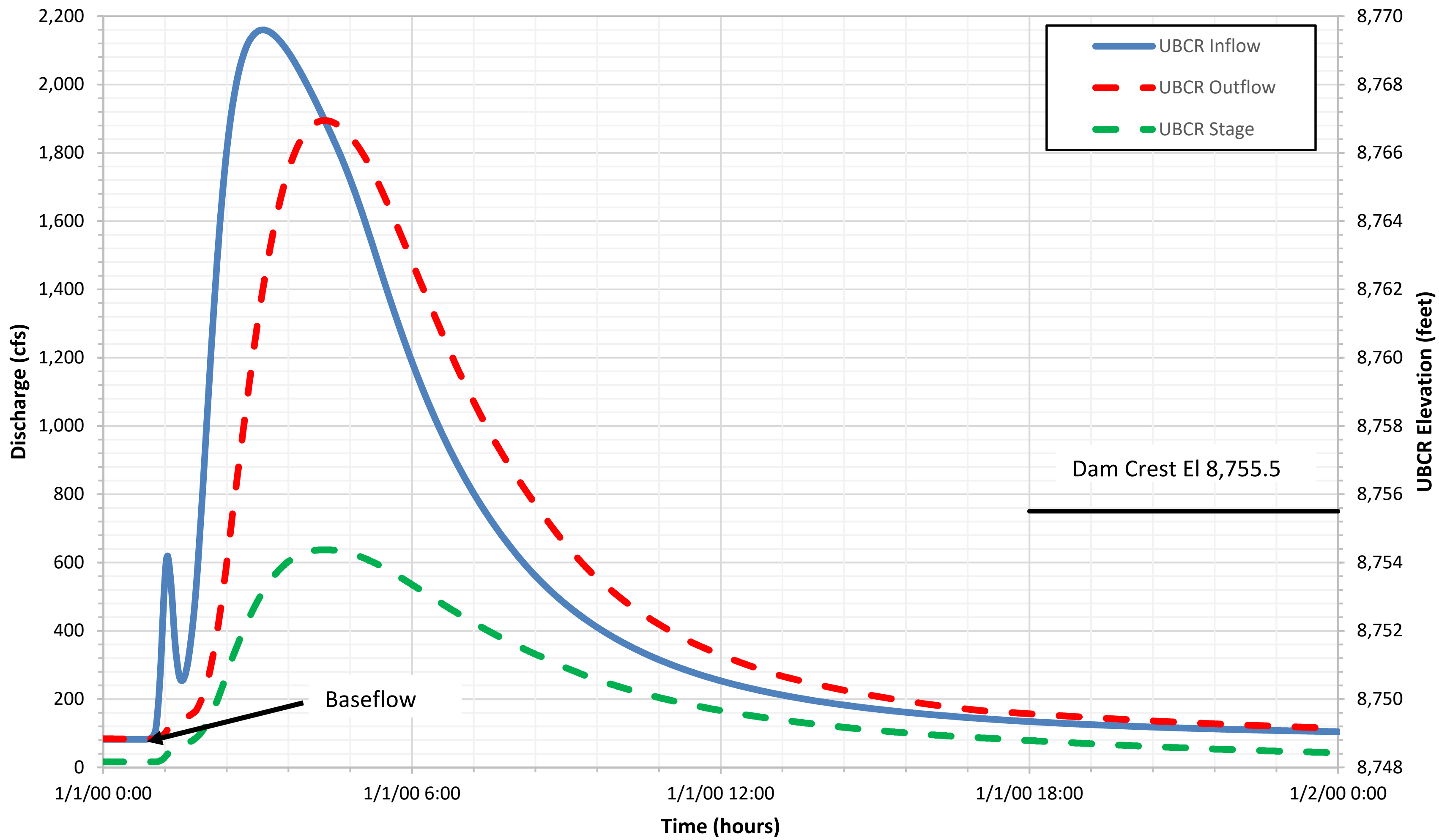
Discharge	WSEL
cfs	ft
0	8,748.0
100	8,748.9
200	8,749.4
300	8,749.8
400	8,750.2
500	8,750.6
600	8,751.0
700	8,751.3
800	8,751.6
900	8,751.9
1,000	8,752.2
1,100	8,752.5
1,200	8,752.8
1,300	8,753.1
1,400	8,753.3
1,500	8,753.6
1,600	8,753.8
1,700	8,754.1
1,800	8,754.3
1,900	8,754.6
2,000	8,754.8
2,100	8,755.0
2,200	8,755.2
2,300	8,755.5

Upper Black Creek Reservoir Reconstruction Project
Client: The Summit Trust
Location: Summit County, Colorado

GEI Consultants

Spillway Rating Curve

Project 1801834 | **September 2019** | **Figure B.3**

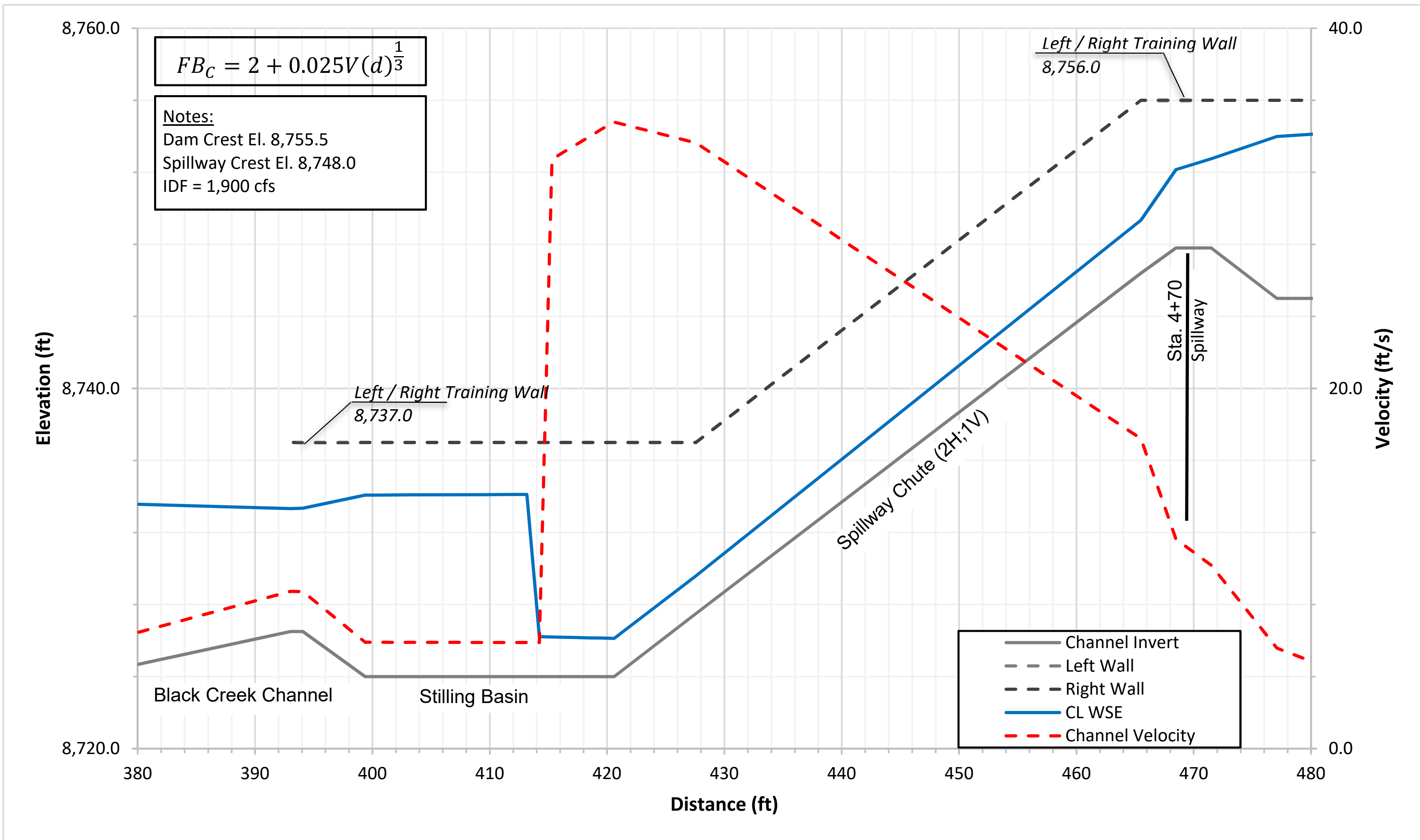


Upper Black Creek Reservoir Reconstruction Project
Client: The Summit Trust
Location: Summit County, Colorado

GEI Consultants

1,000-yr, 6-Hour Local Storm
UBCR (Proposed Conditions)

Project 1801834 | **September 2019** | **Figure B.4**



Upper Black Creek Reservoir Reconstruction Project
Client: The Summit Trust
Location: Summit County, Colorado

GEI Consultants

1,000-yr, 6-Hour Local Storm Freeboard Check

Project 1801834 | **September 2019** | **Figure B.5**

GEI Consultants

CLIENT:	Blue Lake Reservoir Company		
PROJECT:	Upper Black Creek Reservoir Reconstruction	Project: 1801834	Pages:
SUBJECT:	Type III Stilling Basin Design	Date: 8/26/2019	By: P. Drew
		Checked:	By: N. Miller
		Approved:	By: C. Masching

Purpose: Design chute and stilling basin structure required at Upper Black Creek Spillway

Procedure: Follow design steps presented in *Design of Small Canals - Ch. II Conveyance Structures - F. Chutes*.

References: USBR (1978). Design of Small Canal Structures.
USBR (1984). Engineering Monograph No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators.

Input Variables:

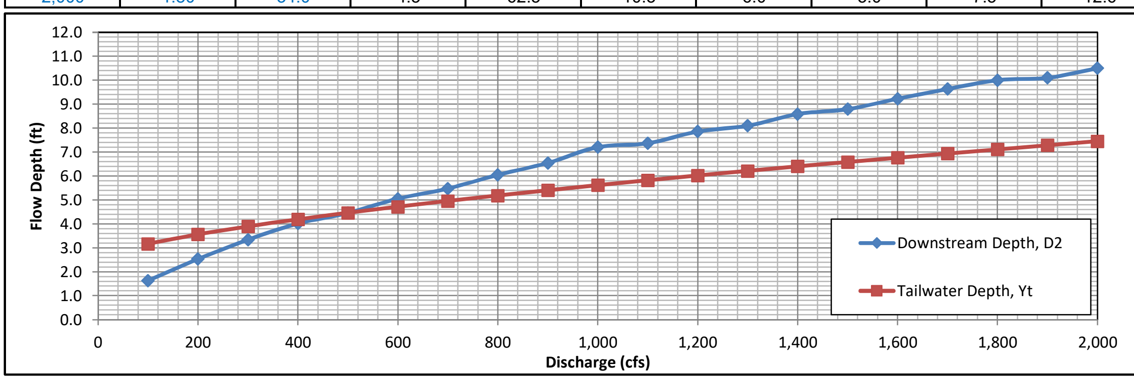
Start Invert El.:	8,748.0	ft
Downstream Channel El.:	8,724.0	ft
Chute Slope:	2.0	H:1V
Chute Width, B:	32.0	ft

Step 1: Range of Inflow Variables From HEC-RAS Model.

Process: Iterate end sill height to obtain a tailwater depth that is greater than or equal to the conjugate depth of the hydraulic jump, for all expected flows. It is assumed that the depth of tailwater is equal to the sill height plus the critical depth of flow over the end sill.

End Sill Height Above Basin Floor: **2.5** ft

Discharge, Q (cfs)	Upstream Depth, D1 (ft)	Upstream Velocity, V1 (ft/sec)	Upstream Froude #, F1	Unit Discharge, q (cfs/ft)	Downstream Depth, D2 (ft)	Velocity Downstream, V2 (ft/sec)	Sill Depth, Ys (ft)	Tailwater Depth, Yt (ft)	Tailwater Velocity, Vt (ft/sec)
100	0.30	13.0	4.2	3.1	1.6	1.9	0.7	3.2	4.7
200	0.40	17.3	4.8	6.3	2.5	2.5	1.1	3.6	5.9
300	0.50	20.3	5.1	9.4	3.3	2.8	1.4	3.9	6.7
400	0.60	22.3	5.1	12.5	4.0	3.1	1.7	4.2	7.4
500	0.60	24.6	5.6	15.6	4.5	3.5	2.0	4.5	8.0
600	0.80	24.4	4.8	18.8	5.1	3.7	2.2	4.7	8.5
700	0.80	26.3	5.2	21.9	5.5	4.0	2.5	5.0	8.9
800	0.90	27.4	5.1	25.0	6.0	4.1	2.7	5.2	9.3
900	1.00	28.2	5.0	28.1	6.5	4.3	2.9	5.4	9.7
1,000	1.10	29.6	5.0	31.3	7.2	4.3	3.1	5.6	10.0
1,100	1.10	30.2	5.1	34.4	7.4	4.7	3.3	5.8	10.3
1,200	1.20	30.9	5.0	37.5	7.9	4.8	3.5	6.0	10.6
1,300	1.30	30.7	4.7	40.6	8.1	5.0	3.7	6.2	10.9
1,400	1.40	31.4	4.7	43.8	8.6	5.1	3.9	6.4	11.2
1,500	1.40	32.1	4.8	46.9	8.8	5.3	4.1	6.6	11.5
1,600	1.50	32.6	4.7	50.0	9.2	5.4	4.3	6.8	11.7
1,700	1.60	33.0	4.6	53.1	9.6	5.5	4.4	6.9	12.0
1,800	1.70	33.3	4.5	56.3	10.0	5.6	4.6	7.1	12.2
1,900	1.70	33.6	4.5	59.4	10.1	5.9	4.8	7.3	12.4
2,000	1.80	34.0	4.5	62.5	10.5	6.0	5.0	7.5	12.6



Step 2: Determine Basin Length, L.

Maximum Froude #, Fr1:	5.6	
Maximum D2:	10.50	ft
Ratio L/D2:	2.4	(from Figure 12 EM-25)
Calculated Basin Length:	25.2	ft
Use Basin Length:	**26.0**	**ft**

GEI Consultants

CLIENT:	Blue Lake Reservoir Company		
PROJECT:	Upper Black Creek Reservoir Reconstruction	Project: 1801834	Pages:
SUBJECT:	Type III Stilling Basin Design	Date: 8/26/2019	By: P. Drew
		Checked:	By: N. Miller
		Approved:	By: C. Masching

Step 3: Determine Chute Blocks and Baffle Pier Dimensions.

Chute Blocks:

Height:	**21.6**	**inches**	=D1 at max. flow, Min. = 8"
Width:	**21.6**	**inches**	=D1 at max. flow, Min. = 8"
Spacing:	**21.6**	**inches**	=D1 at max. flow, Min. = 8"
# of Full Blocks:	9.0		
Partial Blocks:	0.0		

Baffle Piers:

Maximum Froude #, Fr1:	5.6		
Maximum D1:	1.80	ft	
Ratio H3/D1:	1.5	(from chart)	
Baffle Piers Height, H3:	33.4	inches	
Use Baffle Peir Height, H3:	**40.0**	**inches**	
Baffle Peir Width, Pw:	**30.0**	**inches**	=0.75(H3)
Top Width:	**8.0**	**inches**	=0.20(H3)
Spacing, Ps:	**30.0**	**inches**	=0.75(H3)
# of Blocks:	7.0		
Distance to Baffle Face:	**8.40**	**ft**	=0.8(D2)

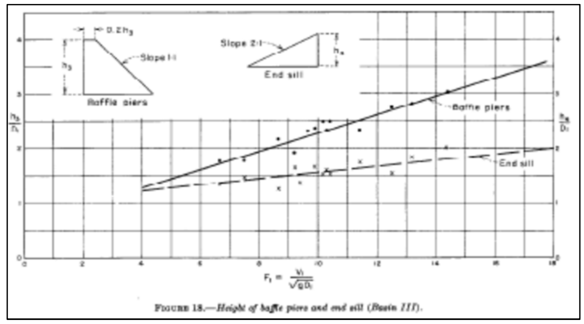


FIGURE 18.—Height of baffle piers and end sill (Basin III).

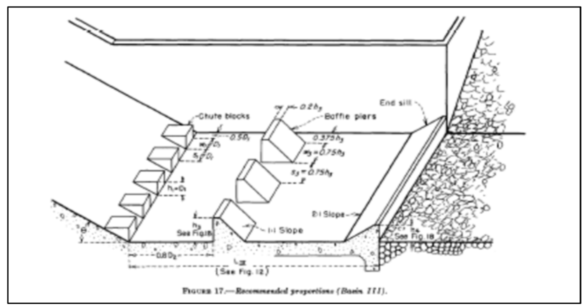


FIGURE 17.—Recommended proportions (Basin III).

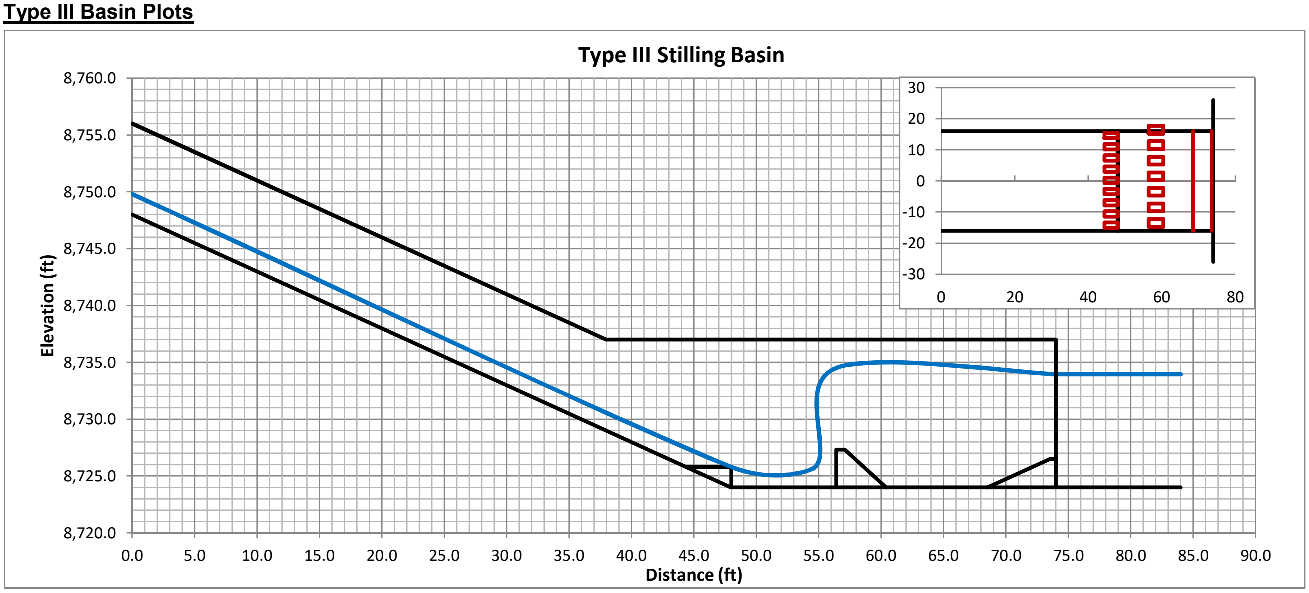
Step 4: Determine End Sill Dimensions.

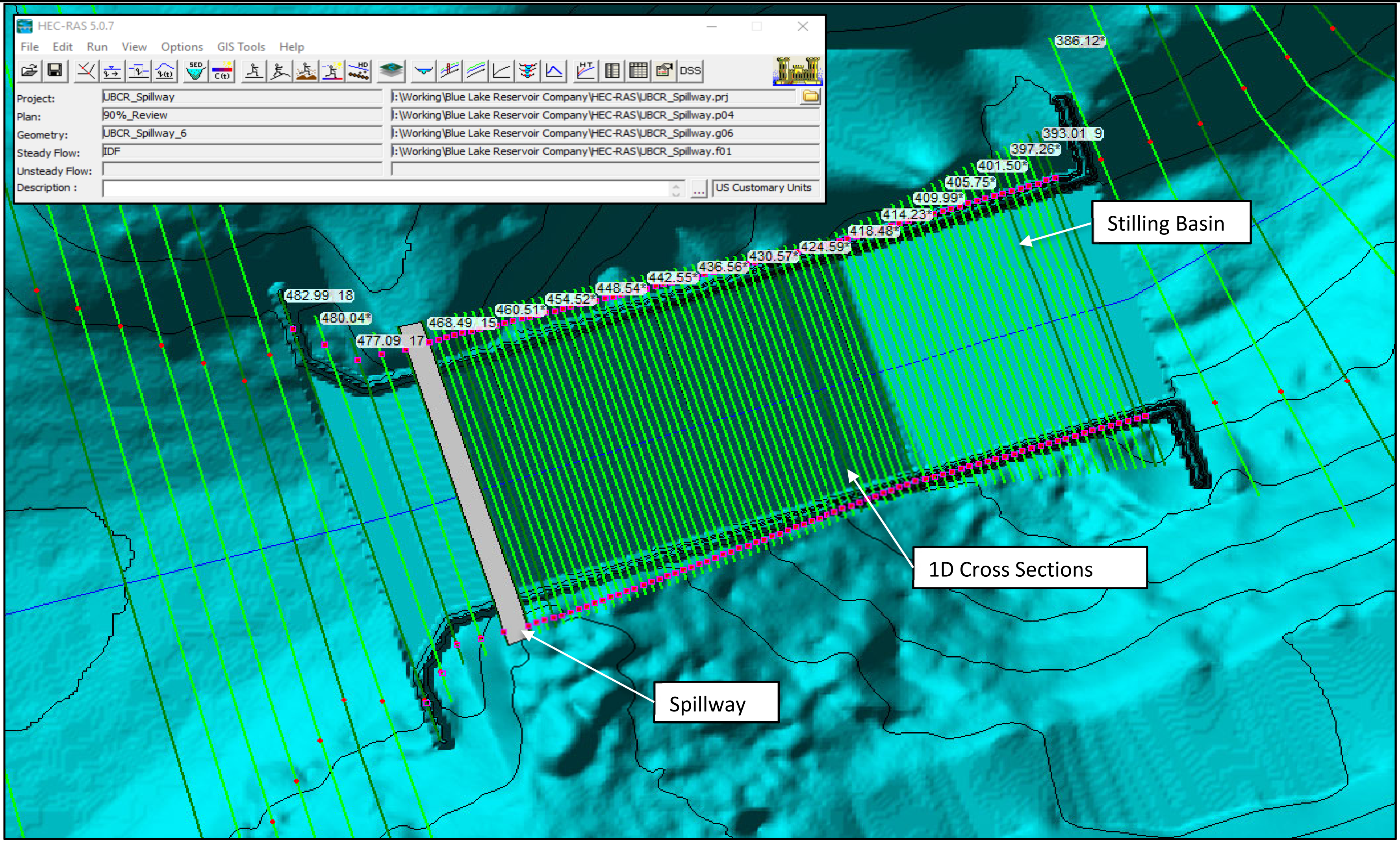
Maximum Froude #, Fr1:	5.6		
Maximum D1:	1.80	ft	
Ratio H4/D1:	1.3	(from chart)	
End Sill Minimum Height, H4:	28.3	inches	
Top Width:	6.0	inches	
US Slope of Sill:	2.0	H:1V	
Use End Sill Height,H4:	**2.50**	**ft**	
Drop to DS Channel:	**0.0**	**ft**	
Final Basin Floor El.:	**8724.0**	ft	=(Int El.)-H4+Drop

Step 5: Wall Heights

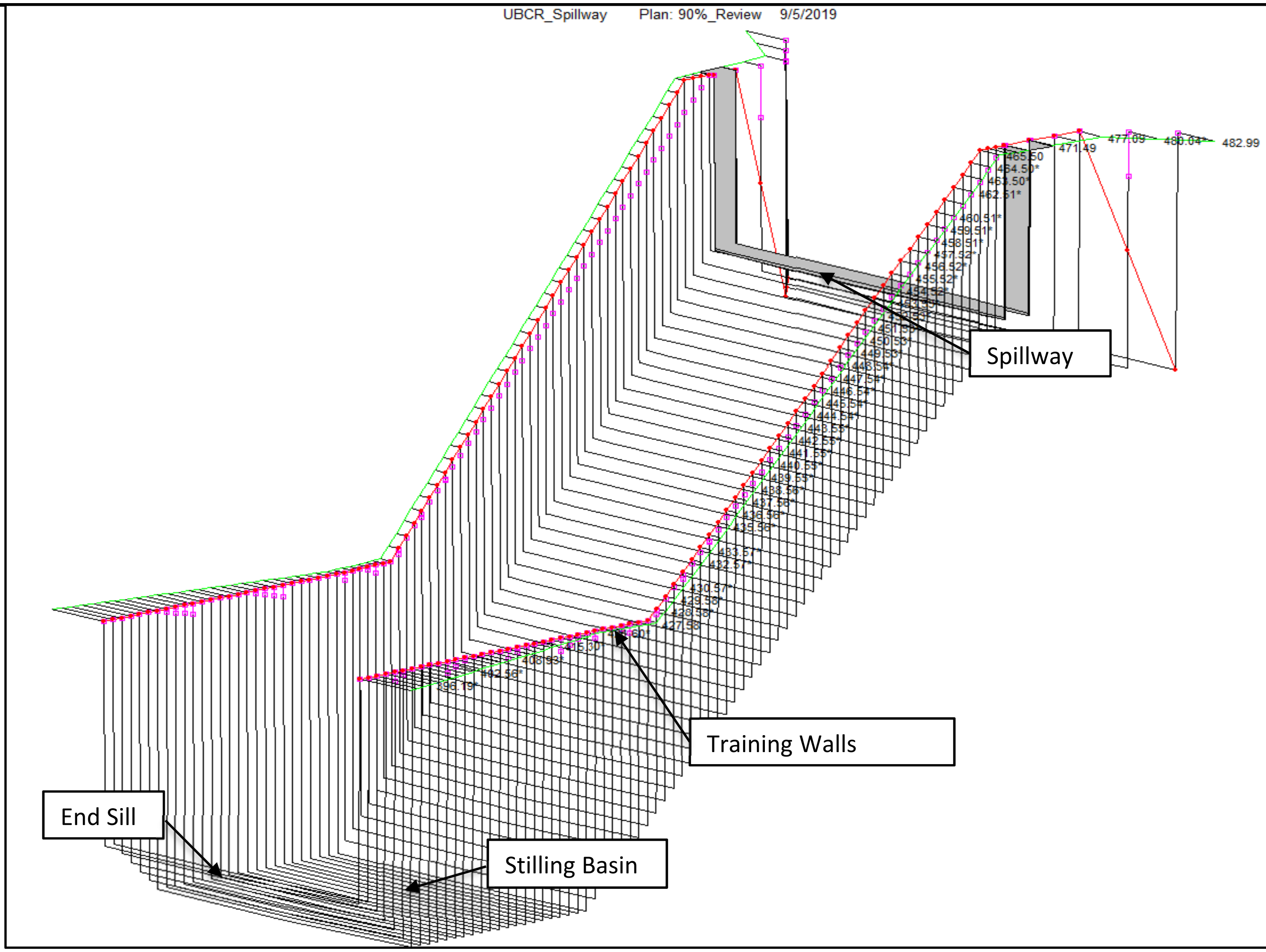
Inlet Structure Wall Height:	8	ft	
Chute Wall Height:	**9.5**	**ft**	
Basin Freeboard:	**2.0**	**ft**	
Basin Wall Height:	**13.0**	**ft**	
Wing Wall Length:	**10.0**	**ft**	=0.75*(basin wall height)

Type III Basin Plots





Upper Black Creek Reservoir Reconstruction Project **Client: The Summit Trust** **Location: Summit County, Colorado**	GEI Consultants	**HEC-RAS Basemap**
	Project 1801834	**September 2019** — **Figure B.6**

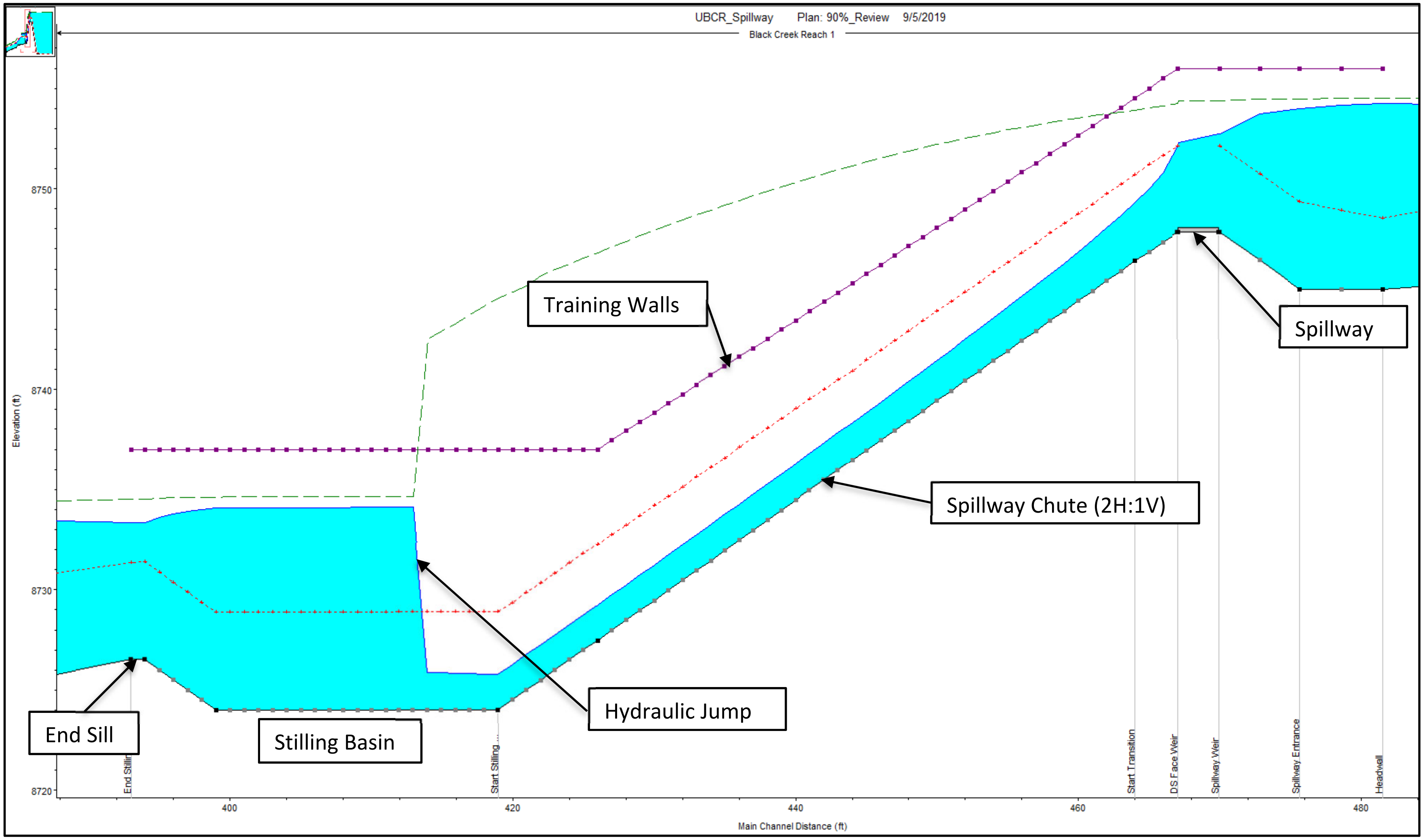


Upper Black Creek Reservoir Reconstruction Project
Client: The Summit Trust
Location: Summit County, Colorado

GEI Consultants

HEC-RAS Isometric View

Project 1801834 | **September 2019** | **Figure B.7**



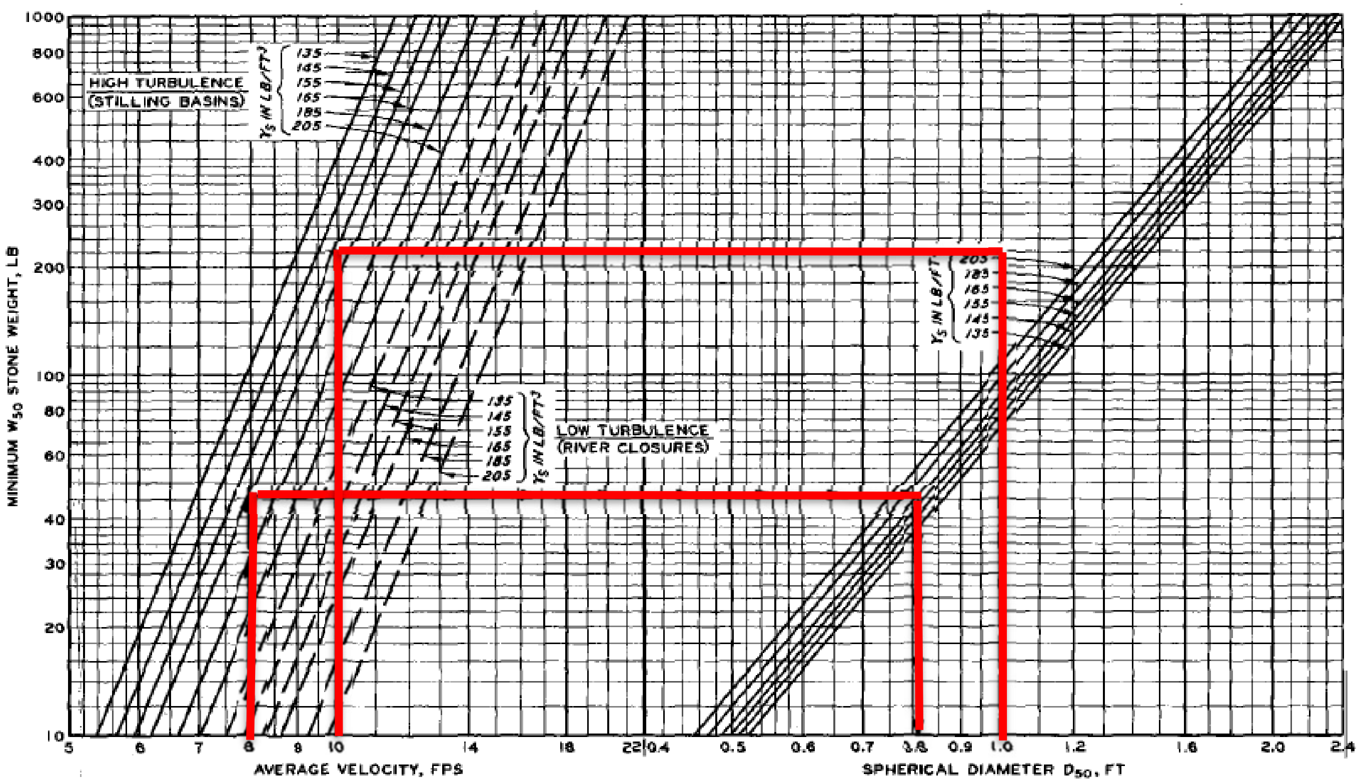
Upper Black Creek Reservoir Reconstruction Project **Client: The Summit Trust** **Location: Summit County, Colorado**	GEI Consultants	**HEC-RAS Profile**	
	Project 1801834	**September 2019**	**Figure B.8**

HEC-RAS Plan: IDF River: Black Creek Reach: Reach 1

Reach	River Sta	Profile	Q Total	W.S. Elev	Vel Total	Min Ch El	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # XS
			(cfs)	(ft)	(ft/s)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	933.09 24	100	100	8748.9	0.0	8713.2	8713.3	8748.9	0.0000	0.0	28791	932	0.0
Reach 1	933.09 24	200	200	8749.4	0.0	8713.2	8713.4	8749.4	0.0000	0.0	29275	934	0.0
Reach 1	933.09 24	300	300	8749.8	0.0	8713.2	8713.4	8749.8	0.0000	0.0	29694	935	0.0
Reach 1	933.09 24	400	400	8750.2	0.0	8713.2	8713.4	8750.2	0.0000	0.0	30078	937	0.0
Reach 1	933.09 24	500	500	8750.6	0.0	8713.2	8713.5	8750.6	0.0000	0.0	30436	938	0.0
Reach 1	933.09 24	600	600	8751.0	0.0	8713.2	8713.5	8751.0	0.0000	0.0	30770	940	0.0
Reach 1	933.09 24	700	700	8751.3	0.0	8713.2	8713.5	8751.3	0.0000	0.0	31089	941	0.0
Reach 1	933.09 24	800	800	8751.6	0.0	8713.2	8713.6	8751.6	0.0000	0.0	31388	942	0.0
Reach 1	933.09 24	900	900	8751.9	0.0	8713.2	8713.6	8751.9	0.0000	0.0	31677	943	0.0
Reach 1	933.09 24	1000	1000	8752.2	0.0	8713.2	8713.6	8752.2	0.0000	0.0	31954	945	0.0
Reach 1	933.09 24	1100	1100	8752.5	0.0	8713.2	8713.6	8752.5	0.0000	0.0	32225	949	0.0
Reach 1	933.09 24	1200	1200	8752.8	0.0	8713.2	8713.7	8752.8	0.0000	0.0	32488	953	0.0
Reach 1	933.09 24	1300	1300	8753.1	0.0	8713.2	8713.7	8753.1	0.0000	0.0	32745	957	0.0
Reach 1	933.09 24	1400	1400	8753.3	0.0	8713.2	8713.7	8753.3	0.0000	0.0	33001	961	0.0
Reach 1	933.09 24	1500	1500	8753.6	0.0	8713.2	8713.7	8753.6	0.0000	0.0	33249	965	0.0
Reach 1	933.09 24	1600	1600	8753.8	0.0	8713.2	8713.8	8753.8	0.0000	0.0	33493	969	0.0
Reach 1	933.09 24	1700	1700	8754.1	0.1	8713.2	8713.8	8754.1	0.0000	0.1	33727	973	0.0
Reach 1	933.09 24	1800	1800	8754.3	0.1	8713.2	8713.8	8754.3	0.0000	0.1	33963	979	0.0
Reach 1	933.09 24	1900	1900	8754.6	0.1	8713.2	8713.8	8754.6	0.0000	0.1	34197	981	0.0
Reach 1	933.09 24	2000	2000	8754.8	0.1	8713.2	8713.8	8754.8	0.0000	0.1	34424	983	0.0
Reach 1	933.09 24	2100	2100	8755.0	0.1	8713.2	8713.9	8755.0	0.0000	0.1	34646	984	0.0
Reach 1	933.09 24	2200	2200	8755.2	0.1	8713.2	8713.9	8755.2	0.0000	0.1	34869	985	0.0
Reach 1	933.09 24	2300	2300	8755.5	0.1	8713.2	8713.9	8755.5	0.0000	0.1	35089	987	0.0
Reach 1	933.09 24	2400	2400	8755.7	0.1	8713.2	8713.9	8755.7	0.0000	0.1	35301	988	0.0
Reach 1	933.09 24	2500	2500	8755.9	0.1	8713.2	8713.9	8755.9	0.0000	0.1	35513	989	0.0

HEC-RAS Plan: IDF River: Black Creek Reach: Reach 1 Profile: 1900

Reach	River Sta	Profile	Q Total	W.S. Elev	Vel Total	Min Ch El	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # XS
			(cfs)	(ft)	(ft/s)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	933.09 24	1900	1900	8754.6	0.1	8713.2	8713.8	8754.6	0.0000	0.1	34197	981	0.0
Reach 1	830.29 23	1900	1900	8754.6	0.1	8713.2	8713.8	8754.6	0.0000	0.1	34614	955	0.0
Reach 1	724.89 22	1900	1900	8754.6	0.1	8713.2	8713.8	8754.6	0.0000	0.1	36921	1043	0.0
Reach 1	621.99 21	1900	1900	8754.6	0.1	8713.2	8714.5	8754.6	0.0000	0.1	26122	1012	0.0
Reach 1	511.49 20	1900	1900	8754.4	2.7	8744.9	8748.9	8754.5	0.0003	2.9	711	179	0.3
Reach 1	491.19 19	1900	1900	8754.2	4.0	8745.4	8749.9	8754.5	0.0007	4.5	479	74	0.3
Reach 1	482.99 18	1900	1900	8754.2	4.1	8745.0	8748.6	8754.5	0.0000	4.1	464	50	0.2
Reach 1	477.09 17	1900	1900	8754.0	5.6	8745.0	8749.3	8754.5	0.0001	5.6	341	38	0.3
Reach 1	471.49 16	1900	1900	8752.8	10.2	8747.8	8752.1	8754.4	0.0008	10.2	187	38	0.8
Reach 1	470		Inl Struct										
Reach 1	468.49 15	1900	1900	8752.1	11.6	8747.8	8752.1	8754.2	0.0103	11.6	163	38	1.0
Reach 1	465.50 14	1900	1900	8749.3	17.2	8746.4	8750.7	8753.9	0.0352	17.2	110	38	1.8
Reach 1	427.58 13	1900	1900	8729.2	33.6	8727.5	8732.3	8746.8	0.2554	33.6	56	33	4.5
Reach 1	420.60 12	1900	1900	8725.8	34.8	8724.0	8728.9	8744.6	0.2758	34.8	55	32	4.7
Reach 1	399.38 11	1900	1900	8734.1	5.9	8724.0	8728.9	8734.6	0.0028	5.9	322	32	0.3
Reach 1	394.07 10	1900	1900	8733.3	8.7	8726.5	8731.4	8734.5	0.0102	8.7	218	32	0.6
Reach 1	393.01 9	1900	1900	8733.3	8.7	8726.5	8731.4	8734.5	0.0107	8.7	218	32	0.6
Reach 1	372.35 8	1900	1900	8733.7	5.1	8723.6	8729.4	8734.2	0.0023	5.7	372	57	0.4
Reach 1	347.15 7	1900	1900	8733.6	5.1	8723.3	8729.8	8734.1	0.0027	5.8	370	65	0.4
Reach 1	307.63 6	1900	1900	8731.2	12.5	8723.3	8731.2	8733.6	0.0263	12.6	151	32	1.0
Reach 1	275.35 5	1900	1900	8729.1	13.9	8722.4	8729.8	8732.2	0.0416	14.0	136	38	1.3
Reach 1	242.23 4	1900	1900	8727.0	14.4	8722.5	8728.0	8730.3	0.0632	14.5	132	52	1.6
Reach 1	200.62 3	1900	1900	8723.9	14.7	8719.8	8724.9	8727.3	0.0752	14.8	129	53	1.7
Reach 1	106.02 2	1900	1900	8722.7	9.3	8717.9	8722.2	8724.0	0.0179	9.3	204	54	0.8
Reach 1	0.00 1	1900	1900	8719.8	10.3	8715.3	8720.0	8721.5	0.0323	10.3	185	68	1.1



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_s}\right)^{1/3}$$

WHERE: V = VELOCITY, FPS
γ_s = SPECIFIC STONE WEIGHT, LB/FT^3
γ_w = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3
W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

Upper Black Creek Reservoir Reconstruction Project
Client: The Summit Trust
Location: Summit County, Colorado

GEI Consultants

Stone Stability Velocity vs. Stone Diameter

Project 1801834	**September 2019**	**Figure B.9**

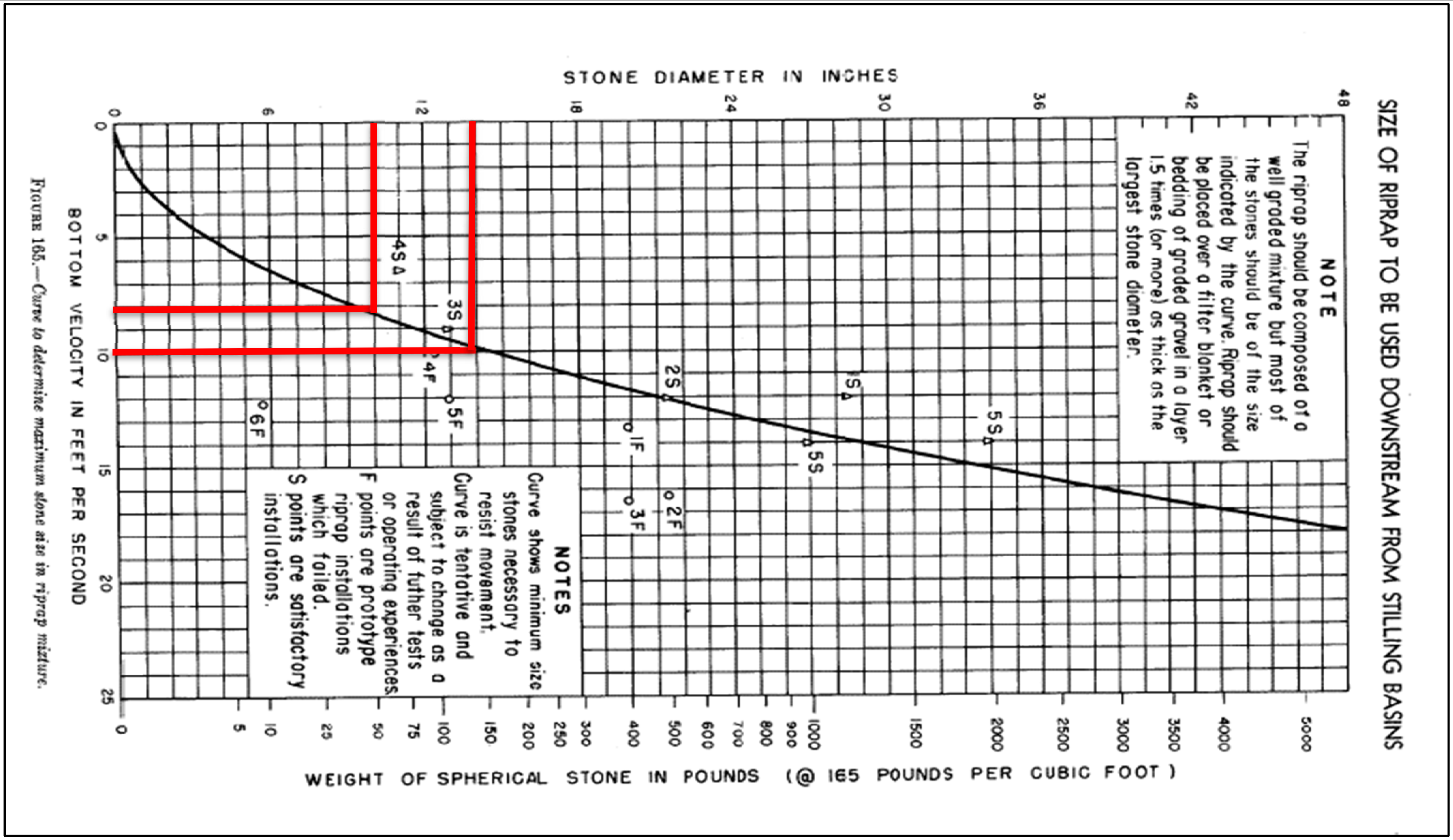


FIGURE 165.—*Curve to determine maximum stone size in riprap mixture.*

Upper Black Creek Reservoir Reconstruction Project **Client: The Summit Trust** **Location: Summit County, Colorado**	GEI Consultants	**Stone Stability Velocity vs. Stone Diameter**
Project 1801834	**September 2019**	**Figure B.10**

Appendix C Geotechnical Calculations

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (upd

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

39.804

Time Horizon

Return period in years

10000

Longitude

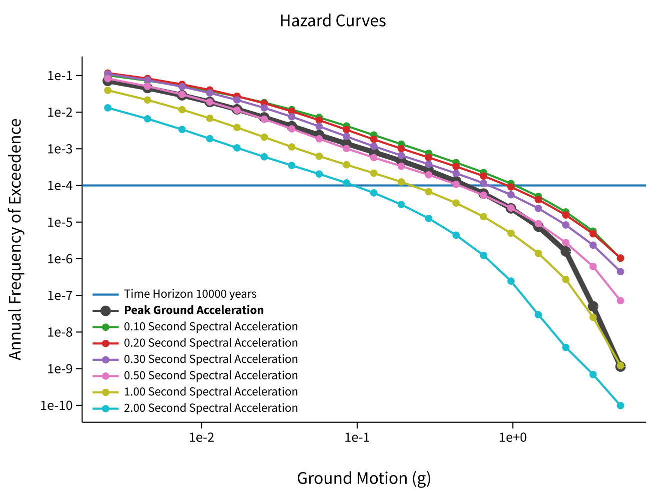
Decimal degrees, negative values for western longitudes

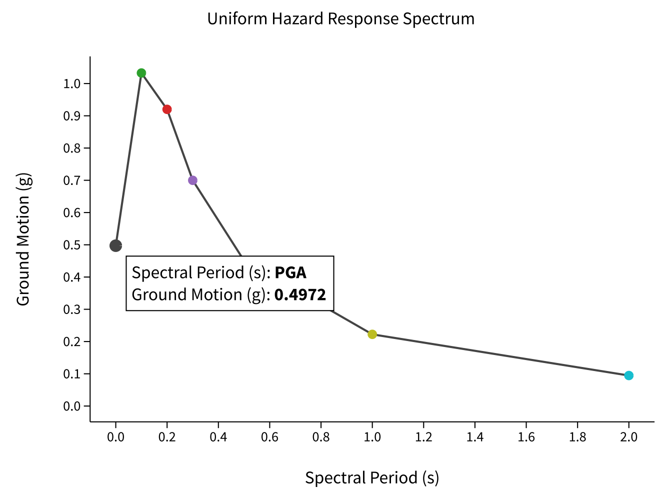
-106.264

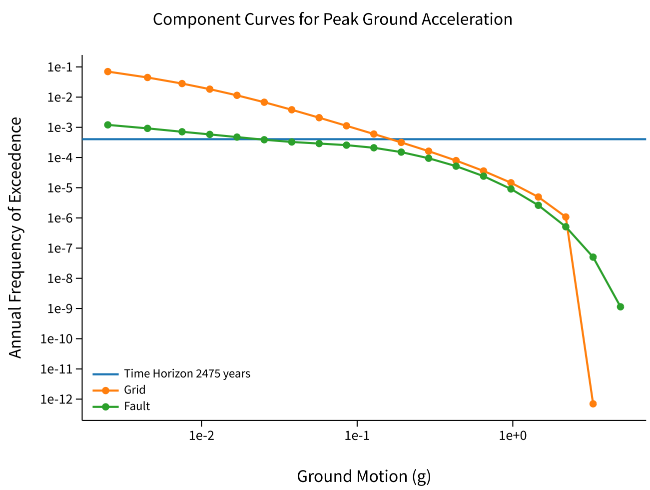
Site Class

760 m/s (B/C boundary)

Hazard Curve





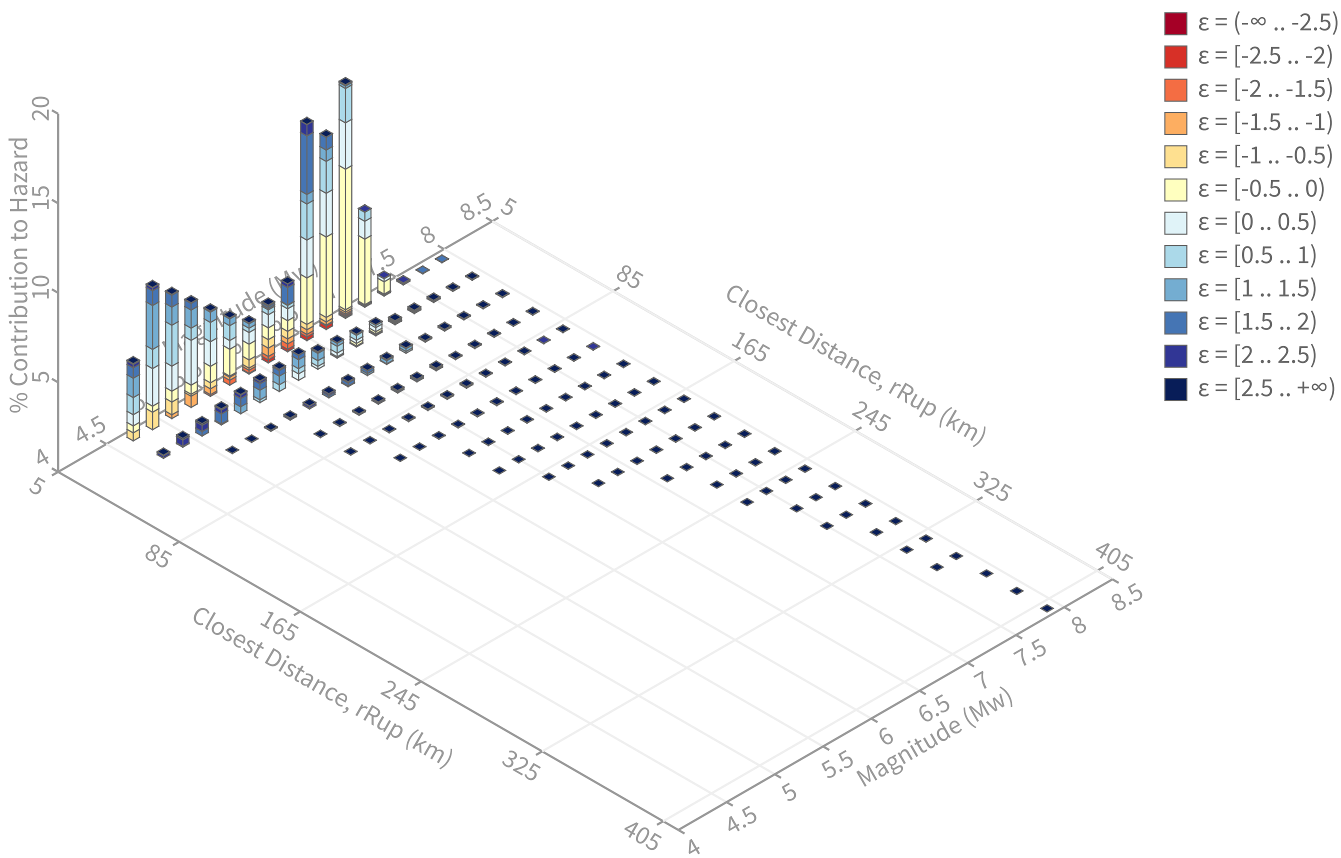


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 10000 yrs
Exceedance rate: 0.0001 yr^{-1}
PGA ground motion: 0.49721063 g

Recovered targets

Return period: 9938.763 yrs
Exceedance rate: 0.00010061614 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.52 %

Mean (over all sources)

m: 6.08
r: 11.9 km
ε_0: 0.43 σ

Mode (largest m-r bin)

m: 6.9
r: 2.82 km
ε_0: -0.04 σ
Contribution: 13.04 %

Mode (largest m-r-ε_0 bin)

m: 6.9
r: 2.28 km
ε_0: -0.19 σ
Contribution: 7.85 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
SSCn Fixed Smoothing Zone 9 (opt)	Grid							20.46
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	6.59
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	4.05
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	2.40
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	2.30
PointSourceFinite: -106.264, 39.961		16.97	5.68	0.70	106.264°W	39.961°N	0.00	1.94
USGS Fixed Smoothing Zone 3 (opt)	Grid							20.46
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	6.59
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	4.05
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	2.40
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	2.30
PointSourceFinite: -106.264, 39.961		16.97	5.68	0.70	106.264°W	39.961°N	0.00	1.94
Geologic Model Full Rupture	Fault							14.09
Gore Range frontal 50		2.06	6.95	-0.04	106.295°W	39.797°N	253.81	6.21
Williams Fork Mountains 50		10.86	6.53	1.71	106.154°W	39.854°N	59.16	2.27
Gore Range frontal 35		1.56	6.95	-0.15	106.295°W	39.797°N	253.81	2.23
Gore Range frontal 65		2.43	6.95	0.09	106.295°W	39.797°N	253.81	1.88
Geologic Model Partial Rupture	Fault							11.79
Gore Range frontal 50		2.54	6.71	0.16	106.295°W	39.797°N	253.81	6.22
Gore Range frontal 65		2.92	6.71	0.21	106.295°W	39.797°N	253.81	2.01
Gore Range frontal 35		2.34	6.71	0.18	106.295°W	39.797°N	253.81	1.96
SSCn Adaptive Smoothing Zone 9 (opt)	Grid							9.78
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	2.97
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	1.87
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	1.11
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	1.02
USGS Adaptive Smoothing Zone 3 (opt)	Grid							9.78
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	2.97
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	1.87
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	1.11
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	1.02
Bird Model Full Rupture	Fault							5.25
Gore Range frontal 50		2.06	6.95	-0.04	106.295°W	39.797°N	253.81	2.68
Bird Model Partial Rupture	Fault							4.73
Gore Range frontal 50		2.54	6.71	0.16	106.295°W	39.797°N	253.81	2.68
Zeng Model Full Rupture	Fault							1.92
Zeng Model Partial Rupture	Fault							1.65

GEI Consultants	Client	BLUE LAKE RESERVOIR CO.			Page	1 OF 2
	Project	UPPER BLACK CREEK RES.			Pg. Rev.	
	By	A. STEIN	Chk.	G. Williams	App.	
	Date	4/23/20	Date	4/24/20	Date	
Project No.	1801834	Document No.				
Subject	SEEPAGE MATERIAL PROPERTIES					

OBJECTIVE: Determine hydraulic conductivities and anisotropic ratios for materials in the seepage analysis.

REFERENCES: GEI Geotechnical Investigation dated Feb. 16, 2018 including Memo, boring logs, and lab data
USBR Design of Small Dams, 3rd Ed. 1987
Das Principles of Foundation Engineering, 5th Ed
Peck, Hanson, Thornburn Foundation Engineering, 2nd Ed.

FROM BORING LOGS: Boring B-1 located on left abutment of spillway. Boring B-2 located in centerline of spillway roughly 5 feet upstream of spillway apron. Both borings have varying layers of clayey sands, silty sands and gravel ranging from 1.5 to 5.5 feet thick with 21-43 % fines.

Native Soil

Consider a homogeneous material of SM-SC with higher permeability.

FROM DESIGN OF SMALL DAMS: COEFFICIENT OF PERMEABILITY, k

SM 1×10^{-3} to 1×10^{-8} Avg $\simeq 1 \times 10^{-4}$ cm/sec

SC 1×10^{-5} to 5×10^{-8} Avg $\simeq 1 \times 10^{-5}$ cm/sec

FROM FOUNDATION ENGINEERING:

Mixtures of sand gravel and clayey sands, fines
1×10^{-4} to 1×10^{-5}

FROM PRINC OF FDN ENGINEERING (DAS):

Fine sand, silty sand 10^{-3} to 10^{-5}

GEI Consultants	Client	BLUE LAKE RESERVOIR CO.			Page	2 OF 2
	Project	UPPER BLACK CREEK RES.			Pg. Rev.	
	By	A. STEIN	Chk.	G. WILLIAMS	App.	
	Date	4/23/20	Date	4/23/20	Date	
Project No.	1801834	Document No.				
Subject	SEEPAGE MATERIAL PROPERTIES					

For seepage analysis, a higher hydraulic conductivity is more conservative.

$$\boxed{\begin{array}{l} k_h = 1 \times 10^{-5} \text{ cm/s} \\ k_h/k_y = 10 \end{array}}$$

$= 3.28 \times 10^{-7}$ ft/sec

(Geostudio input is opposite, so $k_y/k_h = 0.1$)

Bedrock

Bedrock not encountered in the borings, so model assumes no bedrock boundary conditions.

Embankment Fill

FROM DESIGN OF SMALL DAMS:

Assume the native soil is compacted in lifts so k_h is the same and the ratio is decreased.

$$\boxed{\begin{array}{l} k_h = 1 \times 10^{-5} \text{ cm/s} \\ k_h/k_y = 4 \end{array}}$$

($k_y/k_h = 0.25$)

GEI Consultants

		Project	Page
Client	Blue Lake Reservoir Co	By G. Williams	Date 4/22/2020
Subject		Checked A. Stein	Date 4/24/2020
		Approved	Date

Estimate Sheet Pile hydraulic conductivity

From Manufacturer for water proofing joints

$\rho = 1 \times 10^{-8}$ to 1×10^{-10} cm/sec

from attached 3×10^{-8} cm/sec

$K/d = \rho/b$

K = hydraulic conductivity for model

d = model thickness

ρ = from manufacturer

b = width of panel, say 2.0 ft

use $d = 0.375$ inch $= 0.0095$ m

$\rho = 3 \times 10^{-8}$ cm/sec $= 3 \times 10^{-10}$ m/sec

$b = 2.0$ ft $= 0.6$ m

$$K = 0.0095 \left(\frac{3 \times 10^{-10}}{0.6} \right) = 4.75 \times 10^{-12} \text{ m/s} = 4.75 \times 10^{-10} \text{ cm/sec}$$

USE: $K = 4.75 \times 10^{-10}$ cm/sec $= 1.56 \times 10^{-11}$ ft/sec

$d = 0.375$ inch $= 0.03125$ ft

3. In situ measurements

In order to allow the design engineer to make practical use of equation (3) Delft Geotechnics and ProfilARBED have carried out field tests on a large number of filler materials. The results of these tests yield values for ρ.

To expose the filler material to extreme site conditions, the sheet piles for the test wall have been driven in by vibrohammer. Each filler material has been applied in several joints.

The discharge through each joint was measured as a function of the applied pressure drop using a special test apparatus, see Fig. 4. The time dependent behaviour is monitored by taking readings at specific time intervals.

Table 1 shows the relevant criteria for selecting a water sealing system for an SSP wall and the range of values obtained from the tests for different types of filler matrials (bituminous ones as well as water swelling products); the results of the empty joints are also shown. It is most important to note that the r-values obtained for empty joints strongly depend on the soil properties, the variations being very large.

The test results are plotted in Fig. 5 which generally confirms that the hypothesis which leads up to formula (3) is well-founded (see also Fig. 3), at least for a certain pressure range.

The testing programme carried out by Delft Geotechnics and ProfilARBED, clearly demonstrates that the use of filler products in the joints of a SSP wall considerably reduces the seepage rate.

In addition it transpires that the filler material in the joints remains in place, even after the application of a vibrohammer - provided the manufacturer's specifications are strictly adhered to and the special tools, as developed by ProfilARBED for the implantation of the filler materials, are deployed.

Table 1

WATERTIGHTENING SYSTEM	**ρ [10^{-9}m/s]** 100kPa	200kPa	**APPLICATION OF THE SYSTEM**	**COSTS RATIO ****
EMPTY JOINTS	> 100	*	-	0
BITUMINOUS FILLER MATERIAL	< 60	not recommended	EASY	1
WATERSWELLING PRODUCT	0.3	0.3	WITH CARE	2
WELDING OF THE JOINTS	0	0	ONLY AFTER EXCAVATION FOR THE INTERLOCK TO BE THREADED ON JOBSITE	5

* VALUE AVAILABLE ONLY AT 150 kPa: < 450

** The costs ratio = $\dfrac{\text{Costs of the watertightening system}}{\text{Costs of the bituminous filler material solution}}$

Note: See table inside rear cover for values to be used for a first order design approach.

For waterswelling product = 0.3 * 10-9 m/s = 3e-8 cm/sec

Waterloo indicates 1e-8 to 1e-10 cm/sec and panels are 2' wide.

4.2. Comparison with porous media flow

In everyday practice the design engineer often needs to compare the performance (seepage resistance) of a SSP wall with other types of wall design, such as a slurry wall (SW); a cut-off wall is an example where such a comparison is relevant. The slurry wall may be considered as a porous medium and the flow is governed by Darcy's law.

The comparison between the SSP wall and the slurry wall can be carried out by **assuming that the discharge per unit wall area is the same.** With the definitions given in fig. 7, Darcy's law (reference 2 and 4) yields a specific discharge:

$$\mathbf{Q_{sw} = K \bullet (\Delta p / \gamma) / d} \qquad (8)$$

where

d : thickness of the slurry wall, [m]
K : permeability of the wall in horizontal direction, [m/s]
p : pressure drop on both sides of the wall, [kPa]

The specific discharge for a SSP wall (Fig. 7) follows from (3), (6) and (7) with L = 1 m:

$$\mathbf{Q_{ssp} = (1 / b) \bullet \rho \bullet (\ p / \gamma)} \qquad (9)$$

Both specific discharges are equal:

$$\mathbf{Q_{sw} = Q_{ssp}} \qquad (10)$$

This condition yields:

$$\mathbf{(K / d) = (\rho / b)} \qquad (11)$$

For a given SSP wall relation (11) permits the calculation of the properties of a slurry wall with the same seepage properties. Assuming a slurry wall of a thickness d = 1m, the equivalent K-value is:

$$\mathbf{K_e = \rho \bullet (1m) / b} \qquad (12)$$

It must be kept in mind however that the nature of the two flows is quite different!

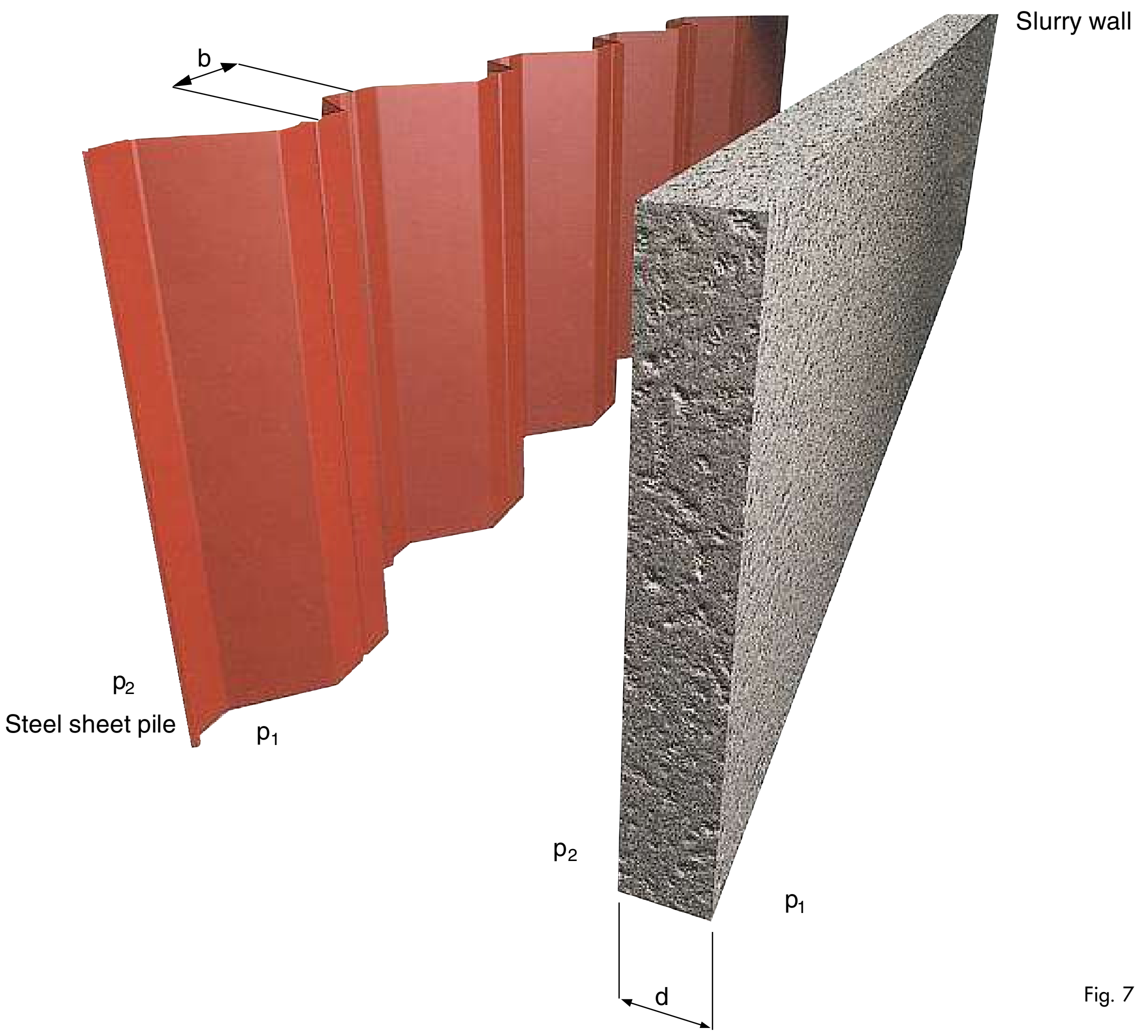


Fig. 7

GEI Consultants

Client	Blue Lake Reservoir Co	Project	1801834
		Page	
		By	G. Williams
		Date	4/22/2020
Subject		Checked	A. Stein
		Date	4/24/2020
		Approved	
		Date	

Estimate mat Property Strengths for Slope W

Existing silty clayey Sand

Avg N-value from B-2 = 29.1

Median = 29.5

from Foundation Eng (Peck et al) $\phi = 36°$

PI from lab results = 8, 11 and NP

Say PI = 10

from Lambe & Whitman $\phi = 35$ to $36°$

Say $\phi = 35°$

from Foundation Eng

	γ_d	γ_{sat}
mixed grain, loose	99	124
mixed grain, dense	116	135

Use $\gamma_m = 130$ pcf
$\phi' = 35°$

Embankment fill is reworked silty/clayey sand that will be placed and compacted in lifts ∴ use same material properties

BORING INFORMATION

STATION:		**OFFSET:**	
GROUND SURFACE EL. (ft):	NM	**DATE START/END:**	12/28/2017 - 12/28/2017
VERTICAL DATUM:		**DRILLING COMPANY:**	Elite Drilling Services, LLC
TOTAL DEPTH (ft):	22.9	**DRILLER NAME:**	Dan Westbrook
LOGGED BY:	Joel Jackson	**RIG TYPE:**	CME 850 Track

DRILLING INFORMATION

HAMMER TYPE:	Automatic	**CASING I.D./O.D.:**	NA/ NA	**CORE BARREL TYPE:**	
AUGER I.D./O.D.:	4.25 inch / 7.625 inch	**DRILL ROD O.D.:**	NM	**CORE BARREL I.D/O.D:**	NA / NA

DRILLING METHOD: Hollow Stem Auger

WATER LEVEL DEPTHS (ft): ▼ 16.0 12/28/2017

ABBREVIATIONS:

Pen. = Penetration Length
Rec. = Recovery Length
RQD = Rock Quality Designation = Length of Sound Cores>4 in / Pen.,%
WOR = Weight of Rods
WOH = Weight of Hammer
S = Split Spoon Sample
C = Core Sample
U = Undisturbed Sample
B = Bag Sample
DP = Direct Push Sample
HSA = Hollow-Stem Auger
Qp = Pocket Penetrometer Strength
Sv = Pocket Torvane Shear Strength
LL = Liquid Limit
PI = Plasticity Index
PID = Photoionization Detector
I.D./O.D.= Inside Diameter/Outside Diameter
NA, NM = Not Applicable, Not Measured
Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.
OVM = Organic Vapor Meter

Sample Information

Elev. (ft)	Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD
	S1*	0 to 2	24/18	16-13-13-16
	S2*	2 to 4	24/7	13-25-15-8
	S3*	4 to 5.3	16/12	5-6-50/4"
	S1	6 to 8	24/15	21-17-11-11
	S2	8 to 10	24/19	7-12-12-9
	S3	10 to 12	24/24	5-12-22-7
	S4	12 to 14	24/20	11-8-9-9
	S5	14 to 16	24/16	7-9-9-10
	S6	16 to 18	24/11	7-14-17-12
	S7	18 to 20	24/14	13-12-21-22
	S8	20 to 22	24/21	18-18-22-22
	S9	22.9 to	11/11	22-50/5"

Drilling Remarks/ Field Test Data

Rig chatter observed in the offset hole between 2.5 and 4 ft

Cuttings observed contained cobbles up to 6 inches in size. Shoe material was transitioning to Silty Sand (SM) and the tip was coated with granite powder.

*Auger refusal, offset 6' further upstream and begain sampling again at 6'

PP = 1.75

Soil and Rock Description

SILTY SAND (SM): moist, brown, 70% fine-med sand, 30% low plastic fines

CLAYEY SAND w/ GRAVEL (SC): moist, dark brown, 25% fine-coarse gravel, 45% fine sand, 30% low-med plastic fines

POORLY GRADED GRAVEL w/ SILT (GP-GM): moist, 80% coarse gravel (granite chunks), 10% sand, 10% fines
Coarse gravel in shoe (possibly a cobble that was drilled through)

CLAYEY SAND (SC): moist, dark brown, 16% fine gravel, 58% fine-med sand, 26% med plastic fines, LL=23, PI= 8, Moisture=8.3%
Shoe material transitioned into silty sands, tip was small granite rock piece
Auger refusal at 6.5'

COBBLE fragments

SILTY SAND w/ GRAVEL (SM): moist, brown, 25% fine gravel, 55% fine to med sand, 20% non-plastic fines.
Interbedded lenses of coarse gravel approximately 2" thick

SILT (ML): moist, brown, 10% fine sand, 90% low plastic fines

SILTY SAND (SM): brown, moist, 20% fine-coarse gravel up to 1" dia, 55% fine-coarse sand, 25% non-plastic fines
@ 11': SILTY SAND as above except 18% fine-coarse gravel, 60% fine-coarse sand, 22% non-plastic fines, Moisture= 7.3%

Saturated at 14'

CLAYEY SAND (SC): wet, brown, 20% fine-coarse gravel, 55% fine-med sand, 25% low-med plastic fines

18' - 20' CLAYEY SAND (SC): 10% fine gravel, 60% fine-med sand, 30% low-med plastic fines

20' - 20.5' CLAYEY SAND (SC): 5% fine gravel, 60% sand, 35% low-med plastic fines

SILTY SAND (SM): moist, dark brown, 5% fine subrounded gravel, 55% fine-coarse sand, 40% non to low plastic fines

21.6' - 21.75' Coarse gravel, subrounded

@ 22: SILTY SAND as above except: wet, 70% fine-med sand, 30% non to low plastic fines

NOTES:

PROJECT NAME: Upper Black Creek
CITY/STATE: Heeney, Colorado
GEI PROJECT NUMBER: 1705095

GEI Consultants

Foundation Engineering

SECOND EDITION

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Table 1.4 Porosity, Void Ratio, and Unit Weight of Typical Soils in Natural State

Description	Porosity (n)	Void Ratio (e)	Water Content (w)[a]	Unit Weight g/cu cm γ_d[b]	g/cu cm γ_{sat}[c]	lb/cu ft γ_d	lb/cu ft γ_{sat}
1. Uniform sand, loose	0.46	0.85	32	1.43	1.89	90	118
2. Uniform sand, dense	0.34	0.51	19	1.75	2.09	109	130
3. Mixed-grained sand, loose	0.40	0.67	25	1.59	1.99	99	124
4. Mixed-grained sand, dense	0.30	0.43	16	1.86	2.16	116	135
5. Windblown silt (loess)	0.50	0.99	21	1.36	1.86	85	116
6. Glacial till, very mixed-grained	0.20	0.25	9	2.12	2.32	132	145
7. Soft glacial clay	0.55	1.2	45	1.22	1.77	76	110
8. Stiff glacial clay	0.37	0.6	22	1.70	2.07	106	129
9. Soft slightly organic clay	0.66	1.9	70	0.93	1.58	58	98
10. Soft very organic clay	0.75	3.0	110	0.68	1.43	43	89
11. Soft montmorillonitic clay (calcium bentonite)	0.84	5.2	194	0.43	1.27	27	80

[a] w = water content when saturated, in per cent of dry weight.
[b] γ_d = dry unit weight.
[c] γ_{sat} = saturated unit weight.

where γ_w is the unit weight of water, taken as 1 g/cu cm in the metric system or 62.5 lb/cu ft in the English system. The value of γ_s or G may be determined by test in the laboratory, but it can usually be estimated with sufficient accuracy. For routine computations, the value of G for sands may be taken as 2.65. Tests on a large number of clay soils have indicated that the value of G usually falls in the range from 2.5 to 2.9 with an average value of about 2.7.

Table 1.3 gives the specific gravity of the most important soil constituents. It may be of assistance in estimating the value of G for a soil of known mineral composition.

Typical values of porosity, void ratio, and unit weight of various soils are listed in Table 1.4.

Density of Soil Aggregate. The behavior of any soil is influenced to a considerable extent by its relative looseness or denseness. In this respect, however, a distinction is necessary between coarse-grained cohesionless soils and cohesive materials. In a mass of coarse-grained soil most of the grains touch several others in point-to-point contact and efforts to densify the mass can reduce the void ratio only through rearrangement or crushing of the particles. On the other hand, the densification of fine-grained soil, especially clay, depends on other factors such as cohesion and the presence of water films on the particle surfaces.

The void ratio or porosity of any soil usually does not in itself furnish a direct indication of its behavior under load or during excavation. Of two coarse-grained soils at the same void ratio, one soil may be in a dense state whereas the other may be loose. Thus, the relative density of a coarse-grained material is much more significant than the void ratio alone. The relative density can be expressed numerically by the *density index*, I_d, defined as

$$\text{Density index, } I_d = \frac{e_{max} - e}{e_{max} - e_{min}} \qquad 1.10$$

in which e_{max} is the void ratio of the soil in its loosest state, e is the actual void ratio, and e_{min} is the void ratio in the densest possible state. Hence, I_d = 1.0 for a very dense soil and 0 for a very loose soil.

within which the sand cannot slip with respect to the base of the footing because of the roughness of the base, moves downward as a unit. As it moves it displaces the adjacent material. Consequently, the sand in two symmetrical zones $aO'bde$, one of which is illustrated on the left side of Fig. 19.4, is subjected to severe shearing distortions and slides outward and upward along the boundaries $O'bd$. The movement is resisted by the shearing strength of the sand along $O'bd$ and the weight of the sand in the sliding masses.

No completely adequate rigorous theory exists for calculating the ultimate capacity of a footing under such circumstances, but satisfactory approximate solutions have been obtained on the basis of various simplifying assumptions (Terzaghi, 1943; Meyerhof, 1955). It is assumed, as illustrated on the right half of Fig. 19.4, that the influence of the soil above the base level of the footing can be replaced by a uniform surcharge γD_f. Theory and experiment then indicate that the surface of sliding consists of a curved portion $O'c'$ and a straight section $c'b'$ that rises at an angle of $45° - \phi/2$ with the horizontal. The load q_d' on the footing, the surcharge γD_f, and the weight W of the sliding mass all produce normal stresses across the surface of sliding $O'c'b'$, which, in turn, develop frictional shearing resistance along the surface of sliding. When the mass is on the verge of sliding the resultant R of the normal and shearing stresses at any point such as f on the surface of sliding is inclined at the angle ϕ to the normal to the surface of sliding. The wedge $O'c'b'a'$ may be considered as a free body and its equilibrium investigated to evaluate q_d'. Various trials must be made to find the surface of sliding corresponding to the least value of q_d' that can be developed. This least value is designated the *gross ultimate bearing capacity*.

The results of such studies indicate that the gross ultimate bearing capacity may be expressed as

$$q_d' = \tfrac{1}{2}B\gamma N_\gamma + \gamma D_f N_q \qquad 19.1$$

and the *net ultimate bearing capacity* as

$$q_d = q_d' - \gamma D_f$$
$$= \tfrac{1}{2}B\gamma N_\gamma + \gamma D_f(N_q - 1) \qquad 19.2$$

In these equations, N_γ and N_q are dimensionless *bearing-capacity factors* depending primarily on ϕ. They may be evaluated by means of the chart, Fig. 19.5.

Equation 19.2 demonstrates that the bearing capacity of a footing on sand is derived from two sources: the frictional resistance due to the weight of the sand below the level of the footing and the frictional resistance due to the weight of the surrounding surcharge or backfill.

The unit weights of most sands, whether dry, moist, or saturated, lie within a fairly narrow range. Therefore, the unit weight of the sand is in itself not an important variable in the determination of the bearing capacity of a footing. However, if the sand is located below the free water surface, only its submerged weight is effective in pro-

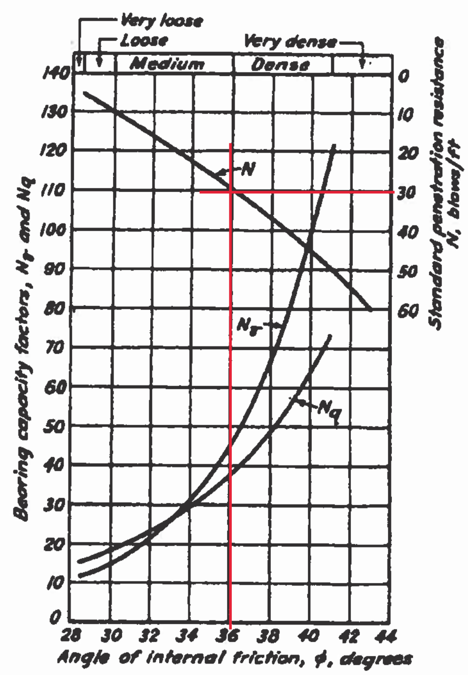

FIGURE 19.5. Curves showing the relationship between bearing-capacity factors and ϕ, as determined by theory, and rough empirical relationship between bearing capacity factors or ϕ and values of standard penetration resistance N.

Soil Mechanics

T. William Lambe • Robert V. Whitman

Massachusetts Institute of Technology

1969

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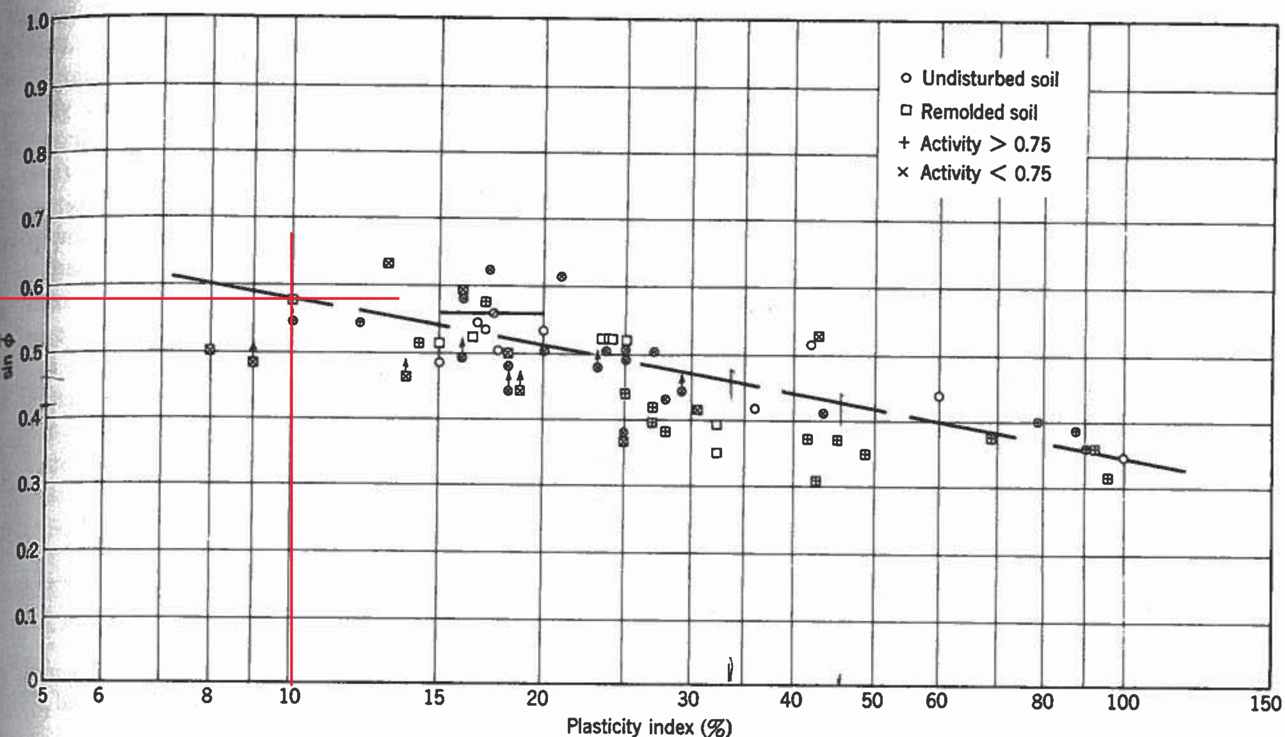


Fig. 21.4 Relationship between sin $\bar{\phi}$ and plasticity index for normally consolidated soils (From Kenney, 1959).

same line obtained from tests on normally consolidated specimens. For lower $\bar{p}_f$ the points fall above the relation from normally consolidated tests. Thus preconsolidation affects the effective stress-strength relation and tends to make the sample stronger at a given $\bar{p}_f$. This preconsolidation effect is difficult to see when results are plotted to the scale of Fig. 21.5*a*, hence the portion of this plot near the origin is magnified in the lower portion of the figure.

▸Example 21.1

A specimen of Weald clay is consolidated to 100 lb/in.², and is then failed by decreasing $\bar{\sigma}_3$ while $\bar{\sigma}_1$ is held constant. Find q_f, $\bar{p}_f$, and w_f.

Solution. On part (*c*) of Fig. 21.1 draw the effective stress

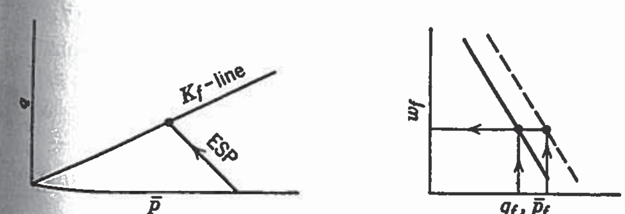


Fig. E21.1

path for this loading until it intersects the q_f–$\bar{p}_f$ relation (see Fig. E21.1). The point of intersection gives q_f and $\bar{p}_f$. Then go to Fig. 21.3, enter with *either* q_f or $\bar{p}_f$, and read w_f.

Answers. $q_f = 27$ lb/in.², $\bar{p}_f = 73$ lb/in.², $w_f = 19.2\%$. Note that w increased slightly during shear. ◀

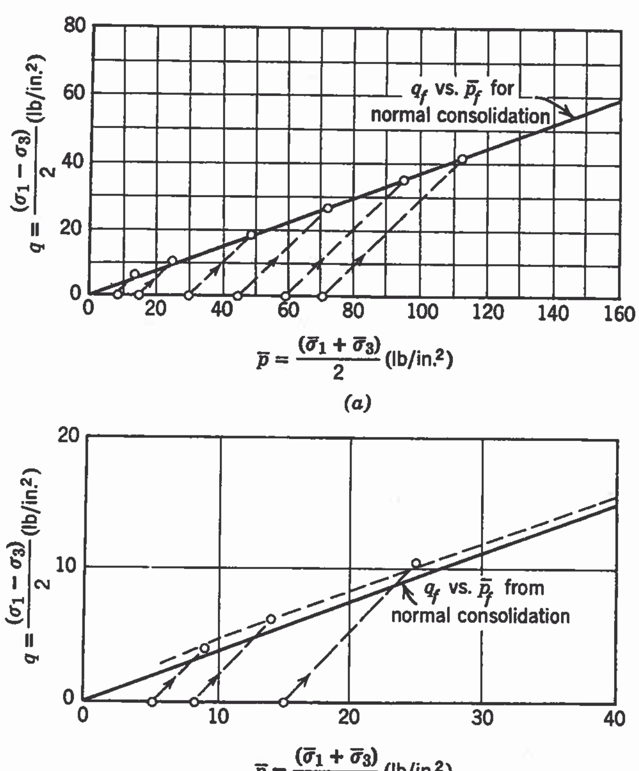


Fig. 21.5 Results of CD tests on overconsolidated Weald clay. $\bar{p}_m = 120$ lb/in.²

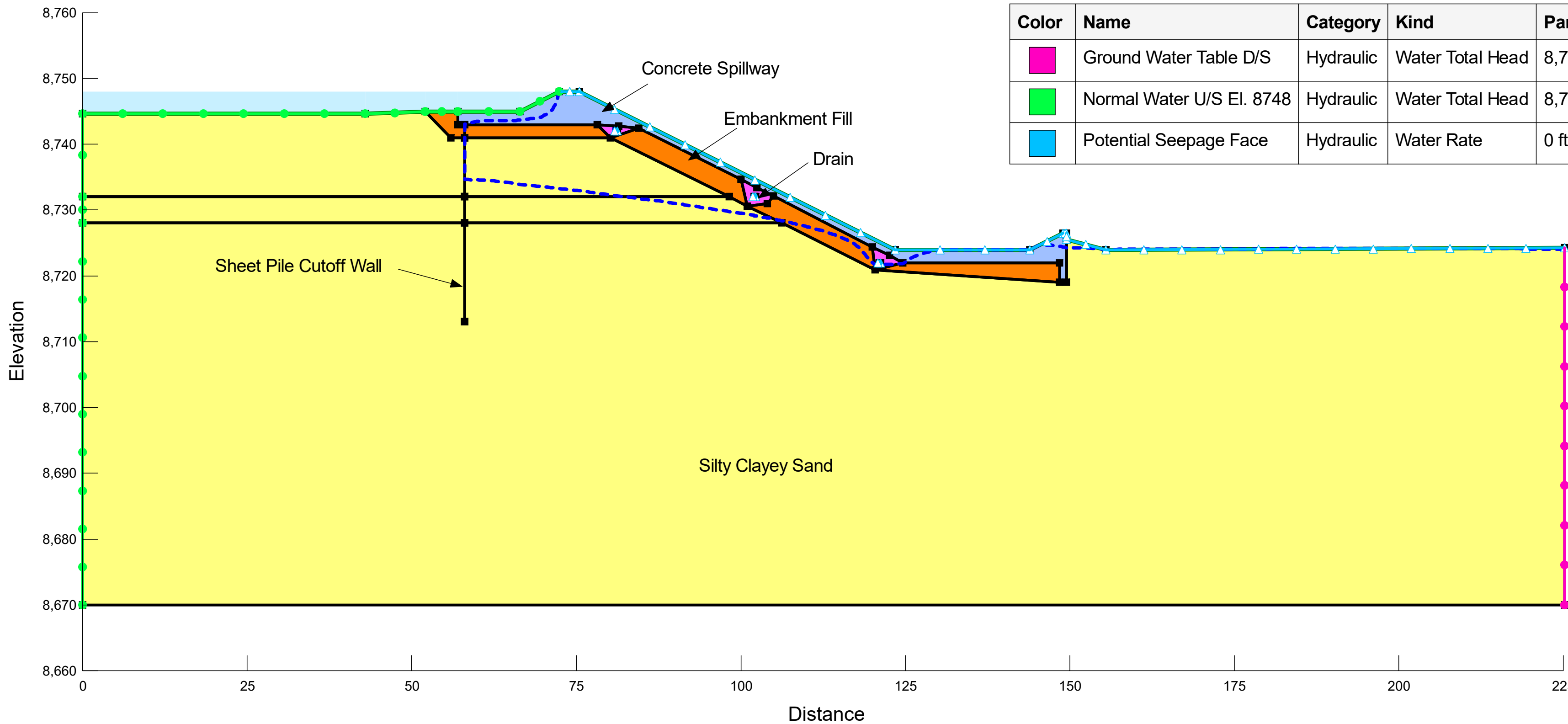
File Name: UBCR Spillway Section A_2019.gsz
Date:04/23/2020
Name: UBCR Steady State Seepage
Kind: SEEP/W
Method: Steady-State

Material Properties

Color	Name	Model	K-Function	Ky'/Kx' Ratio
	Concrete	Saturated / Unsaturated	Concrete Kx=3.28e-9 ft/sec	1
	Embankment Fill	Saturated / Unsaturated	Embankment Fill Kx=3.28e-7 ft/sec	0.25
	Filter Material	Saturated / Unsaturated	Filter, Kx=3.28e-4 ft/sec	0.25
	Sheetpile	Saturated / Unsaturated	Sheet pile, kx=1.56e-11 ft/sec	1
	Silty clayey sands	Saturated / Unsaturated	Silty Clayey Sand Kx=3.28e-07 ft/sec	0.1

Boundary Condition Properties

Color	Name	Category	Kind	Parameters
	Ground Water Table D/S	Hydraulic	Water Total Head	8,724 ft
	Normal Water U/S El. 8748	Hydraulic	Water Total Head	8,748 ft
	Potential Seepage Face	Hydraulic	Water Rate	0 ft³/sec



Upper Black Creek Reservoir Spillway Feasibility Study Summit County, CO	GEI Consultants	STEADY STATE SEEPAGE SEEP/W ANALYSIS	
Blue Lake Reservoir Company Vail, CO	Project 1801834	April 2020	Figure 1

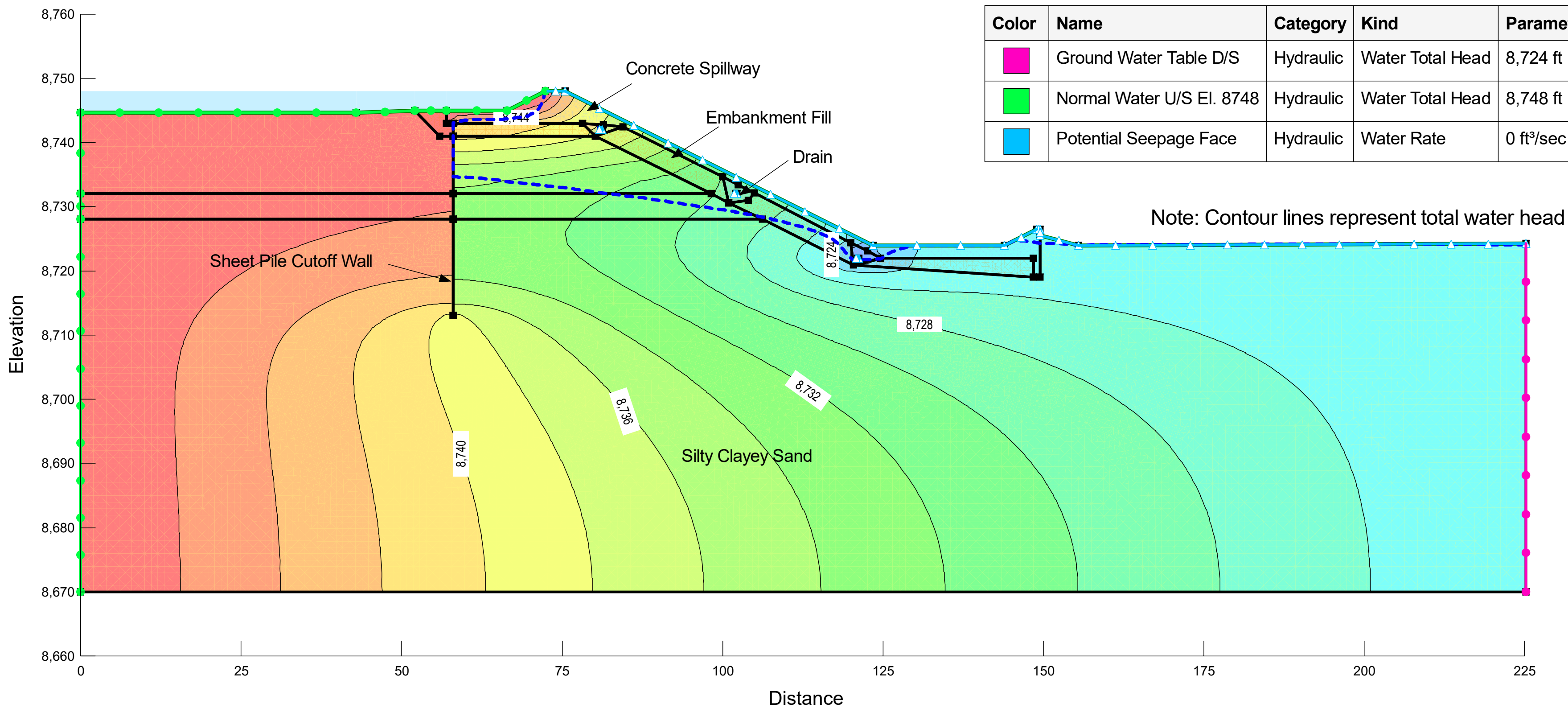
File Name: UBCR Spillway Section A_2019.gsz
Date:04/23/2020
Name: UBCR Steady State Seepage
Kind: SEEP/W
Method: Steady-State

Material Properties

Color	Name	Model	K-Function	Ky'/Kx' Ratio
	Concrete	Saturated / Unsaturated	Concrete Kx=3.28e-9 ft/sec	1
	Embankment Fill	Saturated / Unsaturated	Embankment Fill Kx=3.28e-7 ft/sec	0.25
	Filter Material	Saturated / Unsaturated	Filter, Kx=3.28e-4 ft/sec	0.25
	Sheetpile	Saturated / Unsaturated	Sheet pile, kx=1.56e-11 ft/sec	1
	Silty clayey sands	Saturated / Unsaturated	Silty Clayey Sand Kx=3.28e-07 ft/sec	0.1

Boundary Condition Properties

Color	Name	Category	Kind	Parameters
	Ground Water Table D/S	Hydraulic	Water Total Head	8,724 ft
	Normal Water U/S El. 8748	Hydraulic	Water Total Head	8,748 ft
	Potential Seepage Face	Hydraulic	Water Rate	0 ft³/sec

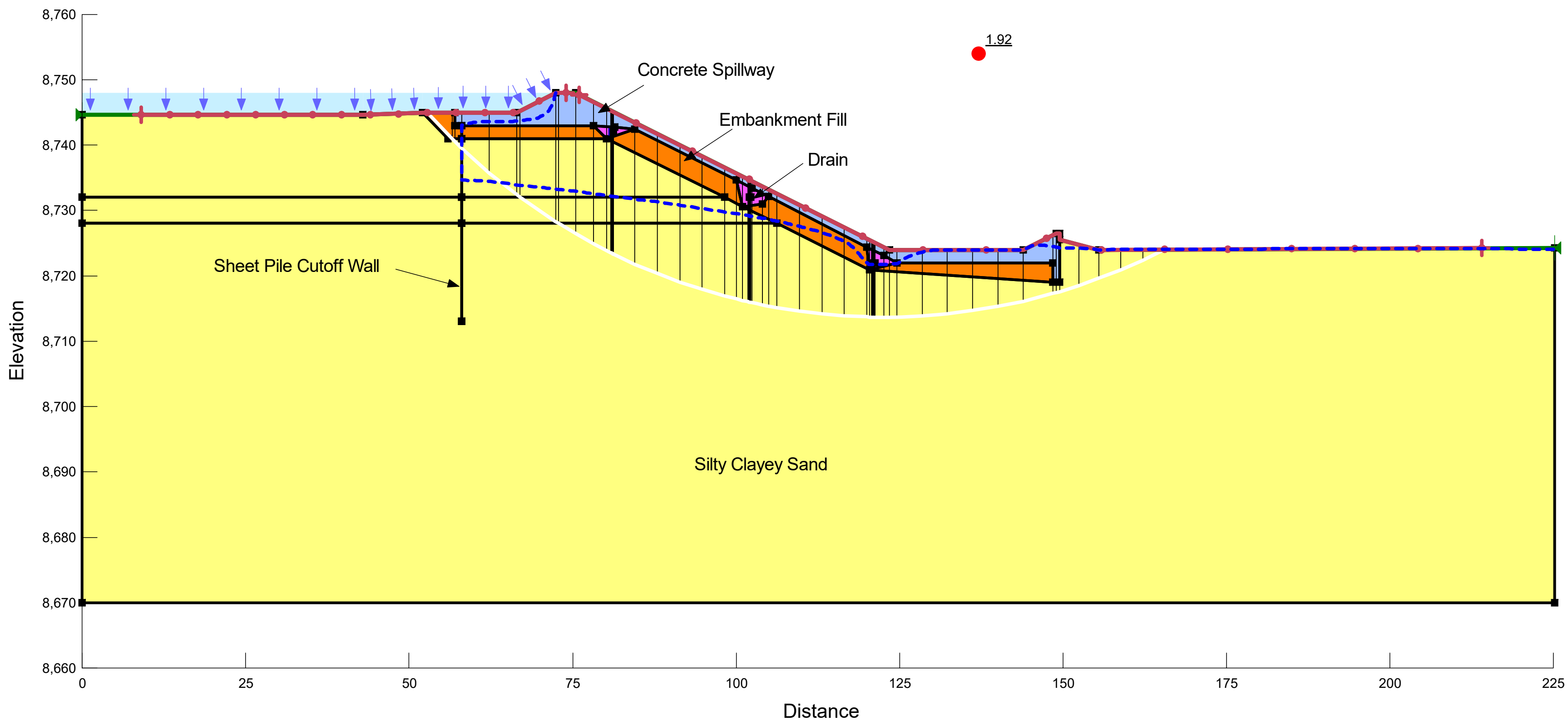


Upper Black Creek Reservoir Spillway Feasibility Study Summit County, CO	GEI Consultants	STEADY STATE SEEPAGE SEEP/W ANALYSIS
Blue Lake Reservoir Company Vail, CO	Project 1801834	April 2020 Figure 2

File Name: UBCR Spillway Section A_2019.gsz
Date:04/23/2020
Name: UBCR Full Reservoir
Kind: SLOPE/W
Method: Spencer

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Concrete	Mohr-Coulomb	150	10,000	0	0
	Embankment Fill	Mohr-Coulomb	130	0	35	0
	Filter Material	Mohr-Coulomb	125	0	32	0
	Silty clayey sands	Mohr-Coulomb	130	0	35	0



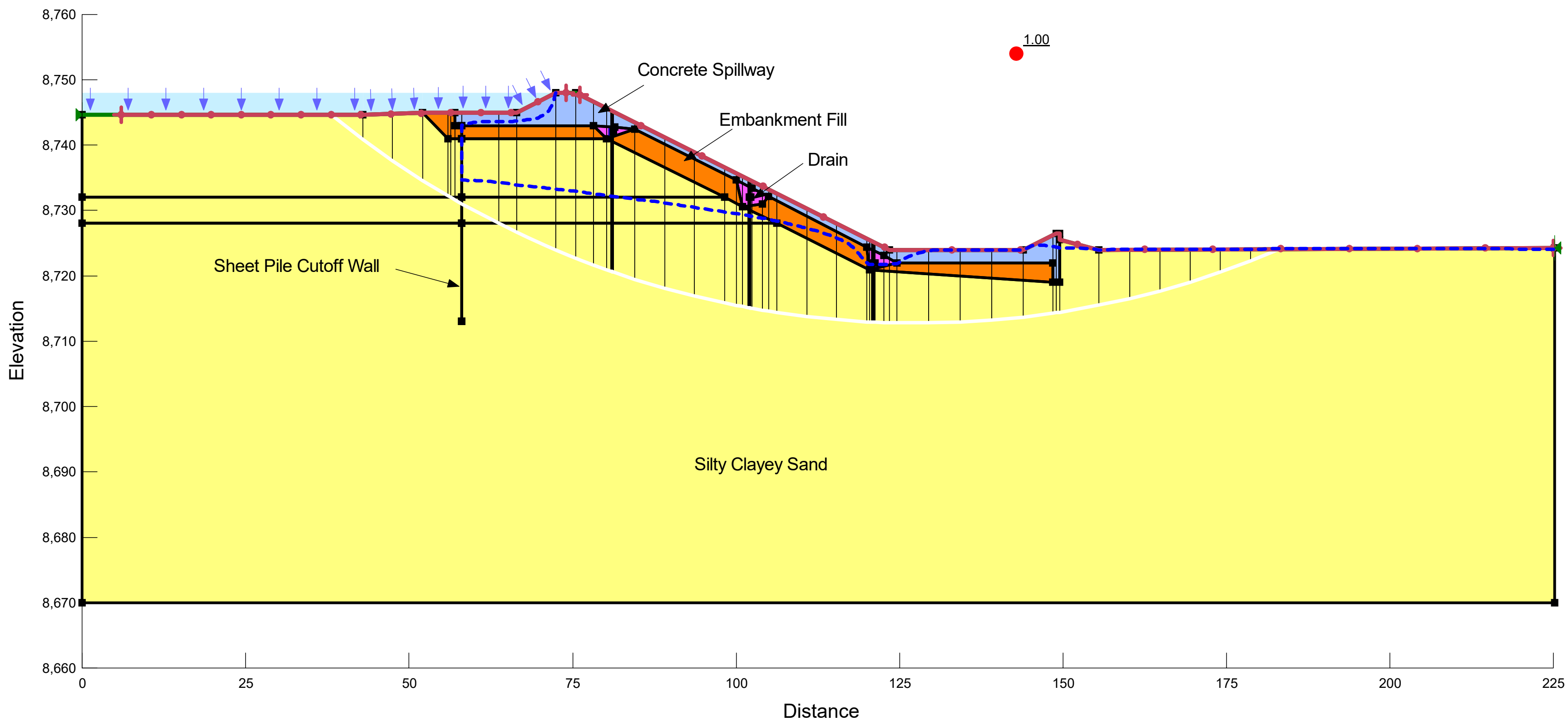
Upper Black Creek Reservoir Spillway Feasibility Study Summit County, CO	GEI Consultants	FULL RESERVOIR SLOPE/W ANALYSIS
Blue Lake Reservoir Company Vail, CO	Project 1801834	April 2020 Figure 3

File Name: UBCR Spillway Section A_2019.gsz
Date:04/23/2020
Name: UBCR Yield Acceleration
Kind: SLOPE/W
Method: Spencer

Horz Seismic Coef.: 0.235g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Concrete	Mohr-Coulomb	150	10,000	0	0
	Embankment Fill	Mohr-Coulomb	130	0	35	0
	Filter Material	Mohr-Coulomb	125	0	32	0
	Silty clayey sands	Mohr-Coulomb	130	0	35	0



Upper Black Creek Reservoir
Spillway Feasibility Study
Summit County, CO

Blue Lake Reservoir Company
Vail, CO

GEI Consultants

Project 1801834

YIELD ACCELERATION
SLOPE/W ANALYSIS

April 2020

Figure 4

GEI Consultants

Client	Blue Lake Reservoir Co	Project	1801834	Page	
		By	G. Williams	Date	4/22/2020
Subject		Checked	M. Paster	Date	4/23/20
		Approved	C. Masching	Date	4/27/2020

Estimate Deformations for spillway (Makdisi & Seed 78)

for 10000 yr return interval PGA = 0.5g (USGS)

Earthquake M = 6.9, say 7.0

$K_y = 0.235\,g$ (see SeepW results)

$\ddot{u}_{max} = 0.7$ (Harder (91) for $PGA_{Base} = 0.5g$)

for $y/H = 1$, $K_{max}/\ddot{u}_{max} = 0.35$ (Fig 7)

$K_{max} = 0.7 \times 0.35 = 0.245\,g$

$K_y/K_{max} = 0.235g/0.245g = 0.96$

For M 7.0 → < 1.0 cm (Fig 10)

* Calc assumes PGA @ rock = PGA @ base of dam

OK based on Seed, Chang, Dickenson, Bray 1997

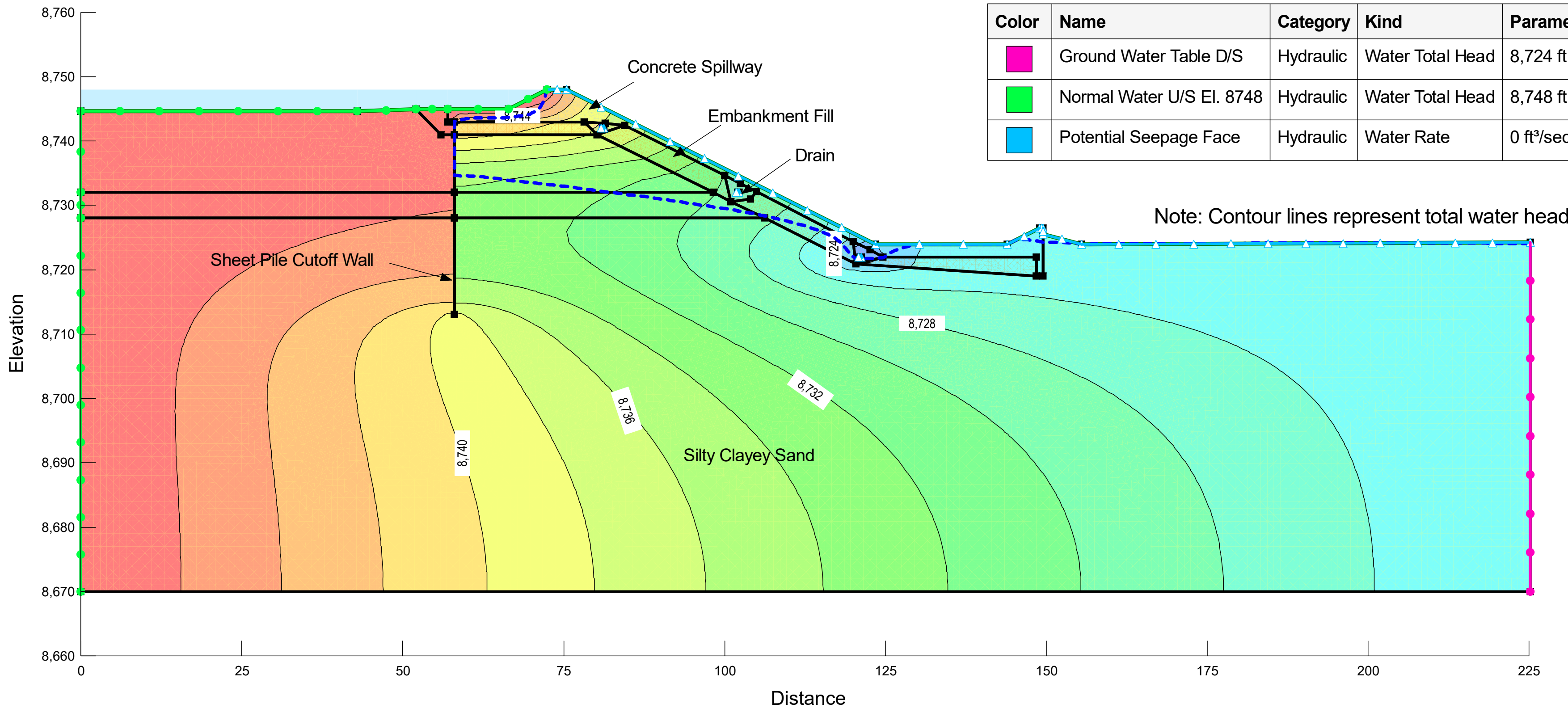
File Name: UBCR Spillway Section A_2019.gsz
Date:04/23/2020
Name: UBCR Steady State Seepage
Kind: SEEP/W
Method: Steady-State

Material Properties

Color	Name	Model	K-Function	Ky'/Kx' Ratio
	Concrete	Saturated / Unsaturated	Concrete Kx=3.28e-9 ft/sec	1
	Embankment Fill	Saturated / Unsaturated	Embankment Fill Kx=3.28e-7 ft/sec	0.25
	Filter Material	Saturated / Unsaturated	Filter, Kx=3.28e-4 ft/sec	0.25
	Sheetpile	Saturated / Unsaturated	Sheet pile, kx=1.56e-11 ft/sec	1
	Silty clayey sands	Saturated / Unsaturated	Silty Clayey Sand Kx=3.28e-07 ft/sec	0.1

Boundary Condition Properties

Color	Name	Category	Kind	Parameters
	Ground Water Table D/S	Hydraulic	Water Total Head	8,724 ft
	Normal Water U/S El. 8748	Hydraulic	Water Total Head	8,748 ft
	Potential Seepage Face	Hydraulic	Water Rate	0 ft³/sec



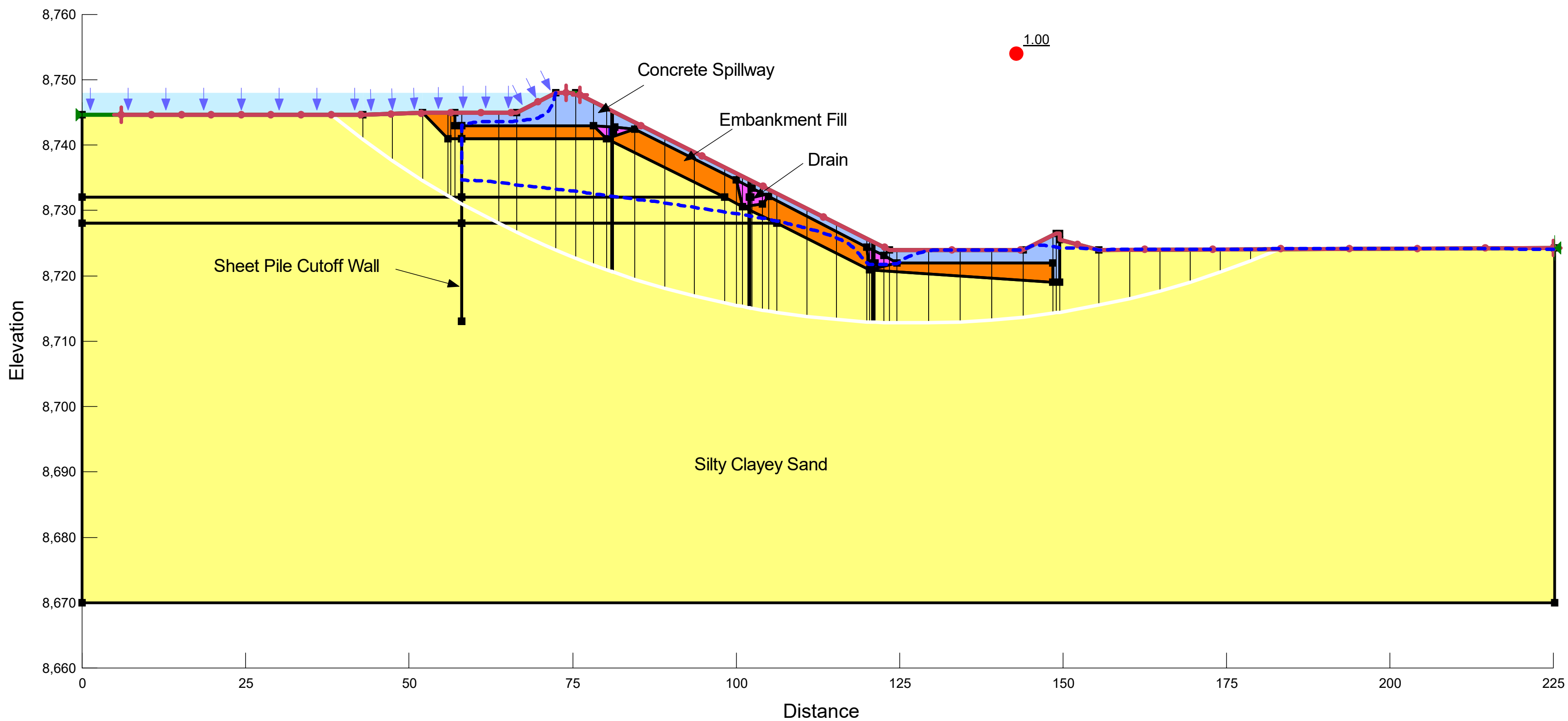
Upper Black Creek Reservoir Spillway Feasibility Study Summit County, CO	GEI Consultants	STEADY STATE SEEPAGE SEEP/W ANALYSIS
Blue Lake Reservoir Company Vail, CO	Project 1801834	April 2020 Figure 2

File Name: UBCR Spillway Section A_2019.gsz
Date:04/23/2020
Name: UBCR Yield Acceleration
Kind: SLOPE/W
Method: Spencer

Horz Seismic Coef.: 0.235g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Concrete	Mohr-Coulomb	150	10,000	0	0
	Embankment Fill	Mohr-Coulomb	130	0	35	0
	Filter Material	Mohr-Coulomb	125	0	32	0
	Silty clayey sands	Mohr-Coulomb	130	0	35	0



Upper Black Creek Reservoir Spillway Feasibility Study Summit County, CO	GEI Consultants	YIELD ACCELERATION SLOPE/W ANALYSIS	
Blue Lake Reservoir Company Vail, CO	Project 1801834	April 2020	Figure 4

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations

By Faiz I. Makdisi,[1] A. M. ASCE and H. Bolton Seed,[2] F. ASCE

Introduction

In the past decade major advances have been achieved in analyzing the stability of dams and embankments during earthquake loading. Newmark (13) and Seed (18) proposed methods of analysis for predicting the permanent displacements of dams subjected to earthquake shaking and suggested this as a criterion of performance as opposed to the concept of a factor of safety based on limit equilibrium principles. Seed and Martin (26) used the shear beam analysis to study the dynamic response of embankments to seismic loads and presented a rational method for the calculation of dynamic seismic coefficients for earth dams. Ambraseys and Sarma (1) adopted the same procedure to study the response of embankments to a variety of earthquake motions.

Later the finite element method was introduced to study the two-dimensional response of embankments (5,7) and the equivalent linear method (21) was used successfully to represent the strain-dependent nonlinear behavior of soils. In addition the nature of the behavior of soils during cyclic loading has been the subject of extensive research (10,20,23,29). Both the improvement in the analytical tools to study the response of embankments and the knowledge of material behavior during cyclic loading led to the development of a more rational approach to the study of stability of embankments during seismic loading. Such an approach was used successfully to analyze the Sheffield Dam failure during the 1925 Santa Barbara earthquake (24) and the behavior of the San Fernando Dams during the 1971 earthquake (25). This method has since been used extensively in the design and analysis of many large dams in the State of California and elsewhere.

Note.—Discussion open until December 1, 1978. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 104, No. GT7, July, 1978. Manuscript was submitted for review for possible publication on August 30, 1977.

[1] Project Engr., Woodward-Clyde Consultants, San Francisco, Calif.
[2] Prof. of Civ. Engrg., Univ. of California, Berkeley, Calif.

average acceleration for a potential sliding mass extending to a specified depth, y.

It would be desirable to establish a relationship showing the variation of the maximum acceleration ratio, $k_{max}/\ddot{u}_{max}$, with depth for a range of embankments and earthquake loading conditions. It would then be sufficient, for design purposes, to estimate the maximum crest acceleration in a given embankment due to a specified earthquake and use this relationship to determine the maximum average acceleration for any depth of the potential sliding mass. A simplified procedure to estimate the maximum crest acceleration and the natural period

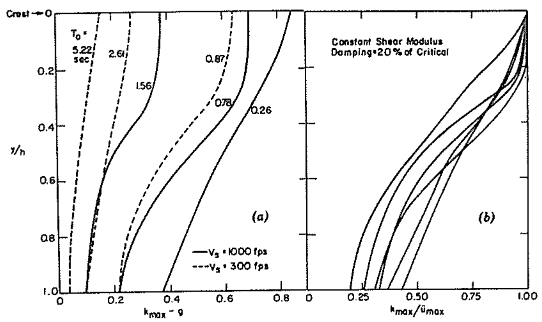


FIG. 5.—El Centro Record (12): (*a*) Variation of Maximum Average Acceleration with Depth of Sliding; (*b*) Variation of Ratio of Average Acceleration to Maximum Crest Acceleration with Depth of Sliding Surface

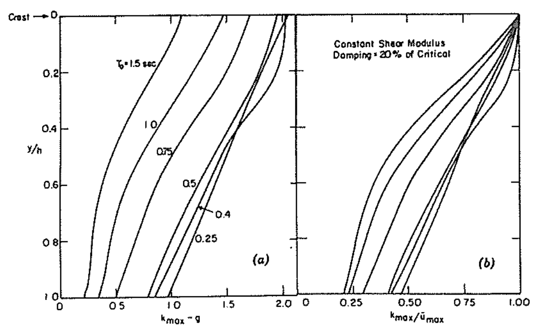


FIG. 6.—Average of Eight Strong Motion Records (1): (*a*) Variation of Maximum Average Acceleration with Depth of Sliding Mass; (*b*) Variation of Ratio of Maximum Average Acceleration to Maximum Crest Acceleration with Depth of Sliding Surface

of an embankment subjected to a given base motion is described in Appendix A of Ref. 11.

To determine the variation of maximum acceleration ratio with depth, use was made of published results of response computations using the one-dimensional shear slice method with visco-elastic material properties (1,26). Martin (12) calculated the response of embankments ranging in height between 100 ft–600 ft (30 m–180 m) and with shear wave velocities between 300 fps–1,000 fps (92 m/s–300 m/s). Using a constant shear modulus and a damping factor of 0.2,

the average acceleration histories for various levels were computed for embankments subjected to ground accelerations recorded in the El Centro earthquake of 1940. The variation of the maximum average acceleration, k_{max}, with depth for these embankments with natural periods ranging between 0.26 sec–5.22 sec is presented in Fig. 5(*a*). The maximum average acceleration in Fig. 5(*a*) is normalized with respect to the maximum crest acceleration and the ratio, $k_{max}/\ddot{u}_{max}$, plotted as a function of the depth of the sliding mass is presented in Fig. 5(*b*).

Ambraseys and Sarma (1) used essentially the same method reported by Seed and Martin (26) and calculated the response of embankments with natural periods ranging between 0.25 sec and 3.0 sec. They presented their results in terms of average response for eight strong motion records. The variation of maximum average acceleration with depth based on the results reported by Ambraseys and Sarma (1) is shown in Fig. 6(*a*) and that for the maximum acceleration ratio, $k_{max}/\ddot{u}_{max}$, is shown in Fig. 6(*b*). A summary of the results obtained

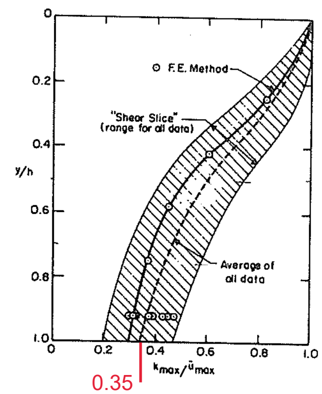


FIG. 7.—Variation of Maximum Acceleration Ratio with Depth of Sliding Mass

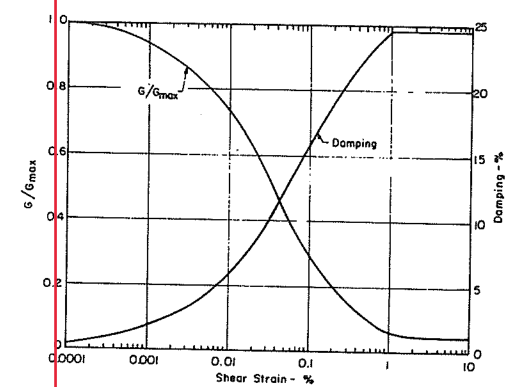


FIG. 8.—Shear Modulus and Damping Characteristics Used in Response Computations

from the different shear slice response calculations mentioned previously is presented in Fig. 7 together with results obtained from finite element calculations made in the present study. As can be seen from Fig. 7 the shape of the curves obtained using the shear slice method and the finite element method are very similar. The dashed curve in Fig. 7 is an average relationship of all data considered. The maximum difference between the envelope of all data and the average relationship ranges from ±10% to ±20% for the upper portion of the embankment and from ±20% to ±30% for the lower portion of the embankment.

Considering the approximate nature of the proposed method of analysis, the use of the average relationship shown in Fig. 7 for determining the maximum average acceleration for a potential sliding mass based on the maximum crest acceleration is considered accurate enough for practical purposes. For design computations where a conservative estimate of the accelerations is desired the upper bound curve shown in Fig. 7 may be used leading to values that are 10%–30% higher than those estimated using the average relationship.

permanent displacements are shown in Fig. 9(*b*). For a ratio of k_y/k_{max} of 0.2 the calculated displacements in this case ranged between 30 cm–200 cm (12 in.–80 in.), and for ratios greater than 0.5 the displacements were less than 25 cm (0.8 ft).

In the cases analyzed for the 8-1/4 magnitude earthquake, an artificial accelerogram proposed by Seed and Idriss (21) was used with maximum base accelerations of 0.4 *g* and 0.75 *g*. Two embankments were analyzed in this case and their calculated natural periods ranged between 0.8 sec and 1.5 sec. Table 4 shows the details of the calculations and in Fig. 10(*a*) the results of the permanent displacement computations are presented. As can be seen from Fig. 10(*a*) the permanent displacements computed for a ratio of k_y/k_{max} of 0.2 ranged between 200 cm–700 cm (80 in.–28 in.), and for ratios higher than 0.5 the values were less than 100 cm (40 in.). Note in this case that values of deformations calculated for a yield ratio less than 0.2 may not be realistic.

An envelope of the results obtained for each of the three earthquake loading

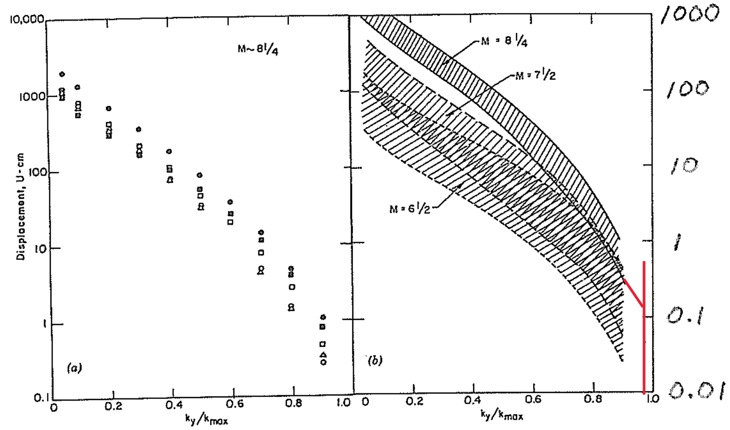


FIG. 10.—Variation of Permanent Displacement with Yield Acceleration: (*a*) Magnitude 8-1/4 Earthquake; (*b*) Summary of All Data

conditions is presented in Fig. 10(*b*) and reveals a large scatter in the computed results reaching, in the case of the magnitude 6-1/2 earthquake, about one order of magnitude.

It can reasonably be expected that for a potential sliding mass with a specified yield acceleration, the magnitude of the permanent deformation induced by a certain earthquake loading is controlled by the following factors: (1) The amplitude of induced average accelerations, which is a function of the base motion, the amplifying characteristics of the embankment, and the location of the sliding mass within the embankment; (2) the frequency content of the average acceleration time history, which is governed by the embankment height and stiffness characteristics, and is usually dominated by the first natural frequency of the embankment; and (3) the duration of significant shaking, which is a function of the magnitude of the specified earthquake.

Thus to reduce the large scatter exhibited in the data in Fig. 10(*b*), the permanent

displacements for each embankment were normalized with respect to its calculated first natural period, T_0, and with respect to the maximum value, k_{max}, of the average acceleration time history used in the computation. The resulting normalized permanent displacements for the three different earthquakes are presented in Fig. 11(*a*). It may be seen that a substantial reduction in the scatter of the data is achieved by this normalization procedure as evidenced by comparing the results in Figs. 10(*b*) and 11(*a*). This shows that for the ranges of embankment heights considered in this study [75 ft–150 ft (50 m–65 m)] the first natural period of the embankment and the maximum value of acceleration time history may be considered as two of the parameters having a major influence on the calculated permanent displacements. Average curves for the normalized permanent displacements based on the results in Fig. 11(*a*) are presented in Fig. 11(*b*). Although some scatter still exists in the results as shown in Fig. 11(*a*), the average curves presented in Fig. 11(*b*) are considered adequate to provide an order of magnitude of the induced permanent displacements for different

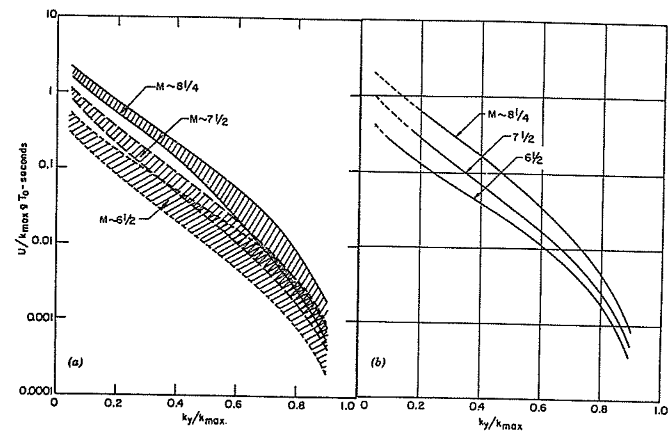


FIG. 11.—Variation of Yield Acceleration with: (*a*) Normalized Permanent Displacement—Summary of All Data; and (*b*) Average Normalized Displacement

magnitude earthquakes. At yield acceleration ratios less than 0.2 the average curves are shown as dashed lines since, as mentioned earlier, the calculated displacements at these low ratios may be unrealistic.

Thus, to calculate the permanent deformation in an embankment constructed of a soil that does not change in strength significantly during an earthquake, it is sufficient to determine its maximum crest acceleration, $\ddot{u}_{max}$, and first natural period, T_0, due to a specified earthquake. Then by the use of the relationship presented in Fig. 7, the maximum value of average acceleration history, k_{max}, for any level of the specified sliding mass may be determined. Entering the curves in Fig. 11(*b*) with the appropriate values of k_{max} and T_0, the permanent displacements can be determined for any value of yield acceleration associated with that particular sliding surface.

It has been assumed earlier in this paper that in the majority of embankments, permanent deformations usually occur due to slip of a sliding mass on a horizontal failure plane. For those few instances where sliding might occur on an inclined

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. LP05

Performance of Earth Dams During the Loma Prieta Earthquake

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SYNOPSIS: The October 17, 1989 Loma Prieta Earthquake shook a large number of earth and rockfill dams. There were actually more than 100 dams within 50 miles of the fault rupture associated with this event. Although more than half of these embankments were less than 60 feet in height, a number of major dams were strongly shaken. In general, the dams performed satisfactory with one major dam and one minor dam developing moderate damage. A small number also developed minor to moderate cracking which required repairs. The great majority, however, sustained no significant damage. Although this result is quite encouraging, this thought should be tempered by the fact that the reservoirs in many of these dams were quite low at the time of the earthquake. Thus, the 1989 Loma Prieta Earthquake was not the full test of these structures.

INTRODUCTION

The Loma Prieta Earthquake of October 17, 1989 resulted from a rupture of a segment of the San Andreas Fault near Santa Cruz, California. The rupture was initiated about 15 seconds past 5:04 p.m. local time and at a depth of approximately 11.5 miles. During the next 7 to 10 seconds, the rupture proceeded about 12 miles to the northwest and about 12 miles to the southeast. The fault also ruptured upward, but apparently stopped about 3 to 4 miles below the ground surface. Fault plane solutions for the numerous aftershocks indicated that the fault in the vicinity of this rupture dips to the southwest at an angle of approximately 70 degrees.

This rupture produced an earthquake with a Richter Magnitude M_L of 7.0 (assessed by the Seismographic Station at the University of California, Berkeley) and a surface wave magnitude M_S = 7.1 (assessed by the U. S. Geological Survey). It represented the largest earthquake in the San Francisco/Santa Cruz area since the great 1906 San Francisco Earthquake of magnitude 8+. The 1989 fault rupture occurred along a portion of the San Andreas Fault segment which ruptured during the 1906 earthquake.

The duration of strong shaking for this earthquake was generally between 7 and 10 seconds, considerably less than that usually associated with a magnitude 7 event. It has been speculated that this short duration was a result of the central location of the earthquake's focal point and its bi-directional rupture pattern (see Seed et al., 1990). The highest horizonal acceleration recorded was 0.64g and was measured at the Corralitos station located adjacent to the surface expression of the fault and only a few miles from the epicenter. Seismographs in the epicentral area also recorded relatively high vertical accelerations that were comparable to those recorded in the horizontal direction. At the Capitola recording station, the peak vertical acceleration was 0.60g whereas the peak horizontal acceleration was 0.54g.

CHARACTERISTICS OF AFFECTED EARTH DAMS

Presented in Figure 1 is a plot showing the epicenter and fault rupture associated with the 1989 Loma Prieta Earthquake. Also shown in this figure are the locations of 111 earth dams found within 50 miles of the fault rupture. The majority of these dams are essentially homogeneous earth dams. The heights and completion dates for these dams are presented in Tables 1 and 2.

Table 1: Maximum Heights of Affected Earth Dams

Maximum Height (feet)	Number of Dams
< 10	1
11 - 20	7
21 - 40	31
41 - 60	24
61 - 80	16
81 - 100	8
101 - 150	14
151 - 200	5
201 - 250	4
251 - 300	0
301 - 350	1

TOTAL 111 dams

The final step in the process was to calculate a factor of safety against triggering liquefaction, F_L, using the results from Equations 1 and 2 as shown below in Equation 3:

$$F_L = \frac{(\tau/\sigma_o')_L}{(\tau/\sigma_o')_{avg}} \qquad (3)$$

The application of these equations are illustrated in Table 5 for the two suspect sites. As shown in this table, the calculated factors of safety against triggering liquefaction are between 1.2 and 1.5. These predicted factors of safety correspond very well with the observation of no damage at this dam.

Table 5: Predicted Factors of Safety Against Triggering Liquefaction at O'Neill Dam

STATION 100 SITE:
CRITICAL DEPTH = 9 FEET
DEPTH TO WATER = 3 FEET

AVERAGE CORRECTED SPT $(N_1)_{60}$	FINES CONTENT (%)	σ_o (psf)	σ_o' (psf)	$(\tau/\sigma_o')_{M=7.5}$	$(\tau/\sigma_o')_L$	$(\tau/\sigma_o')_{avg}$	F_L
8	15	1125	751	0.135	0.161	0.105	1.53

STATION 133 SITE:
CRITICAL DEPTH = 7.5 FEET
DEPTH TO WATER = 2 FEET

AVERAGE CORRECTED SPT $(N_1)_{60}$	FINES CONTENT (%)	σ_o (psf)	σ_o' (psf)	$(\tau/\sigma_o')_{M=7.5}$	$(\tau/\sigma_o')_L$	$(\tau/\sigma_o')_{avg}$	F_L
10	5	938	594	0.110	0.131	0.111	1.18

STRONG MOTIONS RECORDED ON EMBANKMENT DAMS

The 1989 Loma Prieta Earthquake provided an excellent opportunity to calibrate dynamic response techniques. As illustrated in Table 6, strong motion records were recorded at eight embankment dams.

Presented in Figure 19 is a plot comparing the peak transverse accelerations measured at both the base and crest of several earth and rockfill dams. These measurements include those made during the Loma Prieta earthquake as well as those made during previous events. As may be observed, the points indicate that at low accelerations, the amplification through embankment dams is relatively large. However, as the peak base accelerations become larger, the amount of amplification is relatively low, possibly a result of increased damping or yielding of embankment materials. Also shown in Figure 19 is a tentative upper bound curve. This curve should not necessarily be used for design purposes, but it may be useful as a verification tool in the performance of dynamic response analyses.

Table 6: Peak Accelerations Measured on Earth Dams During the Loma Prieta Earthquak

DAM	MAX. HEIGHT (feet)	PEAK ACCELERATIONS (g) BASE T	BASE L	BASE V	ABUTMENT T	ABUTMENT L	ABUTMENT V	CREST T	CREST L	CREST V
LEXINGTON	205	-	-	-	.45	.41	.15	.39	.40	.22
		-	-	-	-	-	-	.45	.34	.20
SAN JUSTO*	135	.26	.16	-	-	-	-	.50	.39	.32
LEROY ANDERSON*	235	.26	.25	.17	.07	.08	.05	.39	.26	.19
		.23	.18	.16	-	-	-	.43	.32	.16
		-	-	-	-	-	-	.38	.32	.23
SAN LUIS	313	.04	.06	.02	-	-	-	.26	.18	.04
		.09	.09	.03	-	-	-	.14	.17	.05
O'NEILL	70	.08	.11	.06	-	-	-	.12	.14	.06
		-	-	-	-	-	-	.15	.10	.06
MARTINEZ	54	.09	.07	.02	-	-	-	.13	.15	.03
DEL VALLE	222	.04	.06	.03	-	-	-	.08	.08	.07
CONTRA LOMA	88	-	-	-	-	-	-	.07	.05	.03

NOTES: * DENOTES THAT OTHER RECORDS ARE AVAILABLE AT THIS DAM
T DENOTES TRANSVERSE DIRECTION
L DENOTES LONGITUDINAL DIRECTION
V DENOTES VERTICAL DIRECTION

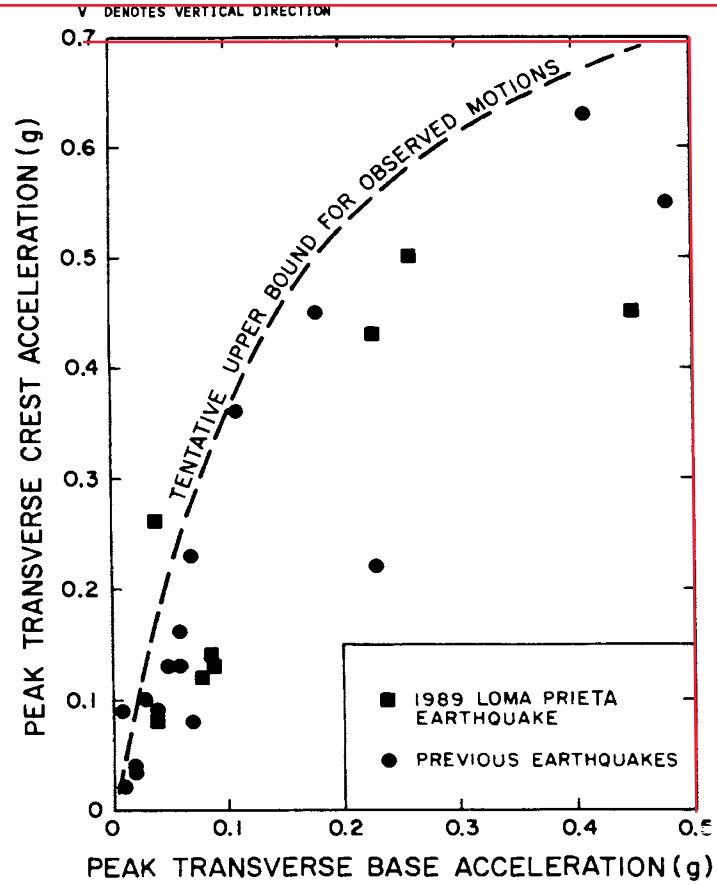

Figure 19: Comparison of Peak Base and Crest Transverse Accelerations Measured at Earth Dams (from Harder et al, 1990)

Site-Dependent Seismic Response Including Recent Strong Motion Data

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Seed, R. B., Chang, S. W., Dickenson, S. E., and Bray, J. D. (1997) "Site-Dependent Seismic Response Including Recent Strong Motion Data." Proc., Special Session on Earthquake Geotechnical Engineering, XIV International Conf. On Soil Mechanics and Foundtion Engineering, Hamburg, Germany, A. A. Balkema Publ., Sept. 6-12, pp. 125-134.

ABSTRACT: This paper presents a brief summary of recently completed studies of site-dependent seismic site response incorporating the wealth of strong motion data provided by recent earthquakes. The empirical data, results of back analyses of various strong motions recording sites, and analyses of the response of sites to various design levels of shaking are combined to develop recommendations for site classification, prediction of site-dependent amplification, and site-dependent design spectra. The adequacy of current U.S. building codes and provisions in addressing site-dependent site response is assessed in light of the strong motion data from these recent earthquakes.

1 INTRODUCTION

In recent years, the importance of site effects on seismic site response has been repeatedly demonstrated during earthquakes such as Mexico City (1985), Armenia (1988), Loma Prieta (1989), the Philippines (1990), Northridge (1994), and Hyogo-ken Nanbu (1995). This paper presents a brief overview of recently completed studies on the seismic response of (a) soft cohesive sites (b) deep, stiff cohesive sites, and (c) deep, stiff cohesionless soil sites which incorporate the wealth of empirical data and analytical results, principally from the Loma Prieta and Northridge Earthquakes (Chang, 1996 and Dickenson, 1994).

The results of these studies were used to develop recommendations for site classifications and site-dependent design spectra for code-based design. The resulting recommendations are then compared with the design levels recommended by the 1994 Uniform Building Code (UBC) and the 1994 National Earthquake Hazards Reduction Program (NEHRP) Provisions.

Maps of the areas affected by the Loma Prieta and Northridge earthquakes are presented in Figure 1 and Figure 2, respectively. The locations of strong motion stations are shown, along with a simplified overview of the regional geology. In Figure 1, soft and deep cohesive soil sites are primarily located along the San Francisco Bay margins; deep stiff soil sites of interest are generally located in the East Bay (Oakland) area. The soil sites in Figure 2 are predominantly deep stiff soil sites.

2 SOFT AND DEEP COHESIVE SOILS

Strong motion records were obtained at ten soft and/or deep cohesive soil sites throughout the San Francisco Bay region during the Loma Prieta earthquake for moderate levels of shaking (A_{max} = 0.14g to 0.33g). Dickenson (1994) back-analyzed these sites and developed one-dimensional site response models using both equivalent linear (SHAKE90) and fully nonlinear (MARDESRA) analysis methods. SHAKE90 is a slightly modified version of the original SHAKE (Schnabel et al., 1972), and MARDESRA is similar to DESRA-2 (Lee and Finn, 1978) except that the dynamic properties of the soil are represented by the Martin-Davidenkov (Martin, 1975) model. The predictive capabilities of these methods can be excellent, as illustrated in the following analysis of Treasure Island, one of the ten soft and/or deep cohesive soil sites of interest.

The generalized soil profile for Treasure Island (TI), shown in Figure 3, indicates that the site consists of loose sandy fill and loose silty sand

Table1: Proposed Site Classification System

Site Class	Site Condition	General Description	Site Characteristics
(A_0)	A_0	Very hard rock	V_s (avg.) > 5,000 ft/s in top 50 ft.
A	A_1	Competent rock with little or no soil and/or weathered rock veneer.	2,500 ft/s ≤ V_s (rock) ≤ 5,000 ft/s, and $H_{soil+weathered\ rock}$ < 40 ft with V_s > 800 ft/s (in all but the top few feet[3]).
AB	AB_1	Soft, fractured and/or weathered rock.	For both AB_1 and AB_2: 40 ft ≤ $H_{soil+weathered\ rock}$ ≤ 150 ft, and V_s ≥ 800 ft/s (in all but the top few feet[3]).
	AB_2	Stiff, very shallow soil over rock and/or weathered rock.	
B	B_1	Deep, primarily cohesionless [4] soils. (H_{soil} ≤ 300 ft.)	No "soft clay" (see Note 5), and $H_{cohesive\ soil}$ < 0.2 $H_{cohesionless\ soil}$
	B_2	Medium depth, stiff cohesive soils and/or mix of cohesionless with stiff cohesive soils; no "soft clay".	$H_{all\ soils}$ ≤ 200 ft, and V_s (cohesive soils) > 500 ft/s (see Note 5).
C	C_1	Medium depth, stiff cohesive soils and/or mix of cohesionless with stiff cohesive soils; thin layer(s) of soft clay.	Same as B_2 above, except 0 ft < $H_{soft\ clay}$ ≤ 10 ft (see Note 5).
	C_2	Very deep, primarily cohesionless soils.	Same as B_1 above, except H_{soil} > 300 ft.
	C_3	Deep, stiff cohesive soils and/or mix of cohesionless with stiff cohesive soils; no "soft clay".	H_{soil} > 200 ft, and V_s (cohesive soils) > 500 ft/s
	C_4	Soft, cohesive soil at small to moderate levels of shaking.	10 ft ≤ $H_{soft\ clay}$ ≤ 100 ft, and $A_{max,rock}$ ≤ 0.25 g
D	D_1	Soft, cohesive soil at medium to strong levels of shaking.	10 ft ≤ $H_{soft\ clay}$ ≤ 100 ft, and 0.25 g < $A_{max,rock}$ ≤ 0.45 g, or (0.25 g < $A_{max,rock}$ ≤ 0.55 g and M ≤ 7-1/4)
(E)[6]	E_1	Very deep, soft cohesive soil.	$H_{soft\ clay}$ > 100 ft (see Note 5).
	E_2	Soft, cohesive soil and very strong shaking.	$H_{soft\ clay}$ > 10 ft and either: $A_{max,rock}$ > 0.55 g, or $A_{max,rock}$ > 0.45 g and M > 7-1/4
	E_3	Very high plasticity clays.	H_{clay} > 30 ft with PI > 75% and V_s < 800 ft/s
(F)[7]	F_1	Highly organic and/or peaty soils.	H > 20 ft of peat and/or highly, organic soils
	F_2	Sites likely to suffer ground failure due either to significant soil liquefaction or other potential modes of ground instability.	Liquefaction and/or other types of ground failure analysis required.

1. H= total (vertical) depth of soils of the type or types referred to.
2. V_s=seismic shear wave velocity (ft/sec) at small shear strains (shear strain ~ 10^{-4}%)
3. If surface soils are cohesionless, V_s may be less than 800 ft/sec in top 10 feet.
4. "Cohesionless soils" = soils with less than 30% "fines" by dry weight. "Cohesive soils" = soils with more than 30% "fines" by dry weight, and 15% ≤ PI (fines) ≤ 90%. Soils with more than 30% fines, and PI (fines) < 15% are considered "silty" soils herein, and these should be (conservatively) treated as "cohesive" soils for site classification purposes in this Table.
5. "Soft Clay" is defined as cohesive soil with: (a) Fines content ≥30%, (b) PI(fines) ≥20%, and (c) V_s ≤500 ft/s.
6. Site-specific geotechnical investigations and dynamic site response analyses are strongly recommended for these conditions. Response characteristics within this Class (E) of sites tends to be more highly variable than for Classes A_o through D, and the response projections herein should be applied conservatively in the absence of (strongly recommended) site-specific studies.
7. Site-specific geotechnical investigations and dynamic site response analyses are *required* for these conditions. Potentially significant ground failure must be mitigated, and/or it must be demonstrated that the proposed structure/facility can be engineered to satisfactorily withstand such ground failure.

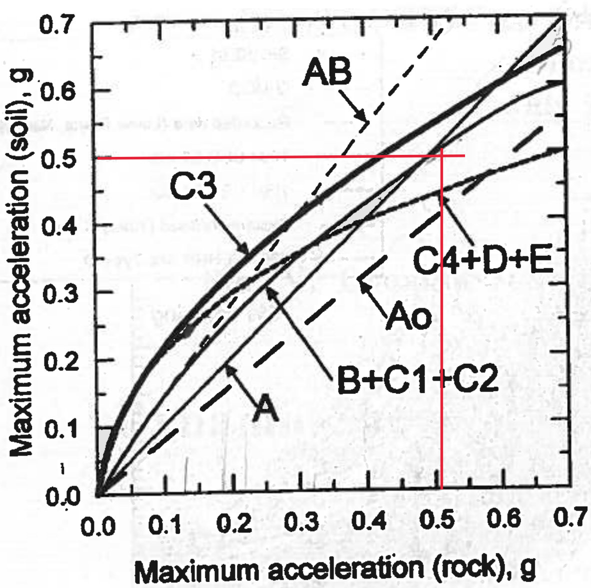


Figure 9: Proposed Site-Dependent Relationship Between Mean A_{max} and A_{max} for "Competent" Rock Sites.

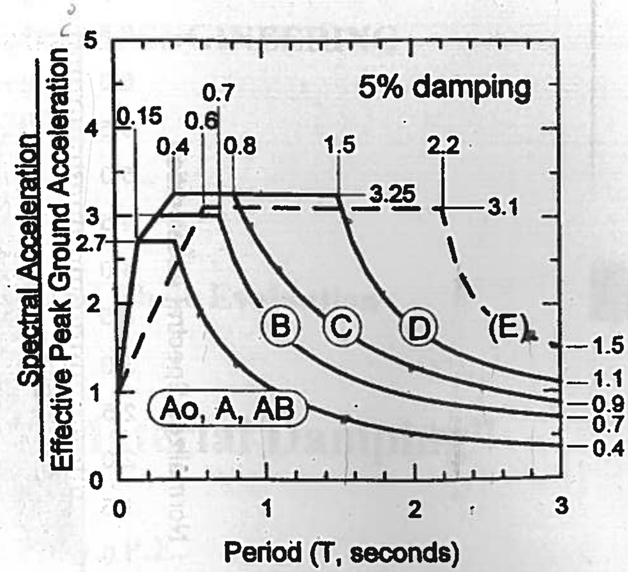


Figure 10: Proposed Site-Dependent Response Spectra (with 5% damping).

$\frac{S_a}{PGA_{rock}}$ $\left(\frac{PGA_{site}}{1.15}\right)$

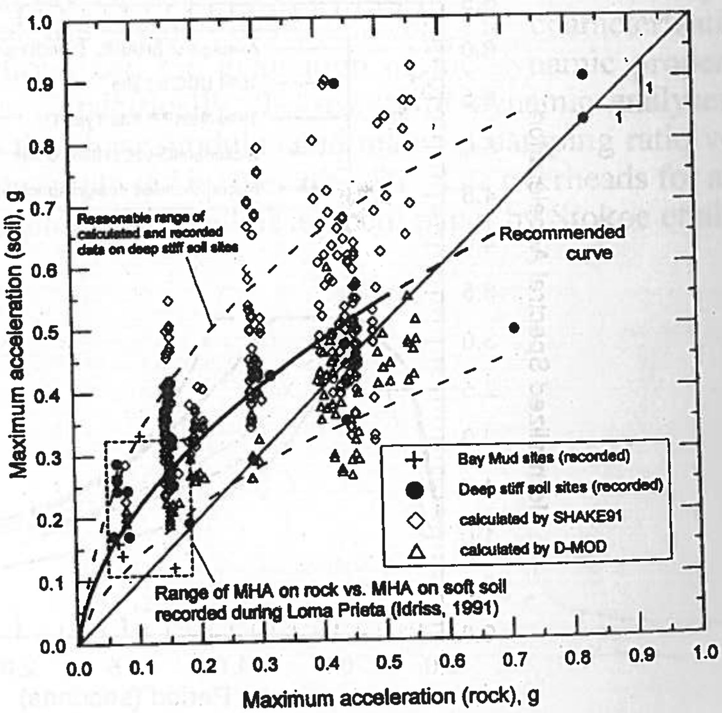


Figure 11: Relationship for A_{max} vs. $A_{max, rock}$ for deep stiff soil sites based on available empirical data from the Loma Prieta and Northridge Earthquakes and calculations using both equivalent linear and fully nonlinear site response methods (modified from Chang, 1996).

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (upd

Latitude

Decimal degrees

39.804

Longitude

Decimal degrees, negative values for western longitudes

-106.264

Site Class

760 m/s (B/C boundary)

Spectral Period

Peak Ground Acceleration

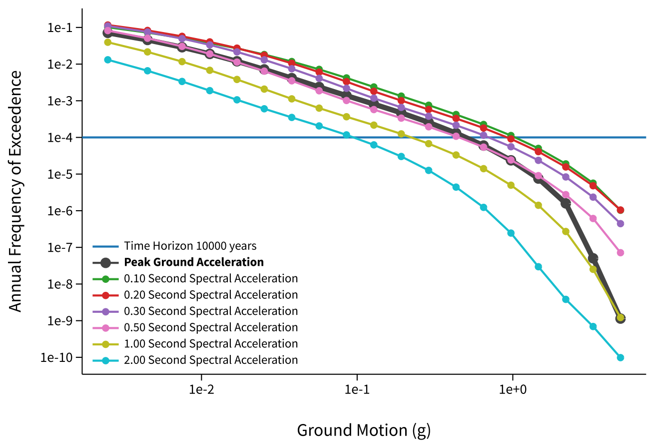
Time Horizon

Return period in years

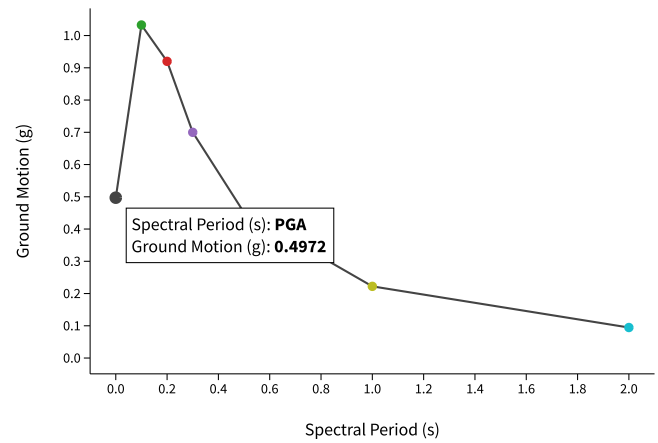
10000

Hazard Curve

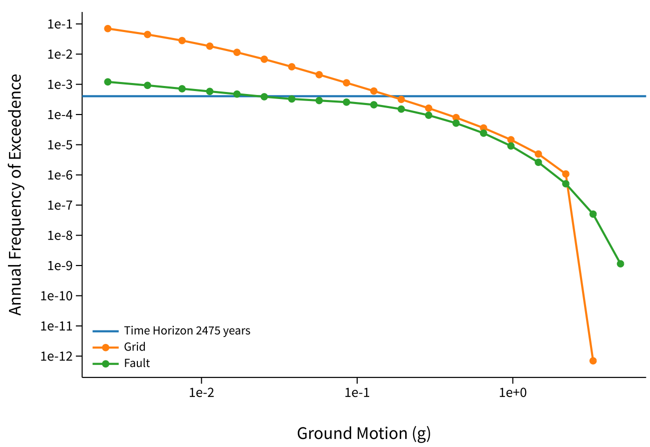
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

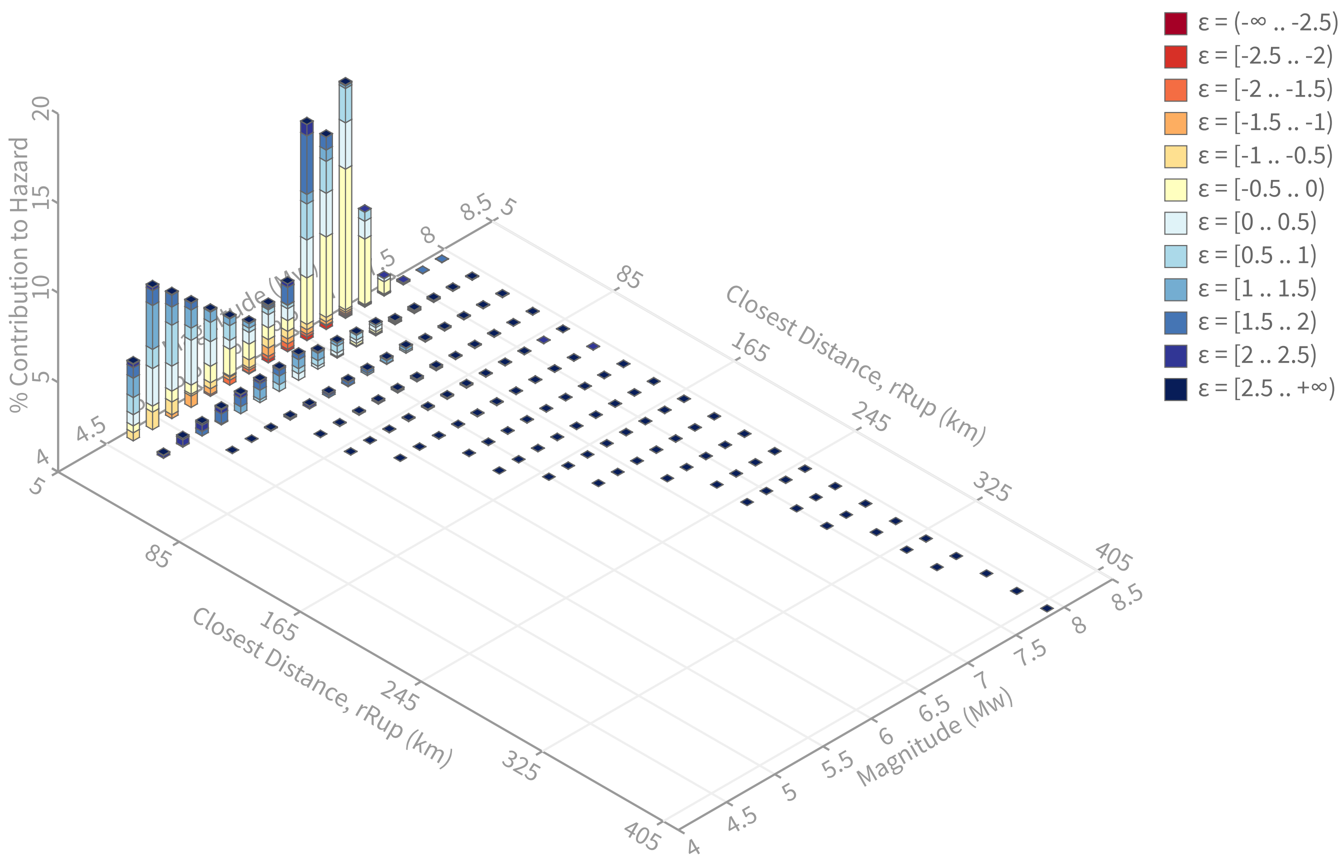


[View Raw Data](#)

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 10000 yrs
Exceedance rate: 0.0001 yr^{-1}
PGA ground motion: 0.49721063 g

Recovered targets

Return period: 9938.763 yrs
Exceedance rate: 0.00010061614 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.52 %

Mean (over all sources)

m: 6.08
r: 11.9 km
ε_0: 0.43 σ

Mode (largest m-r bin)

m: 6.9
r: 2.82 km
ε_0: -0.04 σ
Contribution: 13.04 %

Mode (largest m-r-ε_0 bin)

m: 6.9
r: 2.28 km
ε_0: -0.19 σ
Contribution: 7.85 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
SSCn Fixed Smoothing Zone 9 (opt)	Grid							20.46
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	6.59
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	4.05
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	2.40
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	2.30
PointSourceFinite: -106.264, 39.961		16.97	5.68	0.70	106.264°W	39.961°N	0.00	1.94
USGS Fixed Smoothing Zone 3 (opt)	Grid							20.46
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	6.59
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	4.05
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	2.40
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	2.30
PointSourceFinite: -106.264, 39.961		16.97	5.68	0.70	106.264°W	39.961°N	0.00	1.94
Geologic Model Full Rupture	Fault							14.09
Gore Range frontal 50		2.06	6.95	-0.04	106.295°W	39.797°N	253.81	6.21
Williams Fork Mountains 50		10.86	6.53	1.71	106.154°W	39.854°N	59.16	2.27
Gore Range frontal 35		1.56	6.95	-0.15	106.295°W	39.797°N	253.81	2.23
Gore Range frontal 65		2.43	6.95	0.09	106.295°W	39.797°N	253.81	1.88
Geologic Model Partial Rupture	Fault							11.79
Gore Range frontal 50		2.54	6.71	0.16	106.295°W	39.797°N	253.81	6.22
Gore Range frontal 65		2.92	6.71	0.21	106.295°W	39.797°N	253.81	2.01
Gore Range frontal 35		2.34	6.71	0.18	106.295°W	39.797°N	253.81	1.96
SSCn Adaptive Smoothing Zone 9 (opt)	Grid							9.78
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	2.97
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	1.87
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	1.11
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	1.02
USGS Adaptive Smoothing Zone 3 (opt)	Grid							9.78
PointSourceFinite: -106.264, 39.916		12.82	5.48	0.47	106.264°W	39.916°N	0.00	2.97
PointSourceFinite: -106.264, 39.826		5.56	5.22	-0.32	106.264°W	39.826°N	0.00	1.87
PointSourceFinite: -106.264, 40.006		21.09	5.89	0.83	106.264°W	40.006°N	0.00	1.11
PointSourceFinite: -106.264, 39.871		8.77	5.31	0.11	106.264°W	39.871°N	0.00	1.02
Bird Model Full Rupture	Fault							5.25
Gore Range frontal 50		2.06	6.95	-0.04	106.295°W	39.797°N	253.81	2.68
Bird Model Partial Rupture	Fault							4.73
Gore Range frontal 50		2.54	6.71	0.16	106.295°W	39.797°N	253.81	2.68
Zeng Model Full Rupture	Fault							1.92
Zeng Model Partial Rupture	Fault							1.65

Consulting
Engineers and
Scientists

Memo

To: Bruce Collins, Restruction Corporation
From: Chad Masching, P.E., Project Manager
Jim Niehoff, P.E., Reviewer
CC: Summit Trust
Date: February 16, 2018
Re: Upper Black Creek Spillway Geotechnical Investigation

GEI Consultants, Inc. (GEI) completed a geotechnical exploration at the Upper Black Creek Reservoir on December 28, 2017. The field work included subsurface drilling adjacent to and upstream of the spillway. Restruction Corporation (Restruction) was onsite to oversee the subsurface study and provided coring assistance through the spillway concrete slab to allow extraction of soil samples below the slab. This document summarizes the results of the exploration and associated laboratory testing program.

1. Background

The Upper Black Creek Reservoir is located approximately 24 miles northwest of Silverthorne, in Summit County, Colorado. Restruction, under contract to Summit Trust (Trust), was tasked with evaluating the condition of the concrete spillway structure and with developing options for restoration or replacement of the spillway. As part of the evaluation, Restruction evaluated the condition of the concrete by coring through the spillway concrete slab at five locations. Two cores were extracted from the upper portion of the spillway, two cores were extracted from the middle third of the structure, and one core was removed from near the bottom of the slab. After completing the lowest core, seepage exiting the core hole was noted, indicating a build-up of hydrostatic pressure under the slab.

The State Engineer's Office (SEO) met with Restruction and the Trust's dam tender on November 3, 2017 to discuss the observed spillway condition and the results of the concrete evaluation. Following this meeting, the SEO placed a storage restriction on the reservoir until improvements to the spillway could be made and recommended that a geotechnical investigation be performed to assist in the assessment of the spillway. Restruction contracted with GEI to perform the geotechnical study to aid in evaluating the spillway sub-surface condition.

GEI Consultants, Inc.
4601 DTC Boulevard, Suite 900, Denver, CO 80237
303.662.0100 fax: 303.662.8757
www.geiconsultants.com

2. Subsurface Exploration

2.1 Methods

GEI employed the services of Elite Drilling Services, LLC (Elite) to advance Hollow Stem Auger (HSA) borings through the spillway abutment and upstream of the spillway under the direction of the GEI geotechnical engineer. The exploration program included 4 locations, shown in Figure 1 below.

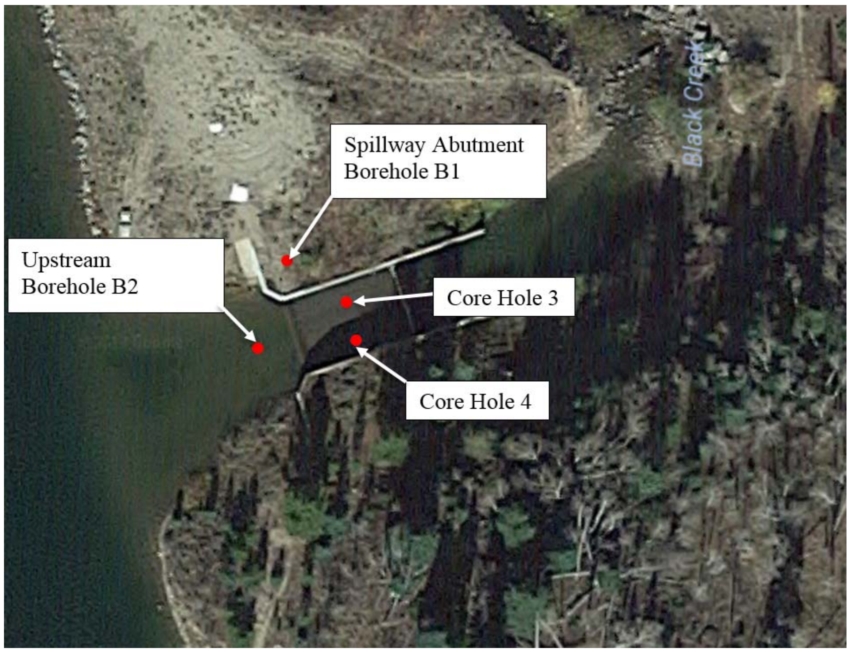


Figure 1: Approximate investigation locations (image by Google Earth, 2017)

Elite used a track-mounted CME 850 drill rig to advance the borings and collected continuous samples using Standard Penetration Tests (SPT) following ASTM D 1586-11. The boreholes were advanced to the depths of auger refusal, then were backfilled with bentonite chips above groundwater or coated bentonite pellets below the water table. Photo 1 shows drilling activities upstream of the spillway apron.

Photo 1: Track rig set up at borehole B2 upstream of the spillway apron

Restruction subsequently cored through two of the previous core holes located in the middle third of the spillway chute, known in the original Restruction report as Core Holes 3 and 4. GEI used a hand auger to collect soil samples from below the concrete slab at these locations. Photo 2 shows the difficult conditions encountered during the concrete coring.

Photo 2: Coring through concrete slab

2.2 Exploration Results

Boring B-1 was advanced through the left abutment (facing downstream) of the spillway, offset about 7 feet from the spillway walls to reduce the risk of intercepting the concrete wall footing. The hole was continuously sampled to a depth of 12 feet before auger refusal was encountered. Elite backfilled the borehole with bentonite chips and moved left (north) about 3 feet in an attempt to extend the boring to greater depth. The second attempt was planned to auger down to 12 feet before resuming sampling. However, auger refusal was encountered at 3 feet below ground surface (bgs). Due to the time of day and limited area to target another borehole in the abutment, it was decided to halt the drilling in the abutment and begin drilling upstream of the spillway.

Boring B-2 was located about 5 feet upstream of the concrete spillway apron. Continuous sampling was performed until refusal was encountered at a depth of 6 feet bgs. The test location was shifted an additional 5 feet upstream and continuous sampling was restarted at 6 feet and continued to a depth of 24 feet bgs. Both holes were abandoned using bentonite chips, with coated bentonite pellets used below the water table.

Both hand auger holes were advanced to a depth of about 2 feet below the bottom of concrete before encountering auger refusal. These core holes were filled with bentonite chips up to the bottom of the concrete. A repair concrete was utilized by Restruction to backfill the concrete section through the slab.

2.3 Subsurface Conditions

Based on the soil recovered in borehole B-1 located on the left spillway abutment, the spillway side walls appear to have been backfilled with a clayey sand material with cobbles within the soil matrix. Large cobbles were also observed along the edge of the spillway and downstream of the stilling basin. The clayey sand was judged to be medium dense, based on Standard Penetration Test (SPT) blow counts during the sampling. A loose zone of clayey sand was noted between 2 and 4 feet below the top of the dam.

Borehole B-2, drilled upstream of the spillway, recovered foundation soils consisting of a combination of medium dense to dense, clayey and silty sand with a large proportion of cobbles. During the drilling, cobbles up to 6 inches in diameter were observed in the auger cuttings.

Soils extracted in the two hand auger boreholes within the spillway chute were similar in composition to that encountered in borehole B-2, except more gravel was observed within the soil matrix. Samples retrieved at these two locations consisted of silty sands with gravels to clayey sands with gravels. Cobbles up to four inches were removed from both core holes. The concrete thickness was 12 to 13 inches.

At the Core Hole 3 location, a potential 3-inch void was noted under the spillway concrete. The concrete core removed at this location also had a “mushroom” shape, suggesting that the concrete replacing Restruction’s initial core hole may have filled this void (see Photo 3). However, because Restruction did not note any voiding below the slab during their concrete investigation, the void observed by GEI could have been the result of multiple core extractions at the core location.

Photo 3. View of the mushroom shape of the concrete core removed from Core Hole 3

Logs of the geotechnical investigation are provided in Attachment 1.

2.4 Laboratory Testing

Representative soil samples from the geotechnical exploration were selected for laboratory testing, which was completed by Hollingsworth Associates, Inc under contract to GEI. Five samples were selected to characterize the range of soil types and properties of the materials encountered in the foundation and adjacent to the spillway. Tests included Grain Size Analysis (ASTM D 6913) and Atterberg Limits (ASTM D 4318 Method A). Soils were generally classified as clayey sands and silty sands. These test results are summarized in Table 1 and are provided in Attachment 2.

Table 1. Summary of Laboratory Testing

Sample Location		Natural Moisture Content (%)	Gradation			Atterberg Limits	
Boring	Depth (feet bgs)		Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plasticity Index
B1	6-8	5	13	44	43	27	11
B2	4-5	8.3	16	58	26	23	8
B2	10.9-12.6	7.3	18	60	22		NP
Core Hole 3	1.3-1.8	18.2*	23	49	28		NP
Core Hole 4	1.8-2.5	8.6	19	60	21		NP

*Sample from Core Hole 3 was saturated, potentially as a result of coring activities

It should be noted that the split barrel sampler could not retrieve cobbles or large gravel. Consequently, the grain size analyses likely under-estimates the coarser constituents of the soil.

3. Conclusions

The encountered spillway soils consisted primarily of clayey sands or silty sands. Cobbles were encountered in all investigation locations and should be assumed to exist throughout the spillway backfill and native foundation soils. Bedrock was not encountered in any of the boreholes to the termination depths. Zones of clean sand (little silt or clay) were observed at the hand auger locations beneath the spillway chute slab. This clean sand could indicate that fine clay and silt particles have washed out due to prolonged periods of seepage below the concrete slab.

During Restruction’s concrete coring investigation, water was observed to emanate from the lowest concrete corehole in the spillway chute. This flowing water indicates a build-up of hydrostatic pressure below the slab. This water could be the result of development of preferential seepage paths below the spillway or could be attributed to the sandy nature of the foundation soils. Because the spillway slab is not anchored into the foundation rock, this uplift pressure reduces the net weight of the concrete and could lead to movement of the slab during spillway operations. Additionally, the water under the slab can lead to freeze/thaw heave of the structure. Spillway modifications should be completed which include construction of drainage provisions under the concrete slab.

The SEO has assisted the Trust in refining the previously completed hydrology study. This study indicated that the existing spillway does not have adequate capacity to pass the current Inflow Design Flood. GEI understands that the SEO is currently updating its hydrologic methodologies for high elevation dams. We recommend that the adequacy of the spillway to pass the Inflow Design Flood be reassessed once these SEO updates are available. If the new regulations significantly reduce the storm routing requirements and the spillway can be shown to pass the required storm inflow, options to rehabilitate the current spillway can be considered. If the spillway is still shown to be deficient in hydraulic capacity with these new guidelines, a replacement structure with a larger or more efficient spillway section should be assumed.

The soil properties obtained during the 2017 geotechnical investigation can be used for determining the adequacy of the existing upstream cutoff, designing underdrains, and for assessing the stability of the existing or proposed structure.

Attachment 1

Upper Black Creek Geotechnical Investigation
Boring Logs

BORING INFORMATION

STATION:
OFFSET:
GROUND SURFACE EL. (ft): NM
DATE START/END: 12/28/2017 - 12/28/2017
VERTICAL DATUM:
DRILLING COMPANY: Elite Drilling Services, LLC
TOTAL DEPTH (ft): 12.4
DRILLER NAME: Dan Westbrook
LOGGED BY: Joel Jackson
RIG TYPE: CME 850 Track

BORING B-1

DRILLING INFORMATION

HAMMER TYPE: Automatic
CASING I.D./O.D.: NA/ NA
CORE BARREL TYPE:
AUGER I.D./O.D.: 4.25 inch / 7.625 inch
DRILL ROD O.D.: NM
CORE BARREL I.D/O.D: NA / NA
DRILLING METHOD: Hollow Stem Auger
WATER LEVEL DEPTHS (ft): Not measured

ABBREVIATIONS:
Pen. = Penetration Length
Rec. = Recovery Length
RQD = Rock Quality Designation = Length of Sound Cores>4 in / Pen.,%
WOR = Weight of Rods
WOH = Weight of Hammer
S = Split Spoon Sample
C = Core Sample
U = Undisturbed Sample
B = Bag Sample
DP = Direct Push Sample
HSA = Hollow-Stem Auger
Qp = Pocket Penetrometer Strength
Sv = Pocket Torvane Shear Strength
LL = Liquid Limit
PI = Plasticity Index
PID = Photoionization Detector
I.D./O.D.= Inside Diameter/Outside Diameter
NA, NM = Not Applicable, Not Measured
Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.
OVM = Organic Vapor Meter

Elev. (ft)	Depth (ft)	Sample No.	Sample Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Graphic Log	Soil and Rock Description
	0–2	S1	0 to 2	24/13	15-8-5-4	PP = 4.25 Then Sample Broke		CLAYEY SAND (SC): brown, moist, 65% fine-coarse sand, 35% med plastic fines, organics (roots up to 1/8" diameter) CLAYEY GRAVEL (GC): brown, moist, 65% fine-coarse gravel, 20% fine-med sand, 15% med plastic fines
	2–4	S2	2 to 4	24/15	5-4-3-6			CLAYEY SAND (SC): brown, moist, 65% fine-coarse sand, 35% med plastic fines CLAYEY GRAVEL (GC): brown, moist, 65% fine-coarse gravel, 20% fine-med sand, 15% med plastic fines
	4–6	S3	4 to 6	24/14	12-12-8-6			CLAYEY SAND (SC): brown, moist, 65% fine-coarse sand, 35% med plastic fines SILTY SAND (SM): brown, moist, 75% fine-med sand, 25% non-plastic fines
	6–8	S4	6 to 8	24/13	7-8-4-4			Coarse Gravel or Cobble Pieces CLAYEY SAND (SC): brown, moist, 13% gravel, 44% fine-med sand, 43% low plastic fines, LL=27, PI= 11, Moisture= 5.0%
	8–10	S5	8 to 10	24/11	11-28-15-7			CLAYEY GRAVEL (GC) lense: white, moist, 85% coarse gravel, 15% low plastic fines CLAYEY SAND (SC) as above Coarse Gravel or Cobble Pieces
	10–12	S6	10 to 10.4	5/5	50/5"	Nails and metal fragments were recovered in sample. Possibly remnants of formwork used to construct the spillway. No concrete fragments were observed.		SILTY SAND (SM): brown, moist, 75% fine-med sand, 25% non to low-plastic fines CLAYEY SAND (SC): brown, moist, 10% fine gravel, 65% fine-med sand, 25% low plastic fines
	12.4							Auger Refusal at 12', Driller felt he was on a cobble/boulder that he couldn't advance past.

Depth scale (ft): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23

NOTES:

PROJECT NAME: Upper Black Creek
CITY/STATE: Heeney, Colorado
GEI PROJECT NUMBER: 1705095

GEI Consultants

BORING INFORMATION

STATION:		**OFFSET:**	
GROUND SURFACE EL. (ft):	NM	**DATE START/END:**	12/28/2017 - 12/28/2017
VERTICAL DATUM:		**DRILLING COMPANY:**	Elite Drilling Services, LLC
TOTAL DEPTH (ft):	22.9	**DRILLER NAME:**	Dan Westbrook
LOGGED BY:	Joel Jackson	**RIG TYPE:**	CME 850 Track

BORING B2

DRILLING INFORMATION

HAMMER TYPE:	Automatic	**CASING I.D./O.D.:**	NA/ NA	**CORE BARREL TYPE:**	
AUGER I.D./O.D.:	4.25 inch / 7.625 inch	**DRILL ROD O.D.:**	NM	**CORE BARREL I.D/O.D:**	NA / NA

DRILLING METHOD: Hollow Stem Auger

WATER LEVEL DEPTHS (ft): ▼ 16.0 12/28/2017

ABBREVIATIONS:
Pen. = Penetration Length
Rec. = Recovery Length
RQD = Rock Quality Designation = Length of Sound Cores>4 in / Pen.,%
WOR = Weight of Rods
WOH = Weight of Hammer
S = Split Spoon Sample
C = Core Sample
U = Undisturbed Sample
B = Bag Sample
DP = Direct Push Sample
HSA = Hollow-Stem Auger
Qp = Pocket Penetrometer Strength
Sv = Pocket Torvane Shear Strength
LL = Liquid Limit
PI = Plasticity Index
PID = Photoionization Detector
I.D./O.D.= Inside Diameter/Outside Diameter
NA, NM = Not Applicable, Not Measured
Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.
OVM = Organic Vapor Meter

Elev. (ft)	Depth (ft)	Sample No.	Sample Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Soil and Rock Description
	0–2	S1*	0 to 2	24/18	16-13-13-16		SILTY SAND (SM): moist, brown, 70% fine-med sand, 30% low plastic fines CLAYEY SAND w/ GRAVEL (SC): moist, dark brown, 25% fine-coarse gravel, 45% fine sand, 30% low-med plastic fines
	2–4	S2*	2 to 4	24/7	13-25-15-8	Rig chatter observed in the offset hole between 2.5 and 4 ft	POORLY GRADED GRAVEL w/ SILT (GP-GM): moist, 80% coarse gravel (granite chunks), 10% sand, 10% fines Coarse gravel in shoe (possibly a cobble that was drilled through)
	4–5.3	S3*	4 to 5.3	16/12	5-6-50/4"	Cuttings observed contained cobbles up to 6 inches in size. Shoe material was transitioning to Silty Sand (SM) and the tip was coated with granite powder.	CLAYEY SAND (SC): moist, dark brown, 16% fine gravel, 58% fine-med sand, 26% med plastic fines, LL=23, PI= 8, Moisture=8.3% Shoe material transitioned into silty sands, tip was small granite rock piece Auger refusal at 6.5' COBBLE fragments
	6–8	S1	6 to 8	24/15	21-17-11-11		SILTY SAND w/ GRAVEL (SM): moist, brown, 25% fine gravel, 55% fine to med sand, 20% non-plastic fines. Interbedded lenses of coarse gravel approximately 2" thick
	8–10	S2	8 to 10	24/19	7-12-12-9	*Auger refusal, offset 6' further upstream and begin sampling again at 6'	SILT (ML): moist, brown, 10% fine sand, 90% low plastic fines
	10–12	S3	10 to 12	24/24	5-12-22-7	PP = 1.75	SILTY SAND (SM): brown, moist, 20% fine-coarse gravel up to 1" dia, 55% fine-coarse sand, 25% non-plastic fines @ 11': SILTY SAND as above except 18% fine-coarse gravel, 60% fine-coarse sand, 22% non-plastic fines, Moisture= 7.3%
	12–14	S4	12 to 14	24/20	11-8-9-9		Saturated at 14'
	14–16	S5	14 to 16	24/16	7-9-9-10		
	16–18	S6	16 to 18	24/11	7-14-17-12		CLAYEY SAND (SC): wet, brown, 20% fine-coarse gravel, 55% fine-med sand, 25% low-med plastic fines
	18–20	S7	18 to 20	24/14	13-12-21-22		18' - 20' CLAYEY SAND (SC): 10% fine gravel, 60% fine-med sand, 30% low-med plastic fines 20' - 20.5' CLAYEY SAND (SC): 5% fine gravel, 60% sand, 35% low-med plastic fines
	20–22	S8	20 to 22	24/21	18-18-22-22		SILTY SAND (SM): moist, dark brown, 5% fine subrounded gravel, 55% fine-coarse sand, 40% non to low plastic fines 21.6' - 21.75' Coarse gravel, subrounded
	22.9–	S9	22.9 to	11/11	22-50/5"		@ 22: SILTY SAND as above except: wet, 70% fine-med sand, 30% non to low plastic fines

NOTES:

PROJECT NAME: Upper Black Creek
CITY/STATE: Heeney, Colorado
GEI PROJECT NUMBER: 1705095

GEI Consultants

STATION:		OFFSET:		BORING B-2
GROUND SURFACE EL. (ft):		DATE START/END:	12/28/2017 - 12/28/2017	PAGE 2 of 2
VERTICAL DATUM:		DRILLING COMPANY:	Elite Drilling Services, LLC	

Elev. (ft)	Depth (ft)	Sample Information: Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Graphic Log	Soil and Rock Description
			23.8					Igneous rock fragments, dark grey up to 2' diameter, unclear if it is bedrock or cobble
	25							Log is a compilation of two holes drilled in close proximity.
	26							
	27							
	28							
	29							
	30							
	31							
	32							
	33							
	34							
	35							
	36							
	37							
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	53							
	54							
	55							

NOTES:

PROJECT NAME: Upper Black Creek

CITY/STATE: Heeney, Colorado

GEI PROJECT NUMBER: 1705095

GEI Consultants

BORING INFORMATION			BORING Core 3
STATION:	OFFSET:		
GROUND SURFACE EL. (ft): NM	DATE START/END: 12/28/2017 - 12/28/2017		
VERTICAL DATUM:	DRILLING COMPANY: GEI		
TOTAL DEPTH (ft): 2.0	DRILLER NAME: Ben Kuchta		
LOGGED BY: Ben Kuchta	RIG TYPE: Hand Auger		PAGE 1 of 1

DRILLING INFORMATION

HAMMER TYPE: NA	CASING I.D./O.D.: NA/ NA	CORE BARREL TYPE:
AUGER I.D./O.D.: NA / 3 inch	DRILL ROD O.D.: NM	CORE BARREL I.D/O.D: NA / NA
DRILLING METHOD: Hand Auger		
WATER LEVEL DEPTHS (ft): Not measured		

ABBREVIATIONS:

Pen. = Penetration Length	S = Split Spoon Sample	Qp = Pocket Penetrometer Strength	NA, NM = Not Applicable, Not Measured
Rec. = Recovery Length	C = Core Sample	Sv = Pocket Torvane Shear Strength	Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.
RQD = Rock Quality Designation = Length of Sound Cores>4 in / Pen.,%	U = Undisturbed Sample	LL = Liquid Limit	
	B = Bag Sample	PI = Plasticity Index	
WOR = Weight of Rods	DP = Direct Push Sample	PID = Photoionization Detector	
WOH = Weight of Hammer	HSA = Hollow-Stem Auger	I.D./O.D.= Inside Diameter/Outside Diameter	OVM = Organic Vapor Meter

Elev. (ft)	Depth (ft)	Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Graphic Log	Soil and Rock Description
	1							CONCRETE
	2	S1 S2 S3						Possible void up to 3". Concrete coring utilized water that could have washed out this material. Removed concrete had a mushroom shape on the bottom.
	3							SILTY SAND (SM): brown, saturated (possibly due to water from coring), 23% fine-coarse subrounded to subangular gravel, 49% fine-coarse sand (predominately fine-med), 2% non-plastic fines
	4							CLAYEY SAND (SC): brown, saturated (likely impacted from water coring), 10% fine-coarse subangular gravel, 55% fine-coarse subangular sand, 35% med plasticity fines, Moisture= 18.2%
	5							2.1' refusal with the hand auger. Sampled with hand, fine to coarse gravel, finer parts of the soil could have been washed away due to 4" of standing water in hole.
	6							Hand auger refusal at 2'.
	7							
	8							
	9							
	10							
	11							
	12							
	13							
	14							
	15							
	16							
	17							
	18							
	19							
	20							
	21							
	22							
	23							

NOTES: Normal to spillway	PROJECT NAME: Upper Black Creek
	CITY/STATE: Heeney, Colorado
	GEI PROJECT NUMBER: 1705095

GEI Consultants

BORING INFORMATION

STATION:
OFFSET:
GROUND SURFACE EL. (ft): NM
DATE START/END: 12/28/2017 - 12/28/2017
VERTICAL DATUM:
DRILLING COMPANY: GEI
TOTAL DEPTH (ft): 2.6
DRILLER NAME: Ben Kuchta
LOGGED BY: Ben Kuchta
RIG TYPE: Hand Auger

BORING Core 4

DRILLING INFORMATION

HAMMER TYPE: NA
CASING I.D./O.D.: NA/ NA
CORE BARREL TYPE:
AUGER I.D./O.D.: NA / 3 inch
DRILL ROD O.D.: NM
CORE BARREL I.D/O.D: NA / NA
DRILLING METHOD: Hand Auger
WATER LEVEL DEPTHS (ft): Not measured

ABBREVIATIONS:
Pen. = Penetration Length
Rec. = Recovery Length
RQD = Rock Quality Designation = Length of Sound Cores>4 in / Pen.,%
WOR = Weight of Rods
WOH = Weight of Hammer
S = Split Spoon Sample
C = Core Sample
U = Undisturbed Sample
B = Bag Sample
DP = Direct Push Sample
HSA = Hollow-Stem Auger
Qp = Pocket Penetrometer Strength
Sv = Pocket Torvane Shear Strength
LL = Liquid Limit
PI = Plasticity Index
PID = Photoionization Detector
I.D./O.D.= Inside Diameter/Outside Diameter
NA, NM = Not Applicable, Not Measured
Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.
OVM = Organic Vapor Meter

Elev. (ft)	Depth (ft)	Sample Information: Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Graphic Log	Soil and Rock Description
	0–0.5							CONCRETE
	0.5–1.8	S1						CLAYEY SAND w/ GRAVEL (SC): brown, moist, 35% fine-coarse subrounded-subangular gravel, up to 1.5" diameter, 50% fine-med sand, 15% low-med plastic fines. 5" diameter cobble located at approximately 2'
	1.8–2.6	S2						
		S3						SILTY SAND (SM): brown, moist, 19% fine-coarse, subrounded gravel up to 1 inch in diameter, 60% fine-med sand, 21% non-plastic fines, Moisture= 8.6%
		S4						SILTY SAND (SM) as above except 5% fine gravel, 70% fine-med sand, 25% non-low plastic fines
	2.6							Hand auger refusal at 2.6'.

Depth scale (ft): 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

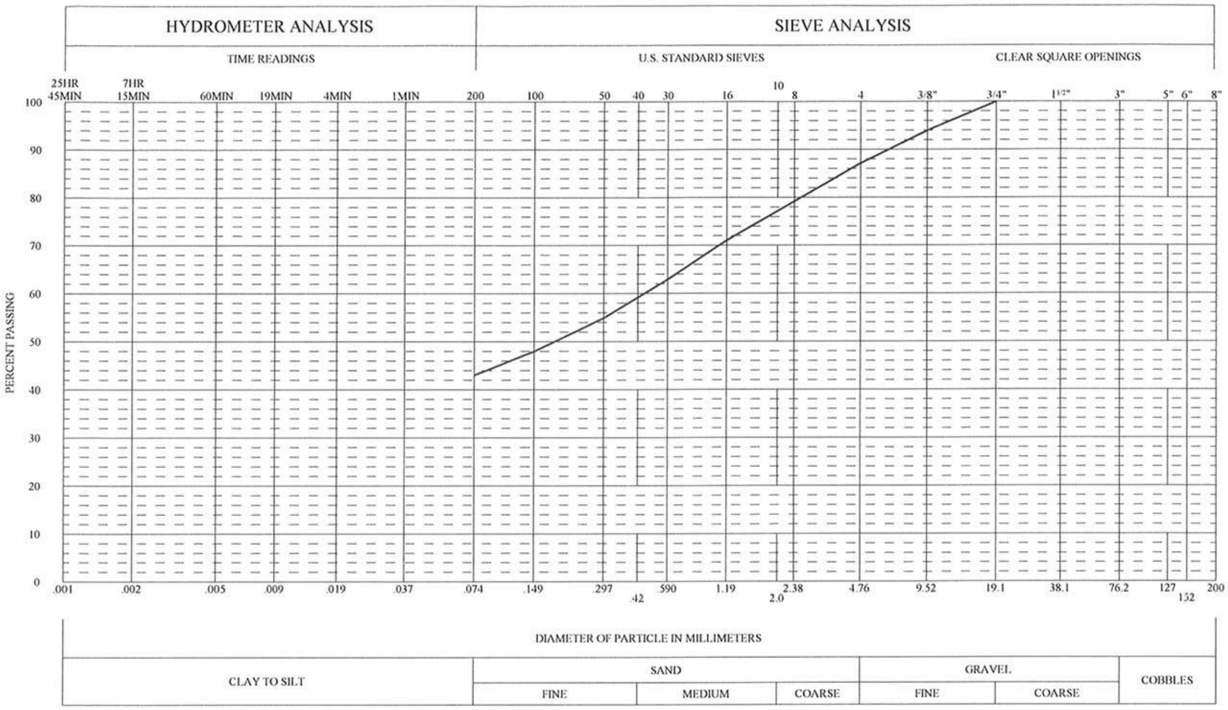
NOTES: Normal to spillway

PROJECT NAME: Upper Black Creek
CITY/STATE: Heeney, Colorado
GEI PROJECT NUMBER: 1705095

GEI Consultants

Attachment 2

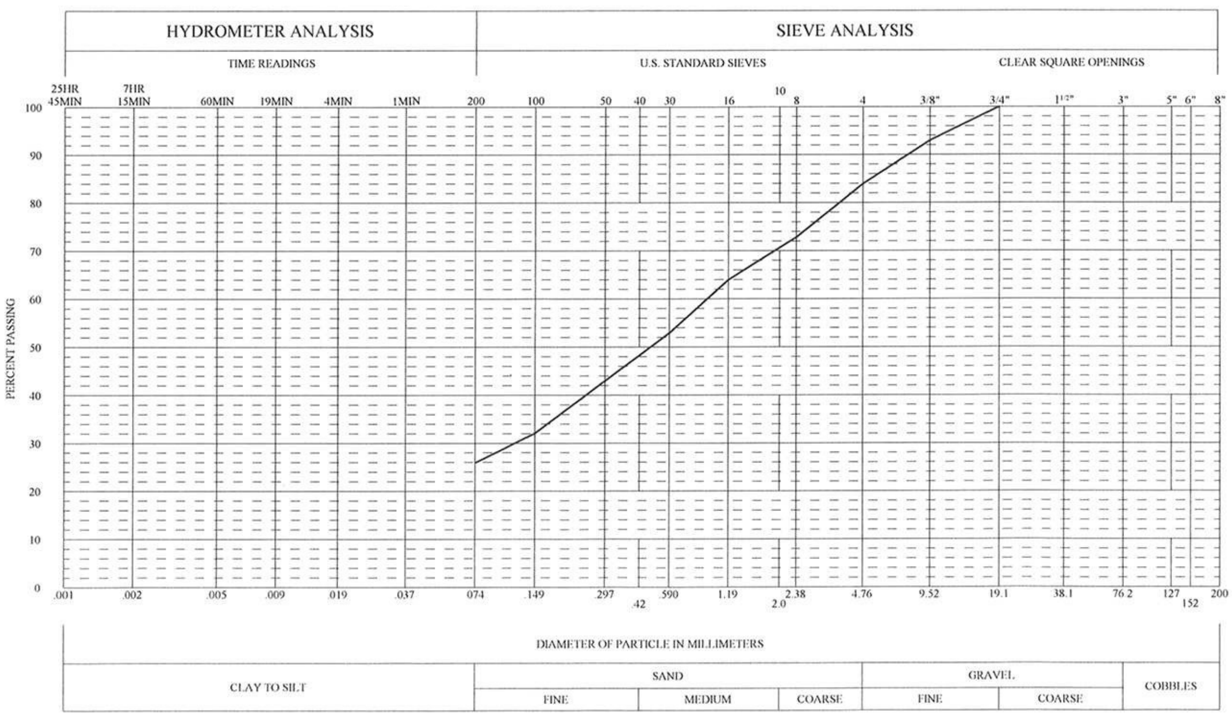
Upper Black Creek Geotechnical Investigation
Laboratory Testing



GRAVEL 13 % SAND 44 % SILT AND CLAY 43 %

LIQUID LIMIT 27 % PLASTICITY INDEX 11 %

SAMPLE OF FROM B-1, S4A & S4C

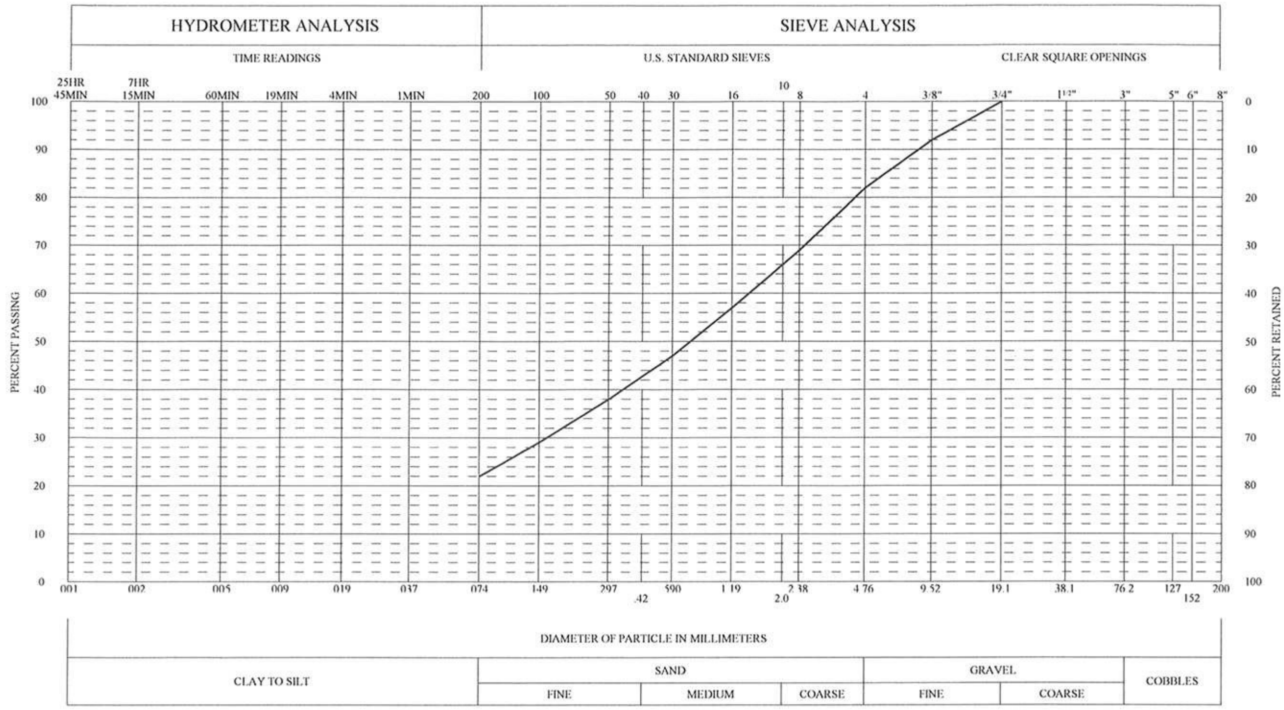


GRAVEL 16 % SAND 58 % SILT AND CLAY 26 %

LIQUID LIMIT 23 % PLASTICITY INDEX 8 %

SAMPLE OF FROM B-2, S3

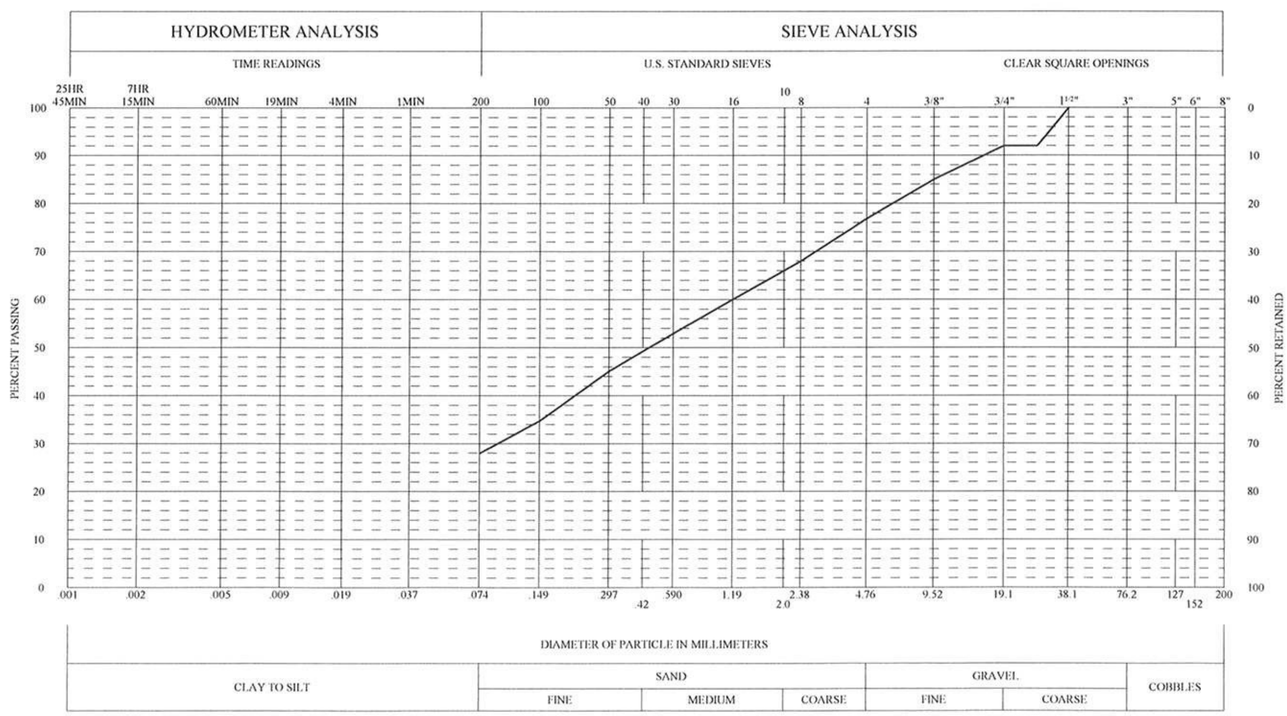
18-26	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Gradation Test Results	Fig. 1



GRAVEL 18 %   SAND 60 %   SILT AND CLAY 22 %

LIQUID LIMIT %   PLASTICITY INDEX NP %

SAMPLE OF   FROM B2A, S3B & S4A

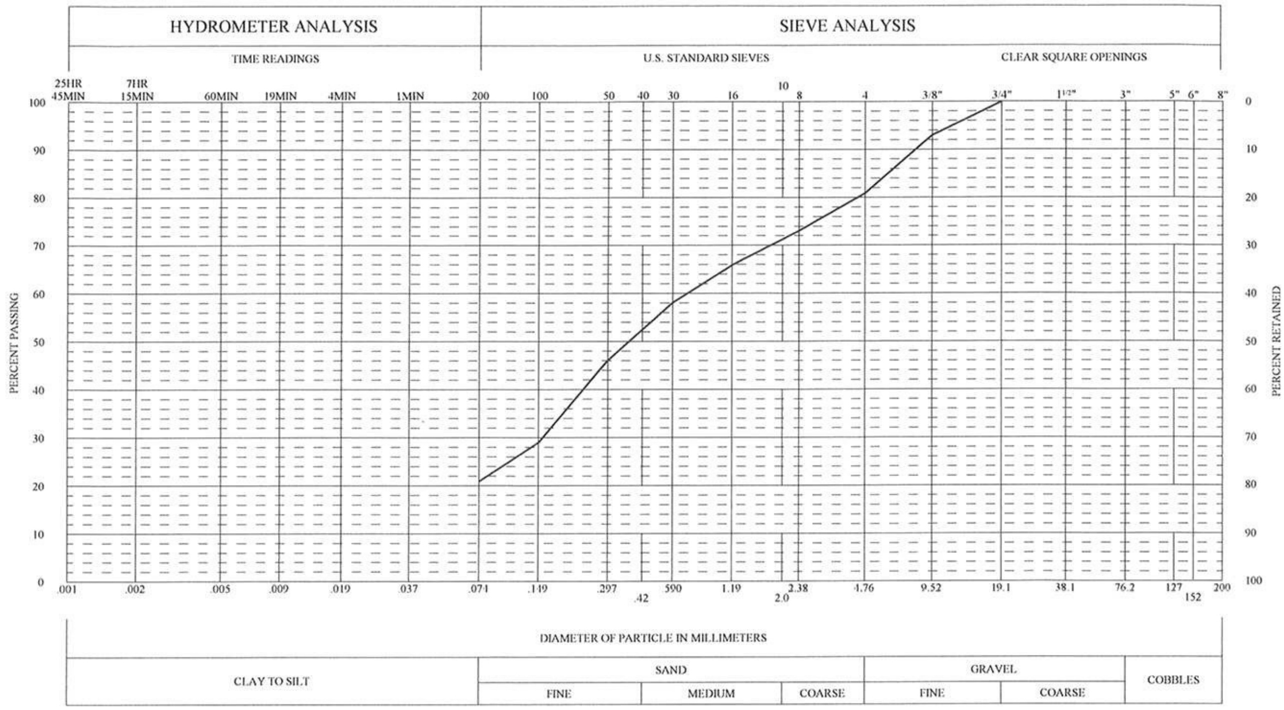


GRAVEL 23 %   SAND 49 %   SILT AND CLAY 28 %

LIQUID LIMIT %   PLASTICITY INDEX NP %

SAMPLE OF   FROM Core Hole 3, S1 & S2

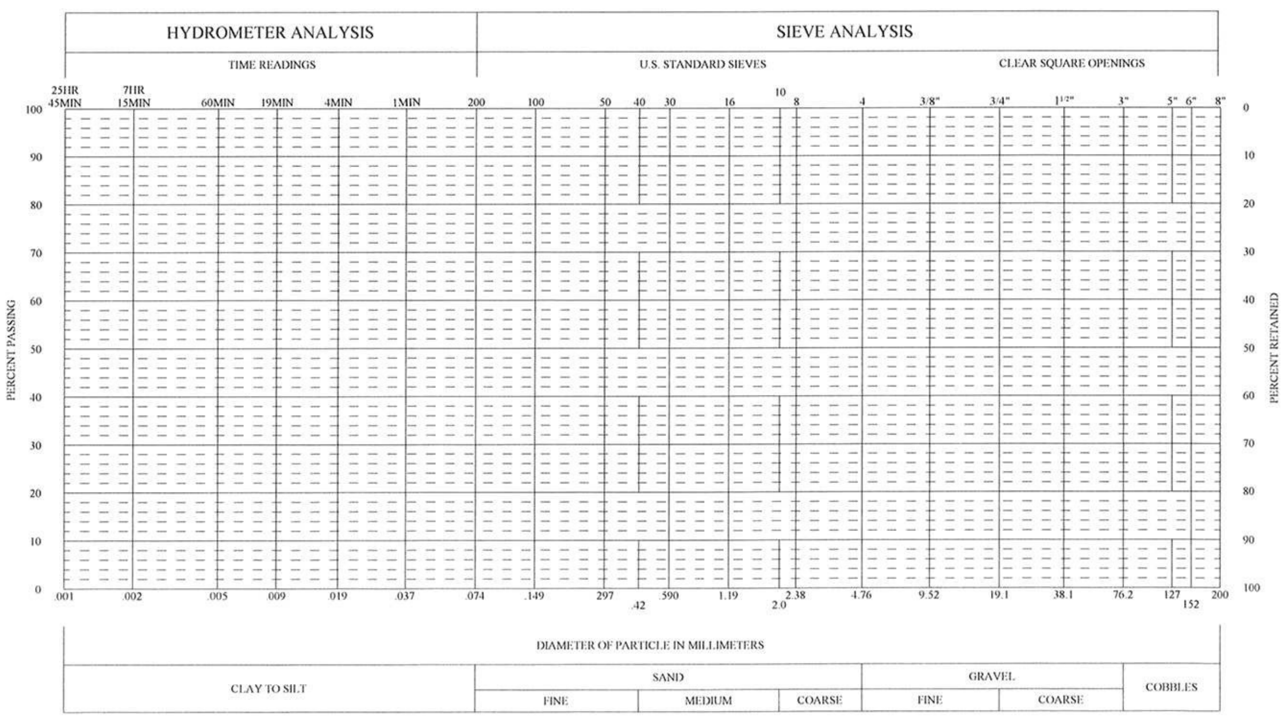
18-26	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Gradation Test Results	Fig. 2



GRAVEL 19 %　　SAND 60 %　　SILT AND CLAY 21 %

LIQUID LIMIT %　　PLASTICITY INDEX NP %

SAMPLE OF　　FROM Core Hole 4, S2 & S3



GRAVEL %　　SAND %　　SILT AND CLAY %

LIQUID LIMIT %　　PLASTICITY INDEX %

SAMPLE OF　　FROM

Hollingsworth Associates, Inc.

Job No.: 18-26

TABLE I
SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content	Natural Dry Density	Gradation			Atterberg Limits		Soil Type
Boring	Sample No.	(%)	(pcf)	Gravel + No.4 (%)	Sand - No. 4 + No. 200 (%)	Minus No. 200 Sieve (%)	Liquid Limit (%)	Plasticity Index (%)	
B1	S4A&S4C	5.0		13	44	43	27	11	
B2	S3	8.3		16	58	26	23	8	
B2A	S3B&S4A	7.3		18	60	22		NP	
Core Hole 3	S1&S2	18.2		23	49	28		NP	
Core Hole 4	S2&S3	8.6		19	60	21		NP	

HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

MOISTURE-DENSITY RELATIONSHIP

Job Number: 18-26
Client/Location: GEI Consultants - Upper Black Creek
Project No. 1705095

Prep By: BW Date: 1/12
Run By: BW Date: 1/12
Calc by: TH Date: 1/15/18

Boring:	B-1	B-2	B-2A	Core Hole 3
Sample:	S4A&S4C	S3	S3B&S4A	S1&S2
Blow Count:				
Soil Classification:				
Remarks:				
Density				
Length: (in)				
Diameter: (in)				
Volume: (ft^3)				
Wet Soil+Liner: (gm)				
Liner: (gm)				
Wet Soil Wt: (gm)				
Dry Soil Wt: (gm)				
Dry Density: (pcf)				
Moisture				
Dish Name:	MT	PC	AZ4	ZY
Wet Soil+Dish:	355.77 gm	352.86 gm	355.57 gm	575.49 gm
Dry Soil+Dish:	349.12 gm	343.96 gm	345.97 gm	520.09 gm
Dish Wt.	216.15 gm	236.15 gm	214.04 gm	215.26 gm
Wt of Water:	6.65 gm	8.90 gm	9.60 gm	55.40 gm
Wt of Dry Soil:	132.97 gm	107.81 gm	131.93 gm	304.83 gm
Moisture:	5.0 %	8.3 %	7.3 %	18.2 %

HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

MOISTURE-DENSITY RELATIONSHIP

Job Number:	18-26	Prep By:	BW	Run By:	BW	Calc by:	TH
Client/Location:	GEI Consultants - Upper Black Creek	Date:	1/12	Date:	1/12	Date:	1/15/18
	Project No. 1705095						

Boring:	Core Hole 4			
Sample:	S2&S3			
Blow Count:				
Soil Classification:				
Remarks:				
Density				
Length:	in	in	in	in
Diameter:	in	in	in	in
Volume:	ft^3	ft^3	ft^3	ft^3
Wet Soil+Liner:	gm	gm	gm	gm
Liner:	gm	gm	gm	gm
Wet Soil Wt:	gm	gm	gm	gm
Dry Soil Wt:	gm	gm	gm	gm
Dry Density:	pcf	pcf	pcf	pcf
Moisture				
Dish Name:	BOB			
Wet Soil+Dish:	426.73 gm	gm	gm	gm
Dry Soil+Dish:	411.08 gm	gm	gm	gm
Dish Wt.	229.95 gm	gm	gm	gm
Wt of Water:	15.65 gm	gm	gm	gm
Wt of Dry Soil:	181.13 gm	gm	gm	gm
Moisture:	8.6 %	%	%	%

HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

GRADATION ANALYSIS

Job Number: 18-26
Client/Location: GEI Consultants - Upper Black Creek, Project No. 1705095

Prep By: BW Date: 1/12
Run By: BW Date: 1/16
Calc by: TH Date: 1/16/18

Boring: B-1

Sample: S4A & S4C
Blow Count:
Soil Classification:

Dish No.:	MT	
Dry Soil & Dish:	349.12	gm
-200 Soil & Dish:	292.61	gm
Dish Weight:	216.15	gm
Dry Soil Weight:	132.97	gm

Sieve Size	Cum Wt Retained	Percent Passing
6		100.0
3		100.0
1.5		100.0
3/4		100.0
3/8	8.36	93.7
4	17.53	86.8
8	28.00	78.9
16	38.87	70.8
30	49.14	63.0
50	59.27	55.4
100	69.36	47.8
200	76.08	42.8
PAN	76.52	42.5
Gravel %		13.2
Sand %		44.0
Clay-Silt %		42.8

Boring: B-2

Sample: S3
Blow Count:
Soil Classification:

Dish No.:	PC	
Dry Soil & Dish:	343.96	gm
-200 Soil & Dish:	316.16	gm
Dish Weight:	236.15	gm
Dry Soil Weight:	107.81	gm

Sieve Size	Cum Wt Retained	Percent Passing
6		100.0
3		100.0
1.5		100.0
3/4		100.0
3/8	7.51	93.0
4	17.03	84.2
8	28.60	73.5
16	39.08	63.8
30	50.21	53.4
50	61.38	43.1
100	72.96	32.3
200	79.63	26.1
PAN	80.01	25.8
Gravel %		15.8
Sand %		58.1
Clay-Silt %		26.1

Boring: B-2A

Sample: S3B & S4A
Blow Count:
Soil Classification:

Dish No.:	AZ4	
Dry Soil & Dish:	345.97	gm
-200 Soil & Dish:	317.61	gm
Dish Weight:	214.04	gm
Dry Soil Weight:	131.93	gm

Sieve Size	Cum Wt Retained	Percent Passing
6		100.0
3		100.0
1.5		100.0
3/4		100.0
3/8	10.88	91.8
4	23.84	81.9
8	40.96	69.0
16	56.77	57.0
30	69.98	47.0
50	81.79	38.0
100	93.83	28.9
200	102.86	22.0
PAN	103.56	21.5
Gravel %		18.1
Sand %		59.9
Clay-Silt %		22.0

Boring: Core Hole 3

Sample: S1 & S2
Blow Count:
Soil Classification:

Dish No.:	ZY	
Dry Soil & Dish:	520.09	gm
-200 Soil & Dish:	436.59	gm
Dish Weight:	215.26	gm
Dry Soil Weight:	304.83	gm

Sieve Size	Cum Wt Retained	Percent Passing
6		100.0
3		100.0
1	23.16	92.4
3/4	23.16	92.4
3/8	47.12	84.5
4	69.94	77.1
8	96.49	68.3
16	121.15	60.3
30	143.74	52.8
50	167.87	44.9
100	198.58	34.9
200	220.02	27.8
PAN	221.36	27.4
Gravel %		22.9
Sand %		49.2
Clay-Silt %		27.8

HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

GRADATION ANALYSIS

Job Number: 18-26
Client/Location: GEI Consultants - Upper Black Creek, Project No. 1705095

Prep By: BW Date: 1/12
Run By: CM Date: 1/16
Calc by: TH Date: 1/16/18

Boring: Core Hole 4
Sample: S2 & S3
Blow Count:
Soil Classification:

Dish No.:	BOB	
Dry Soil & Dish:	411.08	gm
-200 Soil & Dish:	373.55	gm
Dish Weight:	229.95	gm
Dry Soil Weight:	181.13	gm

Sieve Size	Cum Wt Retained	Percent Passing
6		100.0
3		100.0
1.5		100.0
3/4		100.0
3/8	12.47	93.1
4	34.25	81.1
8	48.05	73.5
16	61.20	66.2
30	75.84	58.1
50	98.33	45.7
100	128.72	28.9
200	143.47	20.8
PAN	144.13	20.4
Gravel %		18.9
Sand %		60.3
Clay-Silt %		20.8

Boring:
Sample:
Blow Count:
Soil Classification:

Dish No.:		
Dry Soil & Dish:		gm
-200 Soil & Dish:		gm
Dish Weight:		gm
Dry Soil Weight:		gm

Sieve Size	Cum Wt Retained	Percent Passing
6		
3		
1.5		
3/4		
3/8		
4		
8		
16		
30		
50		
100		
200		
PAN		
Gravel %		
Sand %		
Clay-Silt %		

Boring:
Sample:
Blow Count:
Soil Classification:

Dish No.:		
Dry Soil & Dish:		gm
-200 Soil & Dish:		gm
Dish Weight:		gm
Dry Soil Weight:		gm

Sieve Size	Cum Wt Retained	Percent Passing
6		
3		
1.5		
3/4		
3/8		
4		
8		
16		
30		
50		
100		
200		
PAN		
Gravel %		
Sand %		
Clay-Silt %		

Boring:
Sample:
Blow Count:
Soil Classification:

Dish No.:		
Dry Soil & Dish:		gm
-200 Soil & Dish:		gm
Dish Weight:		gm
Dry Soil Weight:		gm

Sieve Size	Cum Wt Retained	Percent Passing
6		
3		
1.5		
3/4		
3/8		
4		
8		
16		
30		
50		
100		
200		
PAN		
Gravel %		
Sand %		
Clay-Silt %		

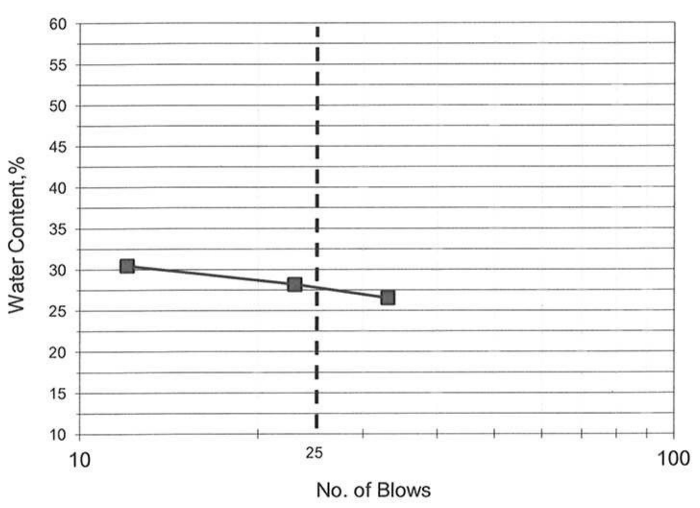
HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

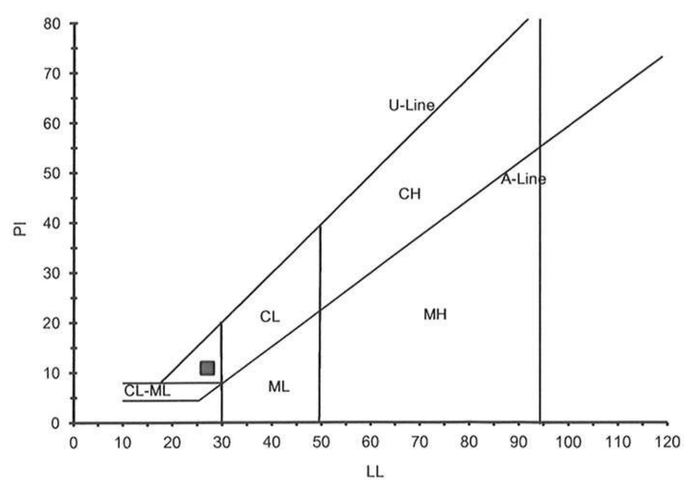
ATTERBERG LIMITS

Job Number: 18-26 Prep By: ________ Run By: ________ Calc by: LG
Client/Location: GEI Consultants Date: ________ Date: ________ Date: 1/16/18
Upper Black Creek
Boring: B-1 Depth: S4A, C Blow Count: ________ Sample No.: ________
Soil Classification: ________

Liquid Limit Determination					
Dish Name:	C4	C7	C8		
Wet Soil & Dish:	35.07	33.52	32.54		
Dry Soil & Dish:	32.90	31.40	30.73		
Dish Weight:	25.78	23.88	23.91		
Wt. Of Water:	2.17	2.12	1.81		
Wt. Of Dry Soil:	7.12	7.52	6.82		
Water Content, %:	30.48	28.19	26.54		
No. of Blows, N	12	23	33		

Liquid Limit, LL 27
Plastic Limit, PL 16
Plasticity Index, PI 11





Plastic Limit Determination					
Dish Name:	SS	SC			
Wet Soil & Dish:	22.02	22.05			
Dry Soil & Dish:	20.85	20.89			
Dish Weight:	13.63	13.63			
Wt. Of Water:	1.17	1.16			
Wt. Of Dry Soil:	7.22	7.26			
Water Content, %:	16.2	16.0			

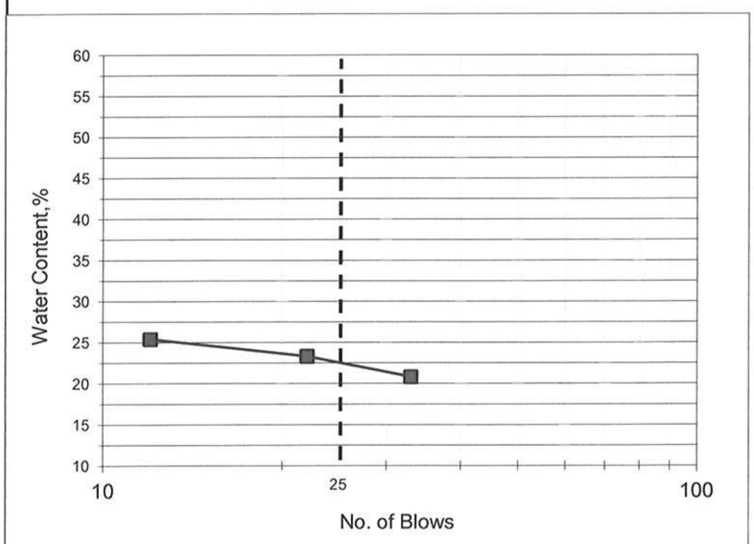
HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

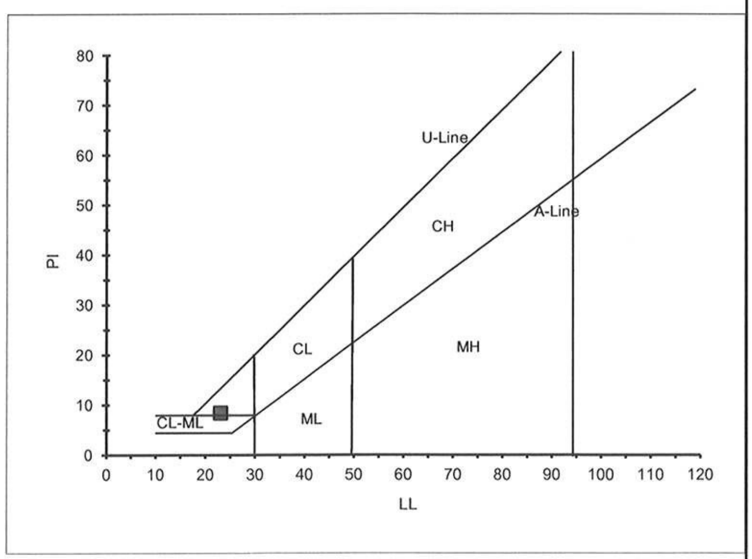
ATTERBERG LIMITS

Job Number: 18-26 Prep By: ________ Run By: ________ Calc by: LG
Client/Location: GEI Consultants Date: ________ Date: ________ Date: 1/16/18
Upper Black Creek
Boring: B2 Depth: S3 Blow Count: ________ Sample No.: ________
Soil Classification: ________________

Liquid Limit Determination					
Dish Name:	MJ	PF	SAC		
Wet Soil & Dish:	21.13	20.10	22.81		
Dry Soil & Dish:	19.61	18.88	21.23		
Dish Weight:	13.63	13.65	13.64		
Wt. Of Water:	1.52	1.22	1.58		
Wt. Of Dry Soil:	5.98	5.23	7.59		
Water Content, %:	25.42	23.33	20.82		
No. of Blows, N	12	22	33		

Liquid Limit, LL 23
Plastic Limit, PL 15
Plasticity Index, PI 8





Plastic Limit Determination					
Dish Name:	QT	AJ			
Wet Soil & Dish:	23.12	23.13			
Dry Soil & Dish:	21.91	21.92			
Dish Weight:	13.62	13.60			
Wt. Of Water:	1.21	1.21			
Wt. Of Dry Soil:	8.29	8.32			
Water Content, %:	14.6	14.5			

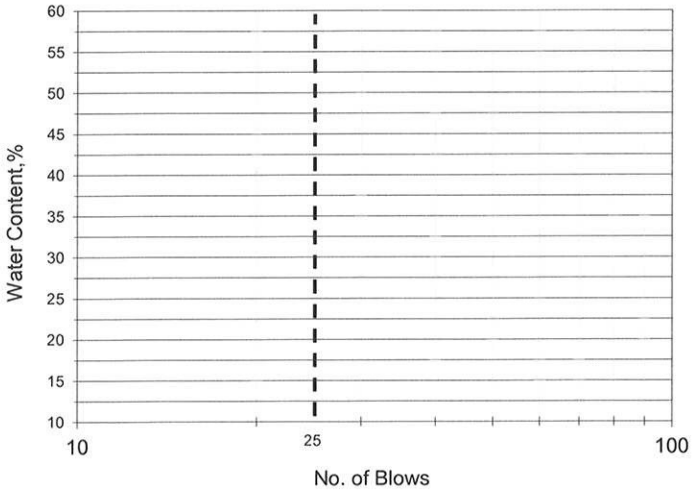
HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

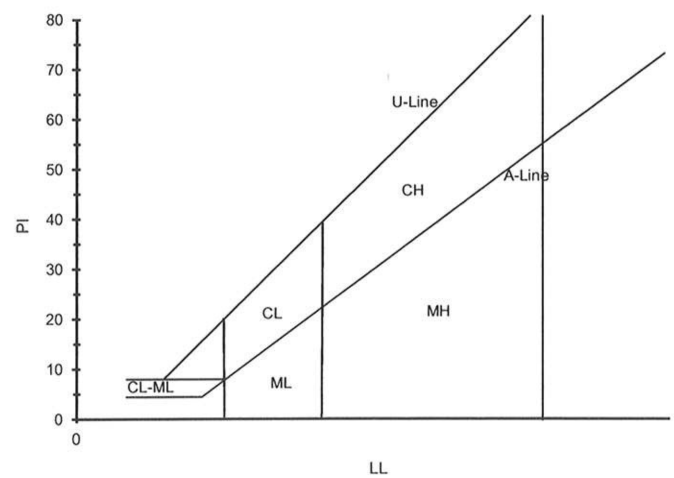
ATTERBERG LIMITS

Job Number: 18-26 Prep By: ________ Run By: ________ Calc by: LG
Client/Location: GEI Consultants Date: ________ Date: ________ Date: 1/16/18
Upper Black Creek
Boring: B2A Depth: S3&S4A Blow Count: ________ Sample No.: ________
Soil Classification: ________

Liquid Limit Determination					
Dish Name:					
Wet Soil & Dish:					
Dry Soil & Dish:					
Dish Weight:					
Wt. Of Water:					
Wt. Of Dry Soil:					
Water Content, %:					
No. of Blows, N					

Liquid Limit, LL ________
Plastic Limit, PL ________
Plasticity Index, PI NP





Plastic Limit Determination					
Dish Name:					
Wet Soil & Dish:					
Dry Soil & Dish:					
Dish Weight:					
Wt. Of Water:					
Wt. Of Dry Soil:					
Water Content, %:					

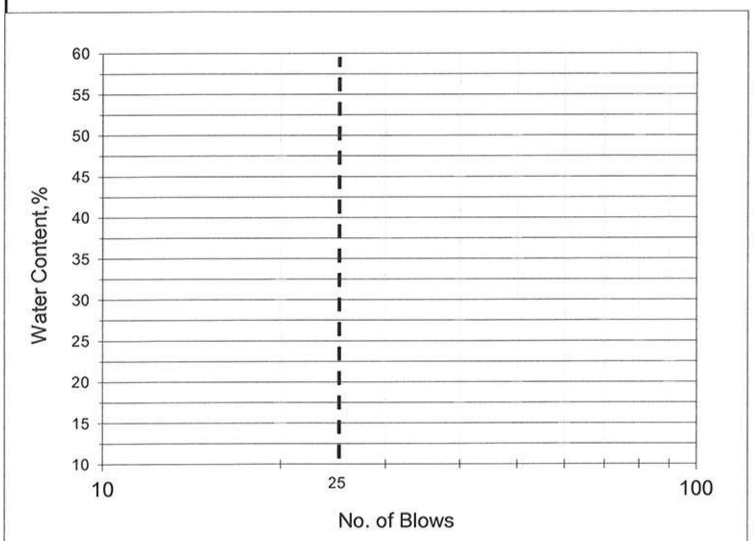
HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

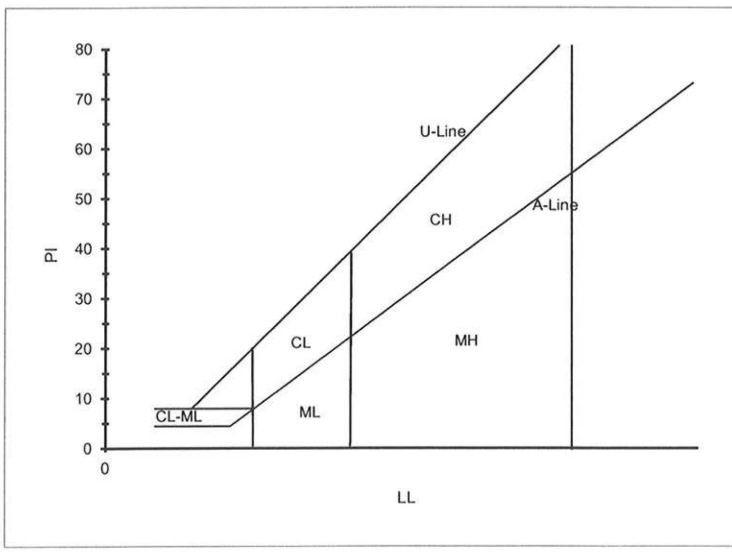
ATTERBERG LIMITS

Job Number: 18-26 Prep By: ________ Run By: ________ Calc by: LG
Client/Location: GEI Consultants Date: ________ Date: ________ Date: 1/16/18
Upper Black Creek
Boring: CORE HOLE 3 Depth: S1&S2 Blow Count: ________ Sample No.: ________
Soil Classification: ________

Liquid Limit Determination					
Dish Name:					
Wet Soil & Dish:					
Dry Soil & Dish:					
Dish Weight:					
Wt. Of Water:					
Wt. Of Dry Soil:					
Water Content, %:					
No. of Blows, N					

Liquid Limit, LL ________
Plastic Limit, PL ________
Plasticity Index, PI NP





Plastic Limit Determination					
Dish Name:					
Wet Soil & Dish:					
Dry Soil & Dish:					
Dish Weight:					
Wt. Of Water:					
Wt. Of Dry Soil:					
Water Content, %:					

HOLLINGSWORTH ASSOCIATES
Geotechnical/Environmental Engineers

ATTERBERG LIMITS

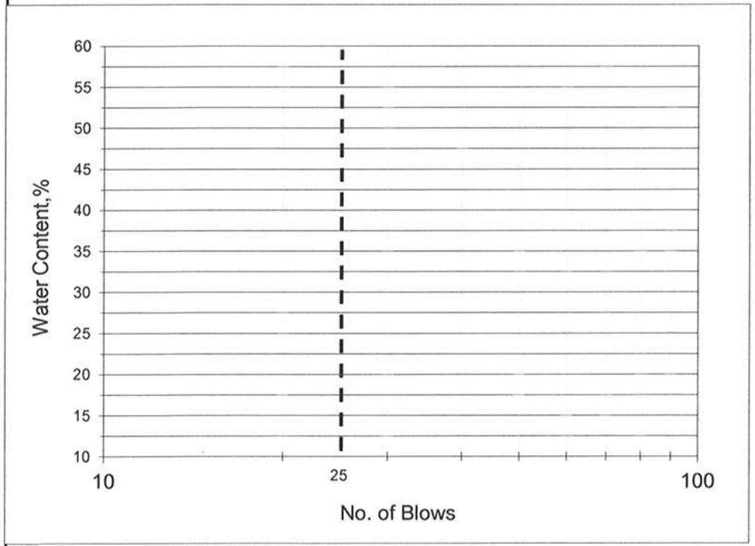
Job Number:	18-26	Prep By:		Run By:		Calc by:	LG
Client/Location:	GEI Consultants	Date:		Date:		Date:	1/16/18
	Upper Black Creek						

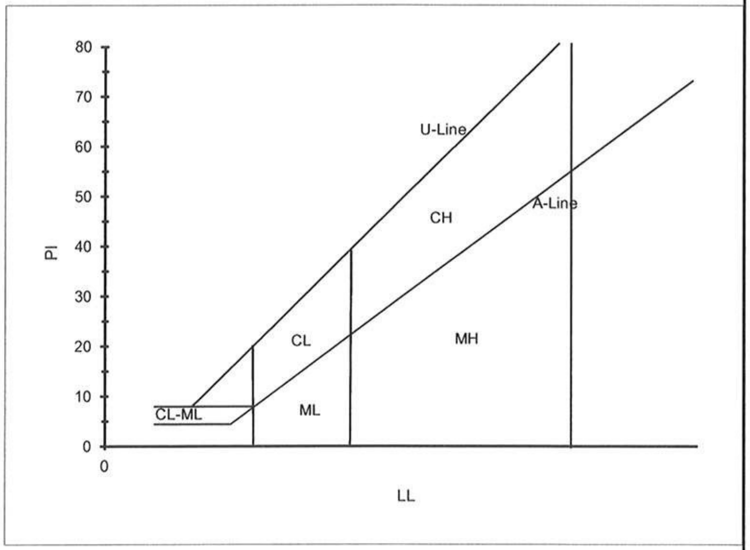
Boring: CORE HOLE 4 Depth: S2&S3 Blow Count: ______ Sample No.: ______

Soil Classification: ______

Liquid Limit Determination					
Dish Name:					
Wet Soil & Dish:					
Dry Soil & Dish:					
Dish Weight:					
Wt. Of Water:					
Wt. Of Dry Soil:					
Water Content, %:					
No. of Blows, N					

Liquid Limit, LL ______
Plastic Limit, PL ______
Plasticity Index, PI NP





Plastic Limit Determination					
Dish Name:					
Wet Soil & Dish:					
Dry Soil & Dish:					
Dish Weight:					
Wt. Of Water:					
Wt. Of Dry Soil:					
Water Content, %:					

Appendix D Structural Calculations

Spillway Wall (Head Wall) - Upper Black Creek Reservoir

Codes References

1) American Concrete Institute (ACI). Building Code Requirements for Structural Concrete (ACI 318-14), (2014).
2) ACI (American Concrete Institute), 2006. Code Requirements for Environmental Engineering Concrete Structures (ACI 350-06).
3) American Society of Civil Engineers (ASCE). Minimum Design Loads for Buildings and Other Structures (ASCE 7-10), (2013).
4) U.S. Army Corps of Engineers (USACE). 2005 "Stability Analysis of Concrete Structures." EM 1110-2-2502.
5) U.S. Army Corps of Engineers (USACE). 2003 "Strength Design for Reinforced-Concrete Hydraulic Structures." EM 1110-2-2104-Appendix E: Table E-1.
6) Agusti, G. C. and Sitar. 2013. UCB GT 13-02 "Seismic Earth Pressures on Retaining Structures with Cohesive Backfills." UCB GT 13-02.

Structural Design

The following design calculates the applied loads and design strength for the Upper Black Creek Reservoir Spillway Head Wall. The wall is designed as a cantilevered retaining wall with soil and seismic loads. The strength/capacity checks are performed using the one foot strip method and code requirements in ACI 318-14, ACI 350-06 and USACE EM 1110-2-2104.

Design Loads (Forces)

The applied loads and load cases for the wall are calculated using guidelines from the documents referenced above. USACE and ACI strength/service load combinations are applied and the resulting combinations are shown below.

Wall Reinforcement Summary

Vertical Reinforcement

Tension Face = #7 at 12" O.C.

Compression Face = #7 at 12" O.C.

Horizontal Reinforcement

Channel = #5 at 12" O.C.

Embankment = #5 at 12" O.C.

Foundation Dowels

Full height bars, mechanical splices or slab tension splices will be allowed. Bars must be developed at wall/foundation joint.

Counterforts

None

Footing Reinforcement Summary

Perpendicular to Wall

Tension Face = #9 at 12" O.C.

Compression Face = #9 at 12" O.C.

Parallel to Wall

Top = #7 at 12" O.C.

Bottom = #7 at 12" O.C.

Wall Design Properties:

Top of Wall Elevation:	$El_{top} := 8756ft$
Bottom of Wall Elevation:	$El_{bot} := 8745ft$
Wall Height:	$H_{wall} := El_{top} - El_{bot} = 11.00\,ft$
Bottom Wall Thickness:	$t_{wall.B} := 1ft + 6in$
Top Wall Thickness:	$t_{wall.T} := 1ft + 0in$
Footing width:	$B := 10.0ft$
Toe projection:	$Toe := 3.5ft$
Heel projection:	$Heel := B - Toe - t_{wall.B} = 5.00\,ft$
Footing thickness:	$t_{base} := 2.0ft$
Key depth below slab:	$d_{key} := 3.0ft$
Design Width:	$b_w := 12in$
Unit weight of concrete:	$\gamma_{conc} := 150 \cdot pcf$
Distance between Control Joints:	$L_{CJ} := 25ft$
Specified Compressive Strength of Concrete:	$f'_c := 4500psi$

Water Surface Elevations and Properties:

Elevation of normal water surface: (Assumed at base of structure)	$El_{nws} := El_{bot} - t_{base} + 0ft = 8743.00\,ft$
Water Unit Weight:	$\gamma_w := 62.4pcf$

Fill Elevations and Properties:

Reference:

Elevation of fill material beside channel: $El_{fill} := 8755ft$

Minimum elevation of fill material upstream of wall: $El_{fill.up} := El_{bot} = 8745.00\,ft$

Fill Properties (Soil type: SM)

Angle of internal friction: $\phi_f := 30 \cdot deg$

Unit weight: $\gamma_f := 130pcf$

Angle of inclined backfill: $\alpha_f := 0deg$

At-rest earth pressure coefficient: $k_0 := 1 - \sin(\phi_f) = 0.50$

Active earth pressure coefficient: $k_a := \tan(45deg - 0.5 \cdot \phi_f)^2 = 0.33$

Passive earth pressure coefficient: $k_p := \tan(45deg + 0.5 \cdot \phi_f)^2 = 3.00$

Friction factor for mass concrete on sound rock: $\delta_1 := 0.7$ *[NAVFAC p7.2-63]*

Seismic Properties:

10,000yr Horizontal Seismic Coefficient: $k_h := 0.4972$ [USGS Unified Hazard Tool]

Earth Pressure Coefficient: $K_{ae} := 0.42 \cdot k_h = 0.21$ *[Augusti: UCB GT 13-02: Eq. 4.3]*

Other Properties:

Construction Surcharge: $q_c := 200psf$

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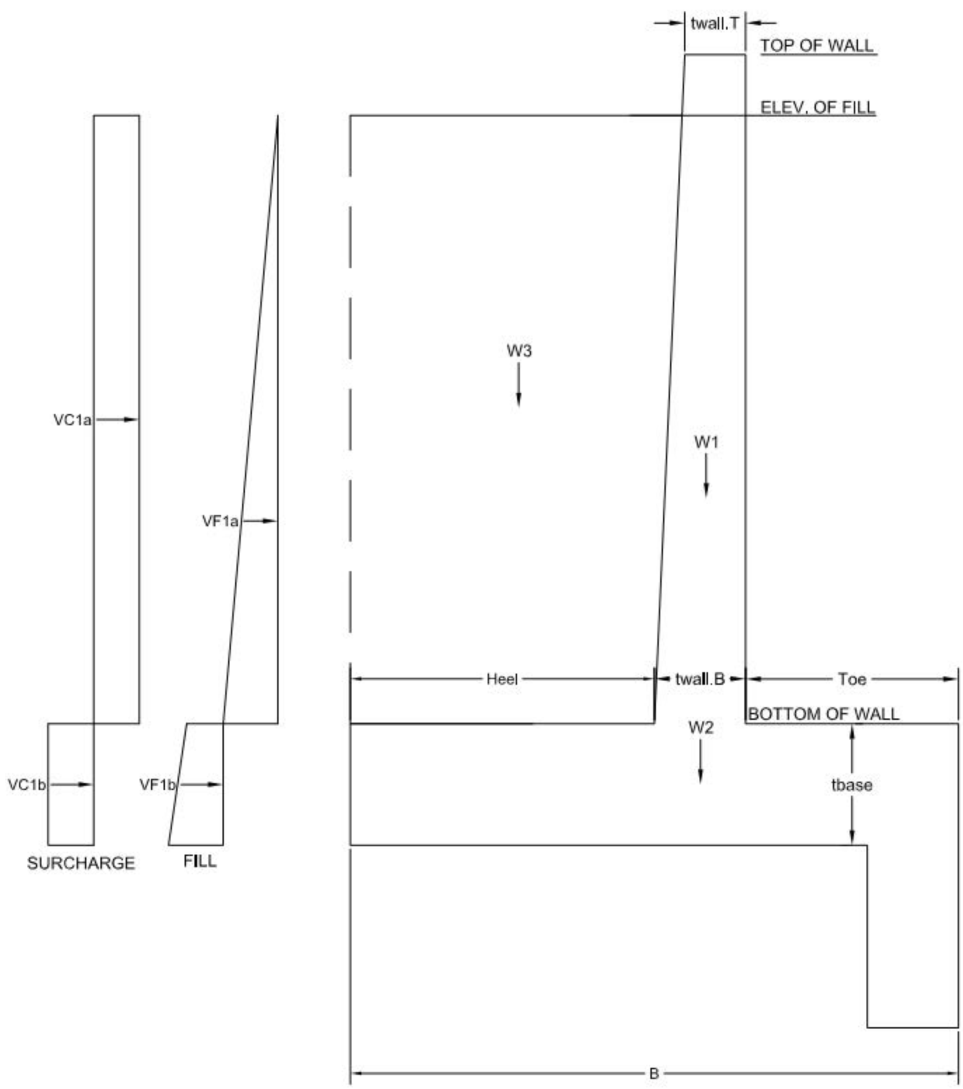
GEI Consultants	Project	Upper Black Creek Reservoir			Pg. Rev.	
	By	M. Provencher	Chk.	C. Diebold	App.	C. Masching
	Date	04/23/2020	Date	04/23/2020	Date	04/23/2020
Project No.	1801834	Document No.	N/A			
Subject	Design of Spillway - Head Wall					

1.) LOAD CASE 1 (Usual Condition) - Spillway Wall Loading:

Note: The retaining wall stem is analyzed as a cantilever beam with fixity provided at the footing. Beam is subject to lateral forces due to at-rest earth forces due to fill and construction live loads. Groundwater table is assumed below the wall section.



1.) LOAD CASE 1 (Usual Condition) - Spillway Wall Loading:

LOAD CASE 1-Applied Loads: Static

Active Earth Pressure

Base Shear:

$$V_{F1a} := 0.5 \cdot k_a \gamma_f \cdot \left(El_{fill} - El_{bot}\right)^2 b_w = 2.17 \cdot kip$$

Footing Shear:

$$V_{F1b} := \left[k_a \gamma_f \cdot \left(El_{fill} - El_{bot}\right) \cdot t_{base} + 0.5 \cdot k_a \cdot \gamma_f \cdot t_{base}^{\ 2}\right] \cdot b_w = 0.95 \cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_{F.1}(y) := \begin{cases} \left[0.5 \cdot k_a \cdot \gamma_f \cdot \left(El_{fill} - El_{bot} - y\right)^2 \cdot b_w\right] & \text{if } \ y < \left(El_{fill} - El_{bot}\right) \\ 0 & \text{otherwise} \end{cases}$$

$V_{F.1}\left(El_{nws} - El_{bot}\right) = 3.12 \cdot kip$ $\quad$ $V_{F.1}(0ft) = 2.17 \cdot kip$

Base Moment:

$$M_{F.1.b} := V_{F1a} \cdot \left[0.33\left(El_{fill} - El_{nws}\right) + \left(El_{nws} - El_{bot}\right)\right] = 4.25\, ft \cdot kip$$

Moment as at a Distance 'y' From Base of Wall:

$$M_{F.1}(y) := 0.5 \cdot k_a \cdot \gamma_f \cdot \left(El_{fill} - El_{bot} - y\right)^2 \cdot b_w 0.33\left(El_{fill} - El_{bot} - y\right)$$

$M_{F.1}\left(El_{nws} - El_{bot}\right) = 12.36 \cdot kip \cdot ft$ $\quad$ $M_{F.1}(0ft) = 7.2 \cdot kip \cdot ft$

Passive Earth Pressure

Resistance to bottom of slab:

$$F_{F.2} := 0.5 \cdot k_p \cdot \gamma_f \cdot \left(t_{base}\right)^2 \cdot 1ft = 0.78 \cdot kip$$

Resistance to bottom of key:

$$F_{F.T2} := 0.5 \cdot k_p \cdot \gamma_f \cdot \left(t_{base} + d_{key}\right)^2 \cdot 1ft = 4.87 \cdot kip$$

Vertical Forces

Weight of stem: $\quad W_1 := \gamma_{conc} \cdot \left(El_{top} - El_{bot}\right) \cdot 0.5\left(t_{wall.B} + t_{wall.T}\right) \cdot 1ft = 2.06 \cdot kip$

Weight of footing: $\quad W_2 := \gamma_{conc} \cdot B \cdot t_{base} \cdot 1ft = 3.00 \cdot kip$

Weight of fill over heel: $\quad W_3 := \gamma_f \cdot \left[Heel + 0.5 \cdot \left(t_{wall.B} - t_{wall.T}\right)\right] \cdot \left(El_{fill} - El_{bot}\right) \cdot 1ft = 6.83 \cdot kip$

1.) LOAD CASE 1 (Usual Condition) Cont'd

LOAD CASE 1-Applied Loads: Static

Construction Surcharge

Base Shear: $V_{C1a} := k_a \cdot q_c \cdot \left(El_{fill} - El_{bot}\right) \cdot b_w = 0.67 \cdot kip$

Shear as at a Distance 'y' From Base of Wall:

$$V_C(y) := \begin{cases} V_{C1a} \cdot \dfrac{\left(El_{fill} - El_{bot} - y\right)}{\left(El_{fill} - El_{bot}\right)} & \text{if} \quad y < \left(El_{fill} - El_{bot}\right) \\ 0 & \text{otherwise} \end{cases}$$

$V_C\left(El_{nws} - El_{bot}\right) = 0.80 \cdot kip$ $\qquad V_C(0ft) = 0.67 \cdot kip$

Footing Shear: $V_{C1b} := k_a \cdot q_c \cdot t_{base} \cdot b_w = 0.13 \cdot kip$

Base Moment: $M_C := 0.5 \cdot V_{C1a} \cdot \left(El_{fill} - El_{bot}\right) = 3.33\,ft \cdot kip$

Moment as at a Distance 'y' From Base of Wall: $M_C(y) := 0.5 \cdot V_C(y) \cdot \dfrac{\left(El_{fill} - El_{bot} - y\right)^2}{\left(El_{fill} - El_{bot}\right)}$

1.) LOAD CASE 1: (Usual Condition) Cont'd - Global Stability Checks

CHECK SLIDING STABILITY

Summation of vertical forces: $V_{sum} := W_1 + W_2 + W_3 = 11.89 \cdot kip$

Summation of horizontal driving forces:

$$H_{sum.1} := \max\left(V_{F1a} + V_{F1b} + V_{C1a} + V_{C1b} - F_{F.T2}, 0.01kip\right) = 0.01 \cdot kip$$

Factor of safety against sliding: $FS_{sl.1} := \dfrac{V_{sum} \cdot \delta_1}{H_{sum.1}} = 832.13$

Sliding stability check:
- EM 1110-2-2100, Critical Structure

$$Check_{sl.1} := \begin{vmatrix} \text{"LC1 Sliding Stability OK"} \quad \text{if} \quad FS_{sl.1} \geq 2.0 \\ \text{"STOP - LC1 Sliding Stability Unacceptable"} \quad \text{otherwise} \end{vmatrix}$$

$Check_{sl.1} =$ "LC1 Sliding Stability OK"

1.) LOAD CASE 1: (Usual Condition) Cont'd - Global Stability Checks

CHECK OVERTURNING STABILITY

Moment Calculations

Horizontal force moment arms:

$y_{1a} := \left(El_{fill} - El_{bot}\right) \div 3 + t_{base} = 5.33\,ft$

$y_{1b} := 0.5 \cdot t_{base} = 1.00\,ft$

$y_{c1a} := \left(El_{fill} - El_{bot}\right) \div 2 + t_{base} = 7.00\,ft$

$y_{c1b} := 0.5 \cdot t_{base} = 1.00\,ft$

$y_{FFT2} := 0.33\left(t_{base}\right) = 0.66\,ft$

Vertical force moment arms:

$x_1 := Toe + t_{wall.T} + 0.33 \cdot \left(t_{wall.B} - t_{wall.T}\right) = 4.66\,ft$

$x_2 := B \div 2 = 5.00\,ft$

$x_3 := B - Heel \div 2 = 7.50\,ft$

Summation of vertical forces:

$V_{sum} := W_1 + W_2 + W_3 = 11.89 \cdot kip$

Overturning Moment:

$M_{o.1} := V_{F1a} \cdot y_{1a} + V_{F1b} \cdot y_{1b} + V_{C1a} \cdot y_{c1a} \ldots + V_{C1b} \cdot y_{c1b} = 17.31 \cdot kip \cdot ft$

Resisting Moment:

$M_r := W_1 \cdot x_1 + W_2 \cdot x_2 + W_3 \cdot x_3 + F_{F.2} \cdot y_{FFT2} = 76.32 \cdot kip \cdot ft$

Location of resultant:

$$x_{bar.1} := \frac{M_r - M_{o.1}}{V_{sum}} = 4.96\,ft$$

Eccentricity of the resultant:

$$ecc_1 := \frac{B}{2} - x_{bar.1} = 0.04\,ft$$

Overturning check:

$$Check_{ot.1} := \begin{cases} \text{"LC1 Overturning Stability OK"} & \text{if } \frac{1}{3} \cdot B \le x_{bar.1} \le \frac{2}{3} B \\ \text{"STOP - LC1 Overturning Stability Unacceptable"} & \text{otherwise} \end{cases}$$

$Check_{ot.1} = \text{"LC1 Overturning Stability OK"}$ $\quad \frac{1}{3} \cdot B = 3.33\,ft \quad \frac{2}{3} \cdot B = 6.67\,ft$

1.) LOAD CASE 1 (Usual Condition) Cont'd

LOAD CASE 1-Loading Conditions & Combinations:

Unfactored Shear At Base of Wall: $V_{LC1}(y) := V_{F.1}(y) + V_C(y)$

Ultimate Shear At Base of Wall:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L) *[Table E-9, Load Case 1A]*

$$V_{uLC1.1}(y) := 2.2 \cdot \left(V_{F.1}(y) + V_C(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$V_{uLC1.2}(y) := 1.6 \cdot V_{F.1}(y)$$

ASCE Load Combination 2: 1.2 (D + F) + 1.6 (L) + 1.6 (H)

$$V_{uLC1.3}(y) := 1.6 \cdot V_C(y) + 1.6 \cdot V_{F.1}(y)$$

Controlling Case:

$$V_{uLC1}(y) := \max\left(V_{uLC1.1}(y), V_{uLC1.2}(y), V_{uLC1.3}(y)\right)$$

$$V_{uLC1}(0ft) = 6.23 \cdot kip \qquad \boxed{V_{uLC1.1}(0ft) = 6.23 \cdot kip}$$

Unfactored Moment At Base of Wall: $M_{LC1}(y) := M_{F.1}(y) + M_C(y)$

Ultimate Moment At Base of Wall:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L)

$$M_{uLC1.1}(y) := 2.2 \cdot \left(M_{F.1}(y) + M_C(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$M_{uLC1.2}(y) := 1.6 \cdot M_{F.1}(y)$$

ASCE Load Combination 2: 1.2 (D + F) + 1.6 (L) + 1.6 (H)

$$M_{uLC1.3}(y) := 1.6 \cdot M_C(y) + 1.6 \cdot M_{F.1}(y)$$

Controlling Case:

$$M_{uLC1}(y) := \max\left(M_{uLC1.1}(y), M_{uLC1.2}(y), M_{uLC1.3}(y)\right)$$

$$M_{uLC1}(0ft) = 23.06 \cdot kip \cdot ft \qquad \boxed{M_{uLC1.1}(0ft) = 23.06\, ft \cdot kip}$$

1.) LOAD CASE 1 (Usual Condition) Cont'd

Environmental Durability Factor (ACI 350-06)

Note: Required for Tension Controlled structure to encourage durability and liquid-tightness.
Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads. The durability factor is applied to service loads only.

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4 $f_{s.ss} := 24000psi$ $S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$

Flexure in normal conditions: R10.6.4
- One way element, Bar at 6" spacing

$f_{s.f} := 34000psi$ $S_{d.f} := \max\left(\frac{f_y}{f_{s.f}}, 1.0\right) = 1.76$

Shear in Wall:

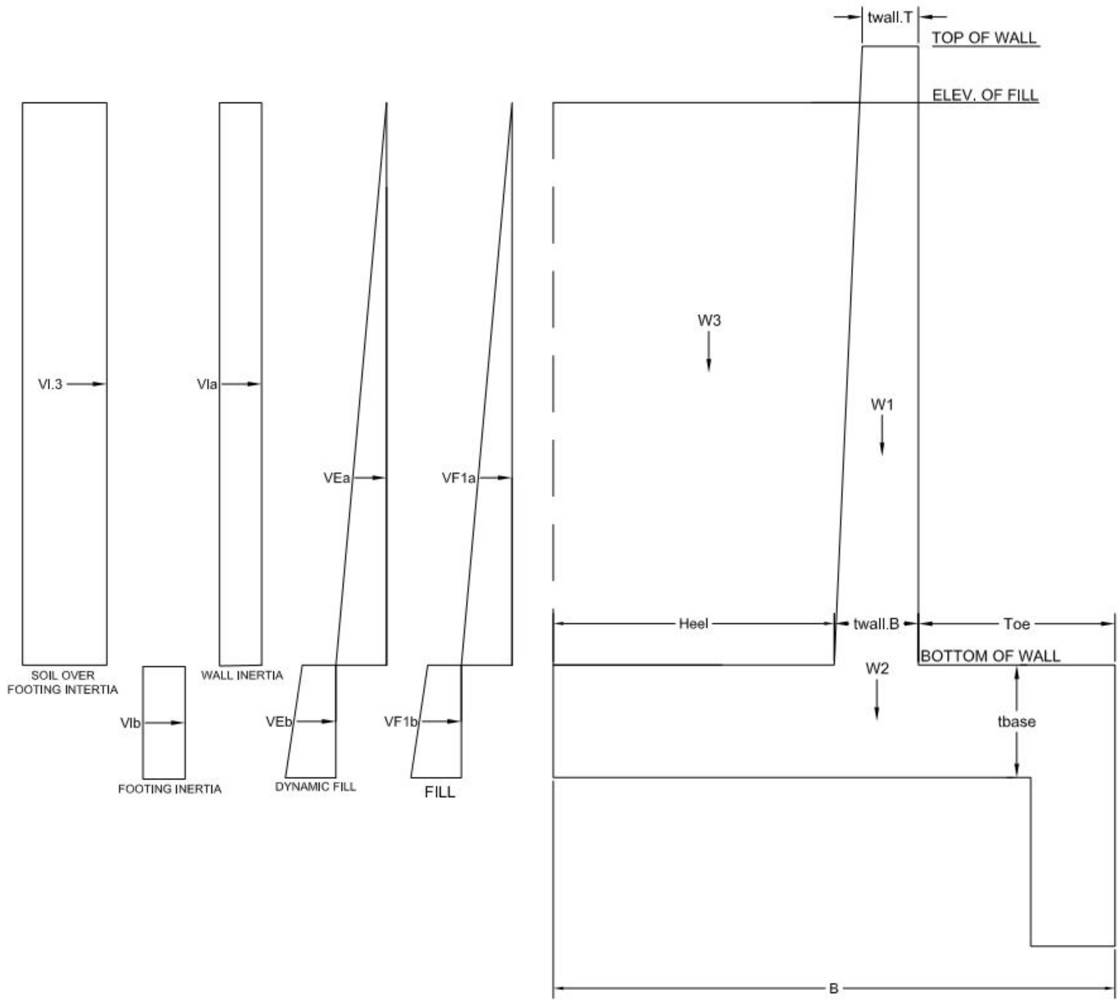
$\boxed{V_{LC1.ED}(y) := S_{d.ss} \cdot \left(V_{F.1}(y) + V_C(y)\right)}$ $\boxed{V_{LC1.ED}(0ft) = 7 \cdot kip}$ > $\boxed{V_{uLC1.1}(0ft) = 6 \cdot kip}$

Moment in Wall:

$\boxed{M_{LC1.ED}(y) := S_{d.f} \cdot \left(M_{F.1}(y) + M_C(y)\right)}$ $\boxed{M_{LC1.ED}(0ft) = 19 \cdot kip \cdot ft}$ < $\boxed{M_{uLC1.1}(0ft) = 23 \cdot kip \cdot ft}$

2.) LOAD CASE 2 (Extreme Condition) - Spillway Wall Loading:

Note: The retaining wall stem is analyzed as a cantilever beam with fixity provided at the footing. Beam is subject to lateral forces due to seismic inertial forces due to self-weight and dynamic fill loads. Groundwater table is assumed below the wall section.



2.) LOAD CASE 2 (Extreme Condition) - Spillway Wall Loading:

LOAD CASE 2-Applied Loads: Seismic

Seismic Incremental Force of Earth Pressure

Base Shear:

$$V_{Ea} := 0.5K_{ae}\cdot b_w\cdot\left[\gamma_f\cdot\left(El_{fill}-El_{bot}\right)^2\right] = 1.36\cdot kip$$

$$V_{Eb} := \left[K_{ae}\gamma_f\cdot\left(El_{fill}-El_{bot}\right)\cdot t_{base} + 0.5\cdot K_{ae}\cdot\gamma_f\cdot t_{base}{}^2\right]\cdot b_w = 0.60\cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_E(y) := 0.5\cdot K_{ae}\cdot\gamma_f\cdot\left(El_{fill}-El_{bot}-y\right)^2\cdot b_w \qquad V_E(0ft) = 1.4\cdot kip$$

Base Moment:

$$M_E := V_{Ea}\cdot\left[0.33\left(El_{fill}-El_{bot}\right)\right] = 4.48\,ft\cdot kip$$

Moment at a Distance 'y' From Base of Wall:

$$M_E(y) := 0.5\cdot K_{ae}\cdot\gamma_f\cdot\left(El_{fill}-El_{bot}-y\right)^2\cdot b_w 0.33\left(El_{fill}-El_{bot}-y\right)$$

$$M_E(0ft) = 4.5\cdot kip\cdot ft$$

Inertial Forces

Wall Base Shear:

$$V_{Ia} := k_h\cdot W_1 = 1.03\cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_I(y) := k_h\cdot\gamma_{conc}\cdot\left[t_{wall.T}+\frac{0.5\cdot\left(t_{wall.B}-t_{wall.T}\right)\cdot\left(H_{wall}-y\right)}{\left(H_{wall}\right)}\right]\cdot\left(H_{wall}-y\right)\cdot b_w$$

Base Moment:

$$M_I := V_{Ia}\cdot\left[\frac{H_{wall}\cdot\left(2t_{wall.T}+t_{wall.B}\right)}{3\left(t_{wall.B}+t_{wall.T}\right)}\right] = 5.26\cdot kip\cdot ft$$

Moment at a Distance 'y' From Base of Wall:

$$M_I(y) := V_I(y)\cdot\frac{\left(H_{wall}-y\right)\cdot\left[2t_{wall.T}+\left[t_{wall.T}+\frac{\left(t_{wall.B}-t_{wall.T}\right)\cdot\left(H_{wall}-y\right)}{H_{wall}}\right]\right]}{3\left[t_{wall.T}+\frac{\left(t_{wall.B}-t_{wall.T}\right)\cdot\left(H_{wall}-y\right)}{H_{wall}}+t_{wall.T}\right]}$$

Footing Shear:

$$V_{Ib} := k_h\cdot W_2 = 1.49\cdot kip$$

Soil over Footing Shear:

$$V_{I.3} := k_h\cdot W_3 = 3.39\cdot kip$$

2.) LOAD CASE 2 (Extreme Condition) Cont'd -Global Stability Checks

CHECK SLIDING STABILITY

Summation of vertical forces: $V_{sum} := W_1 + W_2 + W_3 = 11.89 \cdot kip$

Summation of horizontal driving forces: $H_{sum.2} := V_{F1a} + V_{F1b} + V_{Ea} + V_{Eb} \cdots + V_{Ia} + V_{Ib} + V_{I.3} - F_{F.T2} = 6.11 \cdot kip$

Factor of safety against sliding:

$$FS_{sl.2} := \frac{V_{sum} \cdot \delta_1}{H_{sum.2}} = 1.36$$

Sliding stability check:
EM 1110-2-2100:
- Critical Structure,
- Extreme Seismic Case not based on detailed site-specific data

$$Check_{sl.2} := \begin{cases} \text{"LC2 Sliding Stability OK"} & \text{if } FS_{sl.2} \geq 1.3 \\ \text{"STOP - LC2 Sliding Stability Unacceptable"} & \text{otherwise} \end{cases}$$

$Check_{sl.2} = \text{"LC2 Sliding Stability OK"}$

2.) LOAD CASE 2 (Extreme Condition) Cont'd -Global Stability Checks

CHECK OVERTURNING STABILITY

Moment Calculations

Horizontal force moment arms:

$$y_{Ia} := \left(El_{fill} - El_{bot}\right) \div 2 + t_{base} = 7.00\ ft$$

$$y_{Ib} := 0.5 \cdot t_{base} = 1.00\ ft$$

$$y_{I3} := y_{Ia} = 7.00\ ft$$

[Augusti: UCB GT 13-02]

$$y_{ea} := 0.4\left(El_{fill} - El_{bot}\right) + t_{base} = 6.00\ ft$$

$$y_{eb} := 0.4 \cdot t_{base} = 0.80\ ft$$

Summation of vertical forces:

$$V_{sum} := W_1 + W_2 + W_3 = 11.89 \cdot kip$$

Overturning Moment:

$$M_{o.2} := V_{F1a} \cdot y_{1a} + V_{F1b} \cdot y_{1b} \cdots + V_{Ea} \cdot y_{ea} + V_{Eb} \cdot y_{eb} \cdots + V_{Ia} \cdot y_{Ia} + V_{Ib} \cdot y_{Ib} + V_{I.3} \cdot y_{I3} = 53.55 \cdot kip \cdot ft$$

Resisting Moment:

$$M_r := W_1 \cdot x_1 + W_2 \cdot x_2 + W_3 \cdot x_3 + F_{F.2} \cdot y_{FFT2} = 76.32 \cdot kip \cdot ft$$

Location of resultant:

$$x_{bar.2} := \frac{M_r - M_{o.2}}{V_{sum}} = 1.92\ ft$$

Eccentricity of the resultant:

$$ecc_2 := \frac{B}{2} - x_{bar.2} = 3.08\ ft$$

Overturning check:

$$Check_{ot.2} := \begin{cases} \text{"LC2 Overturning Stability OK"} & \text{if } 0 \le x_{bar.2} \le B \\ \text{"STOP - LC2 Overturning Stability Unacceptable"} & \text{otherwise} \end{cases}$$

$Check_{ot.2}$ = "LC2 Overturning Stability OK"

$B = 10.00\ ft$

2.) LOAD CASE 2 (Extreme Condition) Cont'd

LOAD CASE 2-Loading Conditions & Combinations:

Unfactored Shear At Base of Wall: $V_{LC2}(y) := V_{F.1}(y) + V_E(y) + V_I(y)$

Ultimate Shear At Base of Wall:

EM 1110-2-2104 Strength Load Combination: 1.0 EH + 1.0 Hs + 1.0 EQ *[Table E-9, Load Case 1A]*

$$V_{uLC2.1}(y) := V_{F.1}(y) + V_E(y) + V_I(y)$$

ASCE Load Combination 5: 1.2 (D + F) + 1.0 E + 1.6 H

$$V_{uLC2.2}(y) := \left(V_E(y) + V_I(y)\right) + 1.6 \cdot V_{F.1}(y)$$

Controlling Case: $V_{uLC2}(y) := \max\left(V_{uLC2.1}(y), V_{uLC2.2}(y)\right)$

$$V_{uLC2}(0ft) = 5.85 \cdot kip \qquad \boxed{V_{uLC2.2}(0ft) = 5.85 \cdot kip}$$

Unfactored Moment At Base of Wall: $M_{LC3}(y) := M_{F.1}(y) + M_E(y) + M_I(y)$

Ultimate Moment At Base of Wall:

EM 1110-2-2014 Strength Load Combination: 1.0 EH + 1.0 Hs + 1.0 (EQ)

$$M_{uLC2.1}(y) := M_{F.1}(y) + M_E(y) + M_I(y)$$

ASCE Load Combination 5: 1.2 (D + F) + 1.0 E + 1.6 H

$$M_{uLC2.2}(y) := \left(M_E(y) + M_I(y)\right) + 1.6 \cdot M_{F.1}(y)$$

Controlling Case: $M_{uLC2}(y) := \max\left(M_{uLC2.1}(y), M_{uLC2.2}(y)\right)$

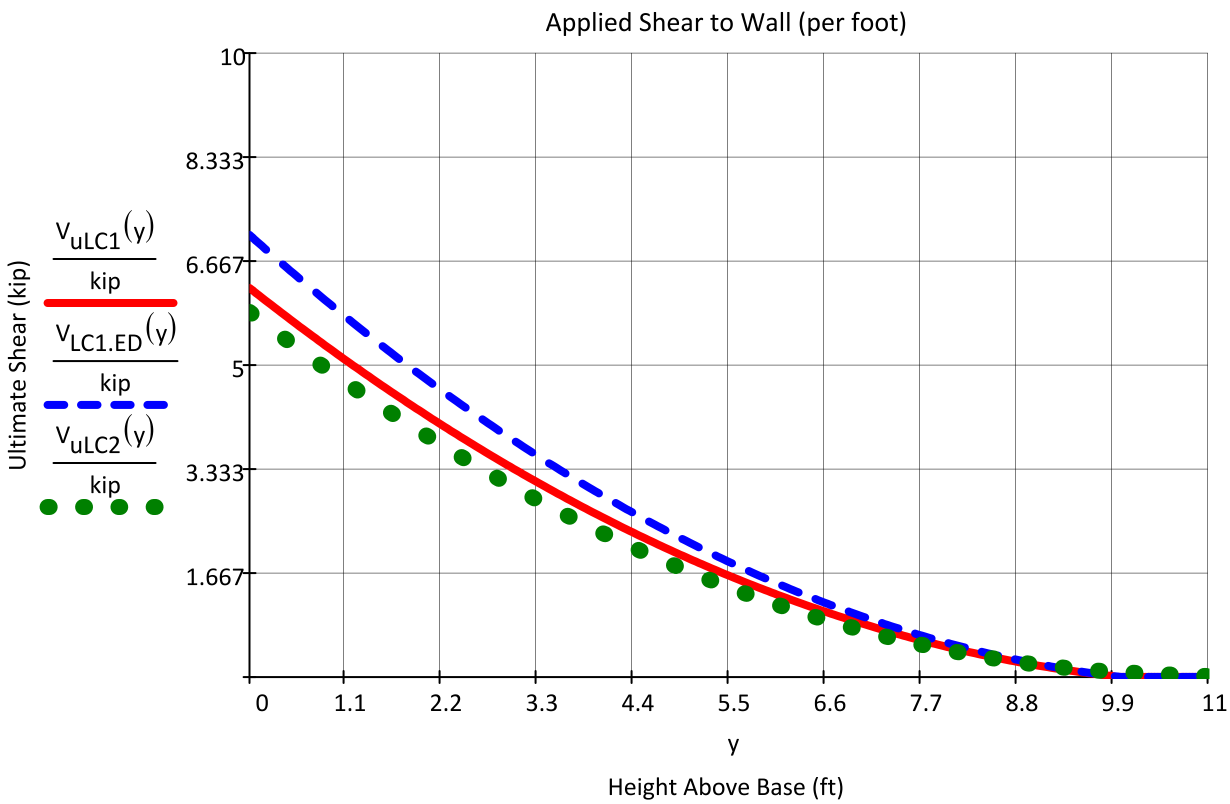
$$M_{uLC2}(0ft) = 21.18 \cdot kip \cdot ft \qquad \boxed{M_{uLC2.2}(0ft) = 21.18\,ft \cdot kip}$$

3.) Load Case Summary Analysis

Controlling Applied Load:

Ultimate Shear at Base of Wall: $V_u(y) := \max\left(V_{uLC1}(y), V_{LC1.ED}(y), V_{uLC2}(y)\right)$

$V_u(0ft) = 7.08 \cdot kip$ $\quad V_{LC1.ED}(0ft) = 7.08 \cdot kip$



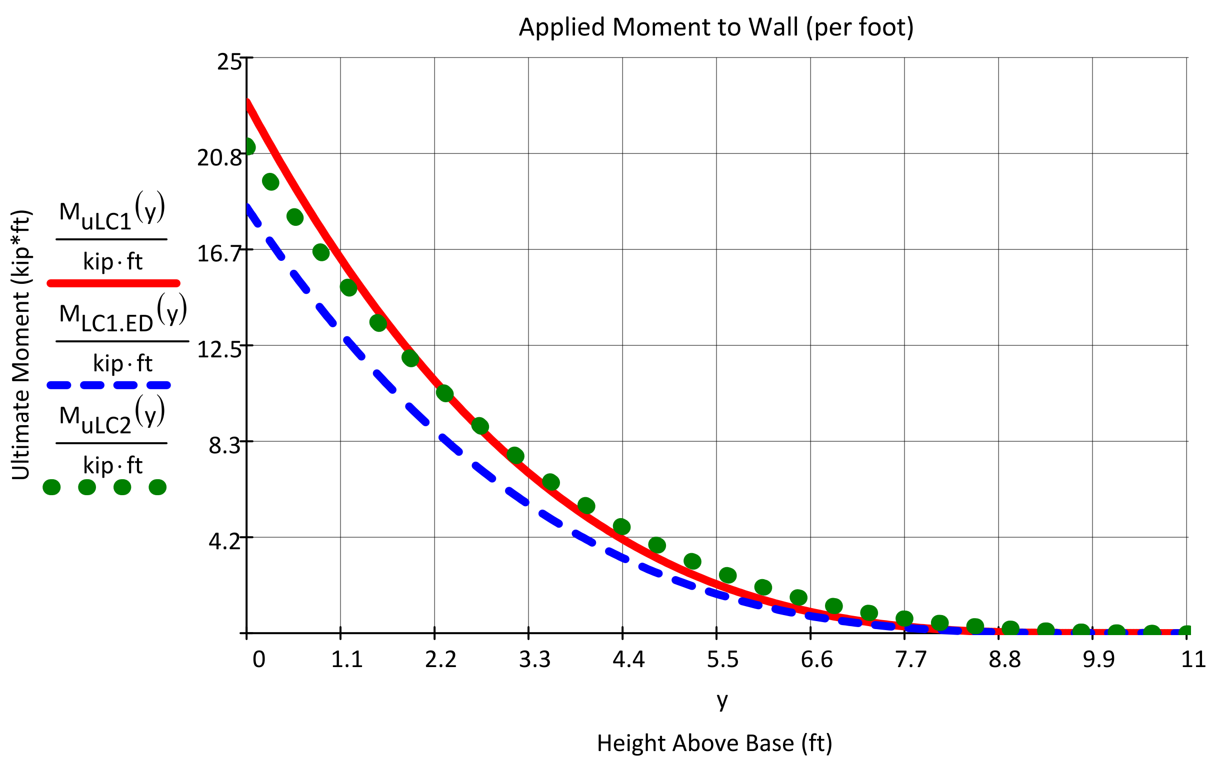
3.) Load Case Summary Analysis Cont'd

Controlling Applied Load:

Ultimate Moment at Base of Wall:

$$M_u(y) := \max\left(M_{uLC1}(y), M_{LC1.ED}(y), M_{uLC2}(y)\right)$$

$$M_u(0ft) = 23.06 \cdot kip \cdot ft \qquad M_{LC1.ED}(0ft) = 18.50 \cdot kip \cdot ft$$



4.) Wall Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05 \cdot \dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Horizontal Reinforcement:

Bar Size and Spacing: (Channel & Embankment) $Size_{SH} := 5$ $sSH := 12in$

Reinforcement Clear Cover: $clrS := 3in$

Diameter & Cross-Sectional Area of Bars: $dbSH := d_{b_{Size_{SH}}} = 0.63 \cdot in$ $AbSH := A_{b_{Size_{SH}}} = 0.31 \cdot in^2$

Area of Reinforcement: $A_{sSH} := AbSH \cdot \dfrac{12in}{sSH} = 0.31 \cdot in^2$

$m := \left(t_{wall.T} - t_{wall.B}\right) \div H_{wall} = -0.05$

Section Depth at a Height 'y' from the base: $t_{wall}(y) := t_{wall.B} + m \cdot y$

Depth to Centroid of Embankment Reinf: $d_{SH}(y) := t_{wall}(y) - clrS - 0.5 \cdot dbSH$

Reinforcement Ratio: $\rho_{SH}(y) := A_{sSH} \div \left(b_w\, d_{SH}(y)\right)$

4.) Wall Concrete & Rebar Strength Design Cont'd

Vertical Reinforcement:

Bar Size and Spacing: (Embankment) $Size_{SE} := 7$ $sSE := 12in$

Diameter & Cross-Sectional Area of Bars: $dbSE := d_{b_{Size_{SE}}} = 0.88 \cdot in$ $AbSE := A_{b_{Size_{SE}}} = 0.60 \cdot in^2$

Area of Tension Reinforcement: $A_{sSE} := AbSE \cdot \frac{12in}{sSE} = 0.60 \cdot in^2$

$m := \left(t_{wall.T} - t_{wall.B}\right) \div H_{wall} = -0.05$

Section Depth at a Height 'y' from the base: $t_{wall}(y) := t_{wall.B} + m \cdot y$

Depth to Centroid of Reinforcement: $d_{SE}(y) := t_{wall}(y) - clrS - dbSH - 0.5 \cdot dbSE$

Reinforcement Ratio: $\rho_{SE}(y) := A_{sSE} \div \left(b_w d_{SE}(y)\right)$

Bar Size and Spacing: (Channel) $Size_{SC} := 7$ $sSC := 12in$

Diameter & Cross-Sectional Area of Bars: $dbSC := d_{b_{Size_{SC}}} = 0.88 \cdot in$ $AbSC := A_{b_{Size_{SC}}} = 0.60 \cdot in^2$

Area of Tension Reinforcement: $A_{sSC} := AbSC \cdot \frac{12in}{sSC} = 0.60 \cdot in^2$

Depth to Centroid of Reinforcement: $d_{SC}(y) := t_{wall}(y) - clrS - dbSH - 0.5 \cdot dbSC$

Reinforcement Ratio: $\rho_{SC}(y) := A_{sSC} \div \left(b_w d_{SC}(y)\right)$

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GEI Consultants	Project	Upper Black Creek Reservoir			Pg. Rev.	
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	Date	04/23/2020	Date	04/23/2020	Date	04/23/2020
Project No.	1801834	Document No.	N/A			
Subject	Design of Spillway - Head Wall					

5.) Wall Strength Capacity

Wall Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
ACI 318-14, §22.5.5.1)

$$V_c(y) := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot \min\left(d_{SE}(y), d_{SC}(y)\right)$$

$$V_c(0ft) = 22.44 \cdot kip$$

Design Shear Capacity:

$$\phi V_n(y) := \phi_v \cdot \left(V_c(y)\right)$$

$$\phi V_n(0ft) = 16.83 \cdot kip$$

$$V_u(0ft) = 7.08 \cdot kip$$

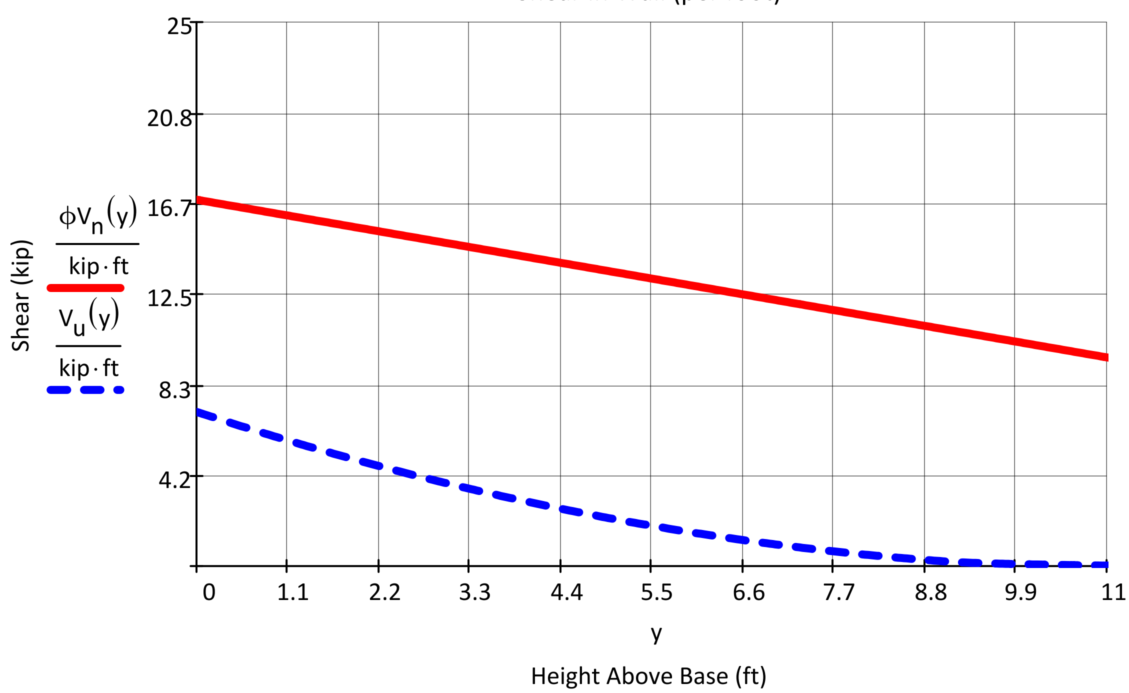
Demand Capacity Ratio:

$$DCR_V(y) := \frac{V_u(y)}{\phi V_n(y)}$$

$$DCR_V(0ft) = 0.42$$

< 1.0 therefore section is okay for shear strength

Shear in Wall (per foot)



5.) Wall Strength Capacity Cont'd

Wall Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(ACI 318-14)

$$M_n(y) := \rho_{SE}(y)\cdot f_y\cdot b_w\cdot d_{SE}(y)^2\cdot\left(1 - 0.59\cdot\rho_{SE}(y)\cdot\frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n(y) := \phi_t\cdot M_n(y)$$

$$\phi M_n(0) = 36.57\,ft\cdot kip$$

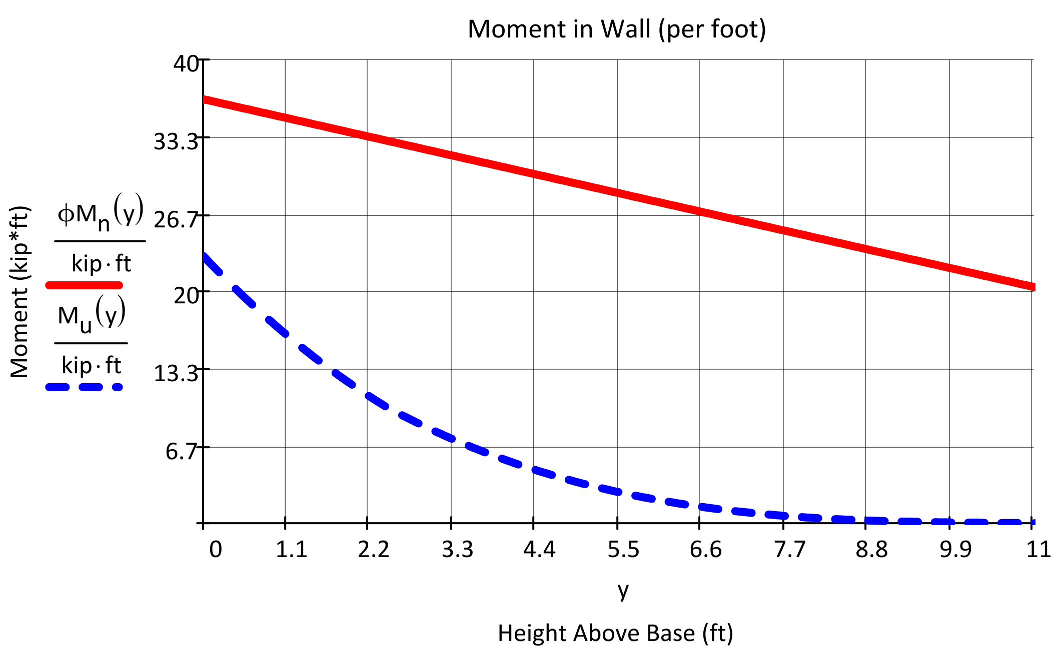
$$M_u(0) = 23.06\cdot kip\cdot ft$$

Demand Capacity Ratio:

$$DCR_M(y) := \frac{M_u(y)}{\phi M_n(y)}$$

$$DCR_M(0ft) = 0.63$$

< 1.0 therefore section is okay for flexural strength



6.) Wall Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in, 5\cdot 0.5\left(t_{wall.B} + t_{wall.T}\right), 12in\right] = 12.00\cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{cases} 0.0020 & \text{if } f_y < 60ksi \\ \max\left[\left(0.0018\cdot 60ksi \div f_y\right), 0.0014\right] & \text{otherwise} \end{cases} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0030$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0030$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min} := \rho_{max}\cdot b_w\cdot 0.5\cdot\left(t_{wall.B} + t_{wall.T}\right) = 0.54\cdot in^2$$

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Vertical Reinforcement:

Bar Size and Spacing of Reinforcement:
(Embankment)

$Size_{SE} = 7.00$ $sSE = 12.00\cdot in$

Diameter & Cross-Sectional Area of Bars:

$dbSE = 0.88\cdot in$ $AbSE = 0.60\cdot in^2$

Area of Reinforcement:

$A_{sSE} = 0.60\cdot in^2$

Bar Size and Spacing of Reinforcement:
(Channel)

$Size_{SC} = 7.00$ $sSC = 12.00\cdot in$

Diameter & Cross-Sectional Area of Bars:

$dbSC = 0.88\cdot in$ $AbSC = 0.60\cdot in^2$

Area of Reinforcement:

$A_{sSC} = 0.60\cdot in^2$

$$\boxed{\frac{A_{s.min}}{A_{sSC} + A_{sSE}} = 0.45}$$

< 1.0 therefore okay

6.) Wall Rebar Design Cont'd

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Horizontal Reinforcement:

Wall is reinforced with two sections. The top 20-feet from the top of the wall use reinforcing at half the spacing of that after 20-feet.

Bar Size and Spacing of Reinforcement: (Embankment & Channel) $Size_{SH} = 5.00$ $sSH = 12.00 \cdot in$

Diameter & Cross-Sectional Area of Bars: $dbSH = 0.63 \cdot in$ $AbSH = 0.31 \cdot in^2$

Area of Reinforcement: $A_{sSH} = 0.31 \cdot in^2$

$$\boxed{\frac{A_{s.min}}{2 \cdot A_{sSH}} = 0.87}$$ < 1.0 therefore okay

Minimum Flexural Steel: (ACI 318-14, §9.6.1.2)

Vertical Reinforcement:

$$A_{sV.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot \min\left(d_{SE}(0ft), d_{SC}(0ft)\right), \frac{200 \cdot psi \cdot b_w \cdot \min\left(d_{SE}(0ft), d_{SC}(0ft)\right)}{f_y}\right) = 0.56 \cdot in^2$$

$$\boxed{\frac{A_{sV.min}}{A_{sSE}} = 0.93}$$ < 1.0 therefore okay

6.) Wall Rebar Design Cont'd

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Reinforcement Ratio Provided: (Vertical)

Embankment: $$\rho_{pVE} := \frac{A_{sSE}}{d_{SE}(0ft) \cdot b_w} = 0.0036$$

Channel: $$\rho_{pVC} := \frac{A_{sSE}}{d_{SE}(0ft) \cdot b_w} = 0.0036$$

Reinforcement Ratio Provided: (Horizontal)

$$\rho_{pH} := \frac{A_{sSH}}{d_{SH}(0ft) \cdot b_w} = 0.0018$$

Depth to Neutral Axis: (Vertical)

$$cVE := \frac{\rho_{pVE} \cdot f_y \cdot d_{SE}(0ft)}{0.85 \cdot \beta_1 \cdot f'_c} = 0.95 \cdot in$$

$$cVC := \frac{\rho_{pVC} \cdot f_y \cdot d_{SC}(0ft)}{0.85 \cdot \beta_1 \cdot f'_c} = 0.95 \cdot in$$

Depth to Neutral Axis: (Horizontal)

$$cH := \frac{\rho_{pH} \cdot f_y \cdot d_{SH}(0ft)}{0.85 \cdot \beta_1 \cdot f'_c} = 0.49 \cdot in$$

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Calculated Strain in Tension Steel at Nominal Strength: (Vertical)

$$\varepsilon_{tVE} := 0.003 \cdot \left(\frac{d_{SE}(0ft)}{cVE} - 1\right) = 0.04 \qquad \varepsilon_{tVC} := 0.003 \cdot \left(\frac{d_{SC}(0ft)}{cVC} - 1\right) = 0.04$$

(Horizontal)

$$\varepsilon_{tH} := 0.003 \cdot \left(\frac{d_{SH}(0ft)}{cH} - 1\right) = 0.09$$

$$\boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tVE}} = 0.10} \quad \boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tVC}} = 0.10} \quad \boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tH}} = 0.05}$$

< 1.0 therefore okay

7.) Footing Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1)*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05 \cdot \dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Reinforcement Parallel to Wall:

Heel Reinforcement:

Bar Size and Spacing (Top): $Size_{HTP} := 7$ $sHTP := 12in$

Diameter & Cross-Sectional Area of Bars:

$dbHTP := d_{b_{Size_{HTP}}} = 0.88 \cdot in$

$AbHTP := A_{b_{Size_{HTP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sHTP} := AbHTP \cdot \dfrac{12in}{sHTP} = 0.60 \cdot in^2$

Depth to Centroid of Tension Reinforcement: $d_{HTP} := t_{base} - 0.5 \cdot dbHTP = 1.96\,ft$

Reinforcement Ratio: $\rho_{HTP} := A_{sHTP} \div \left(b_w\, d_{HTP}\right) = 0.00212$

Bar Size and Spacing (Bottom): $Size_{HBP} := 7$ $sHBP := 12in$

Diameter & Cross-Sectional Area of Bars:

$dbHBP := d_{b_{Size_{HBP}}} = 0.88 \cdot in$

$AbHBP := A_{b_{Size_{HBP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sHBP} := AbHBP \cdot \dfrac{12in}{sHBP} = 0.60 \cdot in^2$

7.) Footing Concrete & Rebar Strength Design Cont'd

Reinforcement Parallel to Wall:

Toe Reinforcement:

Bar Size and Spacing (Top): $Size_{TTP} := 7$ $sTTP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbTTP := d_{b_{Size_{TTP}}} = 0.88 \cdot in$

$AbTTP := A_{b_{Size_{TTP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sTTP} := AbTTP \cdot \frac{12in}{sTTP} = 0.60 \cdot in^2$

Bar Size and Spacing (Bottom): $Size_{TBP} := 7$ $sTBP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbTBP := d_{b_{Size_{TBP}}} = 0.88 \cdot in$

$AbTBP := A_{b_{Size_{TBP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sTBP} := AbTBP \cdot \frac{12in}{sTBP}$

Reinforcement Perpendicular to Wall:

Heel Reinforcement:

Bar Size and Spacing (Top): $Size_{HT} := 9$ $sHT := 12in$

Reinforcement Clear Cover: $clrHT := 3in$

Diameter & Cross-Sectional Area of Bars: $dbHT := d_{b_{Size_{HT}}} = 1.13 \cdot in$ $AbHT := A_{b_{Size_{HT}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sHT} := AbHT \cdot \frac{12in}{sHT} = 1.00 \cdot in^2$

Section Depth: $t_{foot} := t_{base} = 2.00\,ft$

Depth to Centroid of Tension Reinforcement: $d_{HT} := t_{foot} - clrHT - (0.5 \cdot dbHT) = 1.70\,ft$

Reinforcement Ratio: $\rho_{HT} := A_{sHT} \div \left(b_w\, d_{HT}\right) = 0.00408$

7.) Footing Concrete & Rebar Strength Design Cont'd

Reinforcement Perpendicular to Wall:

Heel Reinforcement:

Bar Size and Spacing (Bottom): $Size_{HB} := 9$ $sHB := 12in$

Reinforcement Clear Cover: $clrHB := 4in$

Diameter & Cross-Sectional Area of Bars: $dbHB := d_{b_{Size_{HB}}} = 1.13 \cdot in$ $AbHB := A_{b_{Size_{HB}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sHB} := AbHB \cdot \frac{12in}{sHB} = 1.00 \cdot in^2$

Depth to Centroid of Compression Reinforcement: $d_{HB} := t_{foot} - clrHB - 0.5 \cdot dbHB = 1.62\,ft$

Toe Reinforcement:

Bar Size and Spacing (Top): $Size_{TT} := 9$ $sTT := 12in$

Reinforcement Clear Cover: $clrTT := 3in$

Diameter & Cross-Sectional Area of Bars: $dbTT := d_{b_{Size_{TT}}} = 1.13 \cdot in$ $AbTT := A_{b_{Size_{TT}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sTT} := AbTT \cdot \frac{12in}{sTT} = 1.00 \cdot in^2$

Depth to Centroid of Tension Reinforcement: $d_{TT} := t_{foot} - clrTT - (0.5 \cdot dbTT) = 1.70\,ft$

Reinforcement Ratio: $\rho_{TT} := A_{sTT} \div \left(b_w d_{TT}\right) = 0.00408$

Bar Size and Spacing (Bottom): $Size_{TB} := 9$ $sTB := 12in$

Reinforcement Clear Cover: $clrTB := 4in$

Diameter & Cross-Sectional Area of Bars: $dbTB := d_{b_{Size_{TB}}} = 1.13 \cdot in$ $AbTB := A_{b_{Size_{TB}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sTB} := AbTB \cdot \frac{12in}{sTB} = 1.00 \cdot in^2$

Depth to Centroid of Compression Reinforcement: $d_{TB} := t_{foot} - clrTB - 0.5 \cdot dbTB = 1.62\,ft$

Reinforcement Ratio: $\rho_{TB} := A_{sTB} \div \left(b_w d_{TB}\right) = 0.00429$

8.) Footing Strength Capacity

Bearing Pressure:

Maximum Eccentricity: $ecc := \max(ecc_1, ecc_2) = 3.08\,ft$

Unfactored Shear: $V := (W_1 + W_2 + W_3) = 11.89 \cdot kip$

Factored Shear:
[ASCE 7-10, 2.3.2 Eqtn 2:
DL & pressure of bulk materials factor]

$$V_u := 1.2 \cdot (W_1 + W_2) + 1.6 \cdot W_3 = 17.00 \cdot kip$$

Unfactored bearing pressure at heel:

$$q_{heel.ED} := \frac{V}{B \cdot b_w} \cdot \left[1 - \frac{(6 \cdot ecc_1)}{B}\right] = 1.16 \cdot \frac{kip}{ft^2}$$

Unfactored bearing pressure at toe:

$$q_{toe.ED} := \frac{V}{B \cdot b_w} \cdot \left[1 + \frac{(6 \cdot ecc_1)}{B}\right] = 1.21 \cdot \frac{kip}{ft^2}$$

Factored bearing pressure at heel:

$$q_{u.heel} := \frac{V_u}{B \cdot b_w} \cdot \left[1 - \frac{(6 \cdot ecc)}{B}\right] = -1.45 \cdot \frac{kip}{ft^2}$$

Factored bearing pressure at toe:

$$q_{u.toe} := \frac{V_u}{B \cdot b_w} \cdot \left[1 + \frac{(6 \cdot ecc)}{B}\right] = 4.84 \cdot \frac{kip}{ft^2}$$

$$q_{u.max} := \max(q_{u.heel}, q_{u.toe}) = 4.84 \cdot ksf$$

8.) Footing Strength Capacity Cont'd

Note: The heel slab is designed as a cantilever beam with downward pressure from soil + slab and upward pressure from bearing. The toe slab is designed as a cantilever beam with downward pressure from slab and upward pressure from bearing.

Net Forces on Footings:

Downward weight of heel slab: (unfactored)

$$W_{heel} := \gamma_{conc} \cdot t_{base} \cdot 1ft = 0.30 \cdot \frac{kip}{ft}$$

Downward weight of toe slab: (unfactored)

$$W_{toe} := \gamma_{conc} \cdot t_{base} \cdot 1ft = 0.30 \cdot \frac{kip}{ft}$$

Downward weight of fill over heel slab: (unfactored)

$$W_{heelfill} := \gamma_f \cdot \left(El_{fill} - El_{bot}\right) \cdot 1ft = 1.30 \cdot \frac{kip}{ft}$$

Downward weight of heel slab:

$$W_{u.heel} := 1.2W_{heel} = 0.36 \cdot \frac{kip}{ft}$$

Downward weight of toe slab:

$$W_{u.toe} := 1.2W_{toe} = 0.36 \cdot \frac{kip}{ft}$$

Downward weight of fill over heel slab:

$$W_{u.heelfill} := 1.6W_{heelfill} = 2.08 \cdot \frac{kip}{ft}$$

Net downward force on heel (Unfactored-Load Case 1):

$$p_{h.ED1} := W_{heel} + W_{heelfill} - b_w \cdot \left[q_{heel.ED} + 0.5 \cdot \left(\frac{q_{toe.ED} - q_{heel.ED}}{B}\right) \cdot \left(Heel - \frac{d_{HT}}{2}\right)\right] = 0.43 \cdot \frac{kip}{ft}$$

Tension on Top

Net downward force on toe (Unfactored-Load Case 1):

$$p_{h.ED2} := W_{toe} - b_w \cdot \left[q_{toe.ED} + 0.5 \cdot \left(\frac{q_{heel.ED} - q_{toe.ED}}{B}\right) \cdot \left(Toe - \frac{d_{TT}}{2}\right)\right] = -0.91 \cdot \frac{kip}{ft}$$

Tension on Bottom

Net downward force on heel (Factored-Max):

$$p_h := W_{u.heel} + W_{u.heelfill} - b_w \cdot \left[q_{u.heel} + 0.5 \cdot \left(\frac{q_{u.toe} - q_{u.heel}}{B}\right) \cdot \left(Heel - \frac{d_{HT}}{2}\right)\right] = 2.58 \cdot \frac{kip}{ft}$$

Tension on Top

Net downward force on toe (Factored-Max):

$$p_{h2} := W_{u.toe} - b_w \cdot \left[q_{u.toe} + 0.5 \cdot \left(\frac{q_{u.heel} - q_{u.toe}}{B}\right) \cdot \left(Toe - \frac{d_{TT}}{2}\right)\right] = -3.65 \cdot \frac{kip}{ft}$$

Tension on Bottom

8.) Footing Strength Capacity Cont'd

Environmental Durability Factor (ACI 350-06)

Note: Required for Tension Controlled structure to encourage durability and liquid-tightness.
Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads. The durability factor is applied to service loads only.

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4 $f_{s.ss} := 24000\,psi$

$$S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$$

Flexure in normal conditions: R10.6.4
- One way element, bar ar 6" spacing

$f_{s.f} := 34000\,psi$

$$S_{d.f} := \max\left(\frac{f_y}{f_{s.f}}, 1.0\right) = 1.76$$

EDF Shear:

$$V_{heel.ED} := S_{d.ss} \left|p_{h.ED1}\right| \cdot Heel = 5.33 \cdot kip$$

$$V_{toe.ED} := S_{d.ss} \left|p_{h.ED2}\right| \cdot Toe = 7.94 \cdot kip$$

EDF Moment:

$$M_{heel.ED} := S_{d.f} \left|p_{h.ED1}\right| \cdot Heel^2 \div 2 = 9.40 \cdot kip \cdot ft$$

$$M_{toe.ED} := S_{d.f} \left|p_{h.ED2}\right| \cdot Toe^2 \div 2 = 9.81 \cdot kip \cdot ft$$

Ultimate Shear-Cantilevered Section:

$$V_{u.heel} := \left|p_h\right| \cdot Heel = 12.91 \cdot kip$$

$$V_{u.toe} := \left|p_{h2}\right| \cdot Toe = 12.78 \cdot kip$$

Ultimate Moment-Cantilevered Section:

$$M_{u.heel} := \left|p_h\right| \cdot Heel^2 \div 2 = 32.26 \cdot kip \cdot ft$$

$$M_{u.toe} := \left|p_{h2}\right| \cdot Toe^2 \div 2 = 22.37 \cdot kip \cdot ft$$

8.) Footing Strength Capacity Cont'd

Footing Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
(*ACI 318-14, §22.5.5.1)*

$$V_c := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{HT} = 32.90 \cdot kip$$

Design Shear Capacity:

$$\phi V_n := \phi_v \cdot (V_c) = 24.68 \cdot kip$$

$V_{u.heel} = 12.91 \cdot kip$ $\quad V_{heel.ED} = 5.33 \cdot kip$

Demand Capacity Ratio-Heel:

$$DCR_H := \frac{\max(V_{u.heel}, V_{heel.ED})}{\phi V_n} = 0.52$$

< 1.0 therefore section is okay for shear strength

$V_{u.toe} = 12.78 \cdot kip$ $\quad V_{toe.ED} = 7.94 \cdot kip$

Demand Capacity Ratio-Toe:

$$DCR_{F.T} := \frac{\max(V_{u.toe}, V_{toe.ED})}{\phi V_n} = 0.52$$

< 1.0 therefore section is okay for shear strength

Footing Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(*ACI 318-14)*

$$M_n := \rho_{HT} \cdot f_y \cdot b_w \cdot d_{HT}^{\,2} \cdot \left(1 - 0.59 \cdot \rho_{HT} \cdot \frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n := \phi_t \cdot M_n = 89.01\, ft \cdot kip$$

$M_{u.heel} = 32.26\, ft \cdot kip$ $\quad M_{heel.ED} = 9.40\, ft \cdot kip$

Demand Capacity Ratio-Heel:

$$\boxed{DCR_H := \frac{\max(M_{u.heel}, M_{heel.ED})}{\phi M_n} = 0.36}$$

< 1.0 therefore section is okay for flexural strength

$M_{u.toe} = 22.37\, ft \cdot kip$ $\quad M_{toe.ED} = 9.81\, ft \cdot kip$

Demand Capacity Ratio-Toe:

$$\boxed{DCR_T := \frac{\max(M_{u.toe}, M_{toe.ED})}{\phi M_n} = 0.25}$$

< 1.0 therefore section is okay for flexural strength

9.) Footing Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in, 5 \cdot 0.5\left(t_{wall.B} + t_{wall.T}\right), 12in\right] = 12.00 \cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{cases} 0.0020 & \text{if } f_y < 60ksi \\ \max\left[\left(0.0018 \cdot 60ksi \div f_y\right), 0.0014\right] & \text{otherwise} \end{cases} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0030$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0030$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min.toe} := \rho_{max} \cdot b_w \cdot t_{base} = 0.86 \cdot in^2$$

$$A_{s.min.heel} := \rho_{max} \cdot b_w \cdot t_{base} = 0.86 \cdot in^2$$

Reinforcement perpendicular to Wall:

Bar Size and Spacing of Reinforcement:

$Size_{TT} = 9.00$ $sTT = 1.00\,ft$

$Size_{TB} = 9.00$ $sTB = 1.00\,ft$

Area of Reinforcement:

$A_{sTT} = 1.00 \cdot in^2$ $A_{sTB} = 1.00 \cdot in^2$

$$\boxed{\frac{A_{s.min.toe}}{A_{sTT} + A_{sTB}} = 0.43}$$

< 1.0 therefore okay

Bar Size and Spacing of Reinforcement:

$Size_{HT} = 9.00$ $sHT = 1.00\,ft$

$Size_{HB} = 9.00$ $sHB = 1.00\,ft$

Area of Reinforcement:

$A_{sHT} = 1.00 \cdot in^2$ $A_{sHB} = 1.00 \cdot in^2$

$$\boxed{\frac{A_{s.min.heel}}{A_{sHT} + A_{sHB}} = 0.43}$$

< 1.0 therefore okay

9.) Footing Rebar Design Cont'd

Reinforcement parallel to Wall:

Toe:

Bar Size and Spacing of Reinforcement: $Size_{TTP} = 7.00$ $sTTP = 1.00\,ft$

$Size_{TBP} = 7.00$ $sTBP = 1.00\,ft$

Area of Reinforcement: $A_{sTTP} = 0.60 \cdot in^2$ $A_{sTBP} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min.toe}}{A_{sTTP} + A_{sTBP}} = 0.72}$$ < 1.0 therefore okay

Heel:

Bar Size and Spacing of Reinforcement: $Size_{HTP} = 7.00$ $sHTP = 1.00\,ft$

$Size_{HBP} = 7.00$ $sHBP = 1.00\,ft$

Area of Reinforcement: $A_{sHTP} = 0.60 \cdot in^2$ $A_{sHBP} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min.heel}}{A_{sHTP} + A_{sHBP}} = 0.72}$$ < 1.0 therefore okay

Minimum Flexural Steel: (ACI 318-14, §9.6.1.2)

Perpendicular to Wall

Heel:

$$A_{s.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d_{HT}, \frac{200 \cdot psi \cdot b_w \cdot d_{HT}}{f_y}\right) = 0.82 \cdot in^2$$

Toe:

$$A_{s.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d_{TB}, \frac{200 \cdot psi \cdot b_w \cdot d_{TB}}{f_y}\right) = 0.78 \cdot in^2$$

$$\boxed{\frac{A_{s.min}}{A_{sHT}} = 0.78} \quad \boxed{\frac{A_{s.min}}{A_{sTB}} = 0.78}$$ < 1.0 therefore okay

9.) Footing Rebar Design Cont'd

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Perpendicular to Wall

Reinforcement Ratio Provided: $\rho_p := \dfrac{AbHT}{d_{HT} \cdot b_w} = 0.0041$

Depth to Neutral Axis: $c := \dfrac{\rho_p \cdot f_y \cdot d_{HT}}{0.85 \cdot \beta_1 \cdot f'_c} = 1.58 \cdot in$

Calculated Strain in Tension Steel at Nominal Strength: $\varepsilon_t := 0.003 \cdot \left(\dfrac{d_{HT}}{c} - 1\right) = 0.04$

$\boxed{\dfrac{\varepsilon_{t.min}}{\varepsilon_t} = 0.11}$ < 1.0 Therefore Okay

9.) Key Design

Shear at the Top of the Key:

Summation of horizontal driving forces:

Case 1: $H_{sum.1} := \max\left(V_{F1a} + V_{F1b} + V_{C1a} + V_{C1b} - F_{F.2}, 0.01kip\right) = 3.14 \cdot kip$

Case 2: $H_{sum.2} := V_{F1a} + V_{F1b} + V_{Ea} + V_{Eb} + V_{Ia} + V_{Ib} + V_{I.3} - F_{F.2} = 10.21 \cdot kip$

$$V_u := \max\left(H_{sum.1}, H_{sum.2}\right) = 10.21 \cdot kip$$

Key Capacity (1-foot Design Strip) - SHEAR CAPACITY:

Thickness at the Top of Key: $t_{key} := 1.5ft$

Depth to Centroid of Reinforcement: $d_{SK} := t_{key} - clrS - d_{b_4} - 0.5 \cdot d_{b_4} = 1.19\,ft$

One Way Shear Strength:
ACI 318-14, §22.5.5.1) $V_c := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{SK} = 22.94 \cdot kip$

Design Shear Capacity: $\phi V_n := \phi_v \cdot V_c$

Demand Capacity Ratio: $DCR_V := \dfrac{V_u}{\phi V_n}$

$DCR_V = 0.59$

< 1.0 therefore section is okay for shear strength

Spillway Headwall Key - Upper Black Creek Reservoir

Codes References

1) American Concrete Institute (ACI). Building Code Requirements for Structural Concrete (ACI 318-14), (2014).
2) ACI (American Concrete Institute), 2006. Code Requirements for Environmental Engineering Concrete Structures (ACI 350-06).
3) U.S. Army Corps of Engineers (USACE). 2003 "Strength Design for Reinforced-Concrete Hydraulic Structures." EM 1110-2-2104-Appendix E: Table E-1.

Structural Design

The following design calculates the design strength for the Upper Black Creek Reservoir Spillway Headwall Key necessary to develop the full passive resistance at the base of the slab. The key is designed as a cantilevered beam with soil loads. The strength/capacity checks are performed using the one foot strip method and code requirements in ACI 318-14, ACI 350-06 and USACE EM 1110-2-2104.

Design Loads (Forces)

The applied loads and load cases for the key are calculated using guidelines from the documents referenced above. USACE and ACI strength/service load combinations are applied and the resulting combinations are shown below.

Key Reinforcement Summary

Vertical Reinforcement Tension Face = #4 at 6" O.C.

Horizontal Reinforcement Channel = #4 at 6" O.C.

Key Design Properties:

Key Height: $H_{key} := 3ft$

Bottom Key Thickness: $t_{key.B} := 2ft + 0in$

Top Key Thickness: $t_{key.T} := 2ft + 0in$

Thickness provided for connection to cutoff wall

Angle of Chute Inclination: $\theta := 26.57deg$

Slab thickness: $t_{slab} := 2.0ft$

Key depth below slab: $d_{key} := 3.0ft$

Design Width: $b_w := 12in$

Unit weight of concrete: $\gamma_{conc} := 150 \cdot pcf$

Distance between Control Joints: $L_{CJ} := 30ft$

Specified Compressive Strength of Concrete: $f'_c := 4500psi$

Fill Properties:

Fill Properties

Angle of internal friction: $\phi_f := 30 \cdot deg$

Unit weight: $\gamma_f := 130pcf$

Active earth pressure coefficient: $k_a := \tan\left(45deg - 0.5 \cdot \phi_f\right)^2 = 0.33$

Passive earth pressure coefficient: $k_p := \tan\left(45deg + 0.5 \cdot \phi_f\right)^2 = 3.00$

Applied Loads: Static

Passive Earth Pressure

Active Pressure on the upstream side of key is conservatively ignored for shear.

Shear at Top of Key: $F := k_p \cdot \left[\gamma_f \cdot t_{slab} \cdot H_{key} + 0.5 \cdot \gamma_f \cdot \left(H_{key}\right)^2\right] \cdot b_w = 4.09 \cdot kip$

Shear as at a Distance 'y' From Base of Key:

$$V_F(y) := \begin{cases} F - k_p \cdot \left[\gamma_f \cdot t_{slab} \cdot \left(H_{key} - y\right) + 0.5 \cdot \gamma_f \cdot \left(H_{key} - y\right)^2\right] \cdot b_w & \text{if } y \le H_{key} \\ 0 & \text{otherwise} \end{cases}$$

$V_F\left(H_{key}\right) = 4.09 \cdot kip$ $\qquad V_F(0ft) = 0.00 \cdot kip$

Moment at Top of Key:

$$M_F := \left(k_p - k_a\right) \cdot \left[\gamma_f \cdot t_{slab} \cdot 0.5\left(H_{key}\right)^2 + 0.5 \cdot \gamma_f \cdot \frac{2}{3}\left(H_{key}\right)^3\right] \cdot b_w = 6.24 \cdot kip \cdot ft$$

Moment as at a Distance 'y' From Base of Key:

$$M_F(y) := \begin{cases} \left(k_p - k_a\right) \cdot \left[\gamma_f \cdot t_{slab} \cdot 0.5\,(y)^2 + 0.5 \cdot \gamma_f \cdot \frac{2}{3}\,(y)^3\right] \cdot b_w & \text{if } y \le H_{key} \\ 0 & \text{otherwise} \end{cases}$$

$M_F\left(H_{key}\right) = 6.24 \cdot kip \cdot ft$ $\qquad M_F(0ft) = 0.0 \cdot kip \cdot ft$

LOAD CASE 1-Loading Conditions & Combinations:

Unfactored Shear At Top of Key: $V(y) := V_F(y)$

Ultimate Shear At Top of Key:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L) *[Table E-9, Load Case 1A]*

$$V_{u.1}(y) := 2.2 \cdot \left(V_F(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$V_{u.2}(y) := 1.6 \cdot V_F(y)$$

Controlling Case: $V_u(y) := \max\left(V_{u.1}(y), V_{u.2}(y)\right)$

$$V_u\left(H_{key}\right) = 9.01 \cdot kip \qquad \boxed{V_{u.1}\left(H_{key}\right) = 9.01 \cdot kip}$$

Unfactored Moment At Top of Key: $M(y) := M_F(y)$

Ultimate Moment At Top of Key:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L)

$$M_{u.1}(y) := 2.2 \cdot \left(M_F(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$M_{u.2}(y) := 1.6 \cdot M_F(y)$$

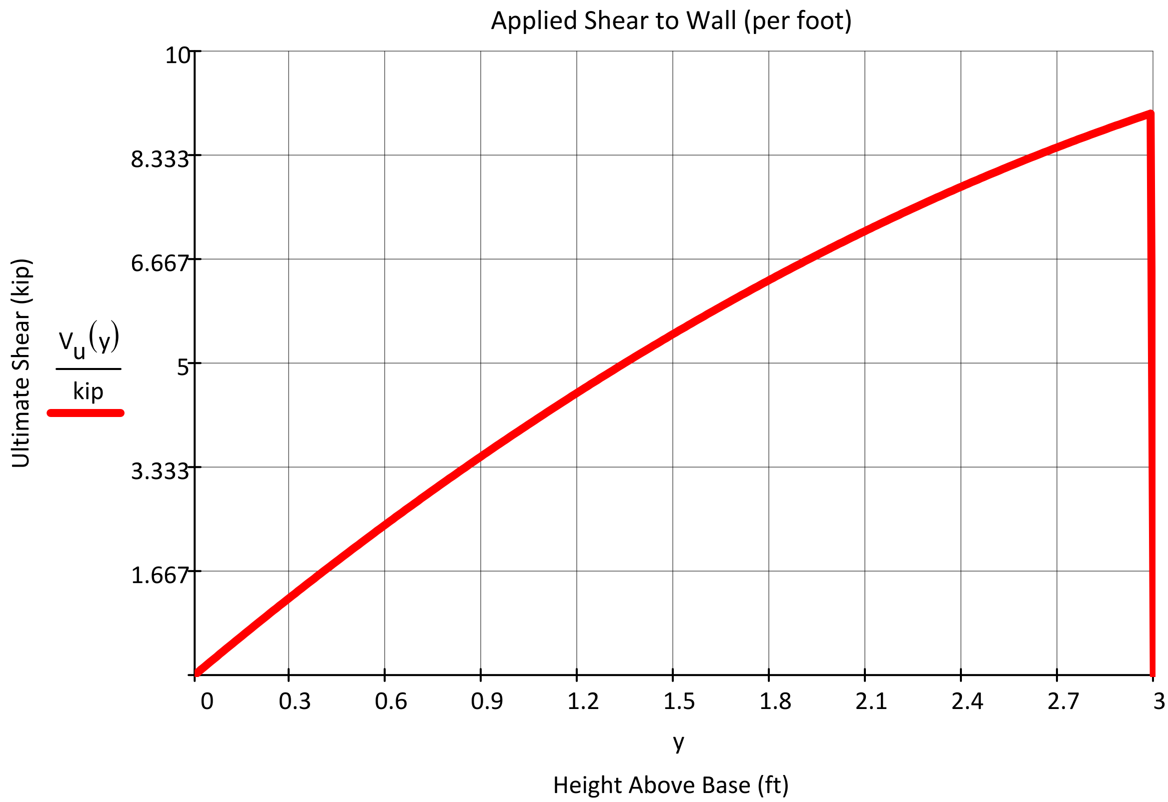
Controlling Case: $M_u(y) := \max\left(M_{u.1}(y), M_{u.2}(y)\right)$

$$M_u\left(H_{key}\right) = 13.73 \cdot kip \cdot ft \qquad \boxed{M_{u.1}\left(H_{key}\right) = 13.73\, ft \cdot kip}$$

Load Summary

Applied Load:

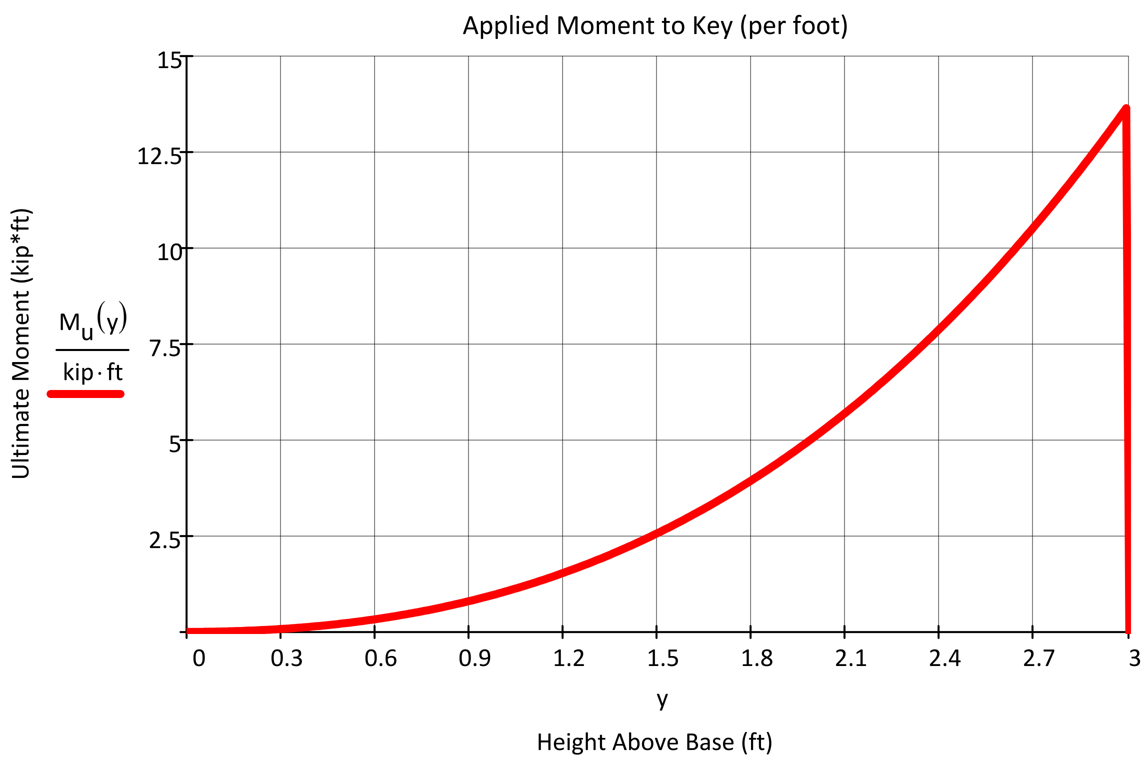
Ultimate Shear at Top of Key: $V_u(H_{key}) = 9.01 \cdot kip$



Load Summary Cont'd

Controlling Applied Load:

Ultimate Moment at Top of Key: $M_u(H_{key}) = 13.73 \cdot kip \cdot ft$



Key Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05 \cdot \dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Horizontal Reinforcement:

Bar Size and Spacing: $Size_{SH} := 5$ $sSH := 6in$
(Channel & Embankment)

Reinforcement Clear Cover: $clrS := 3in$

Diameter & Cross-Sectional Area of Bars: $dbSH := d_{b_{Size_{SH}}} = 0.63 \cdot in$ $AbSH := A_{b_{Size_{SH}}} = 0.31 \cdot in^2$

Area of Reinforcement:

$$A_{sSH} := AbSH \cdot \frac{12in}{sSH} = 0.62 \cdot in^2$$

$$m := \left(t_{key.B} - t_{key.T}\right) \div H_{key} = 0.00$$

Section Depth at a Height 'y' from the base: $t_{key}(y) := t_{key.T} + m \cdot y$

Depth to Centroid of Embankment Reinf: $d_{SH}(y) := t_{key}(y) - clrS - 0.5 \cdot dbSH$

Reinforcement Ratio: $\rho_{SH}(y) := A_{sSH} \div \left(b_w\, d_{SH}(y)\right)$

Key Concrete & Rebar Strength Design Cont'd

Vertical Reinforcement:

Bar Size and Spacing: (Embankment) $Size_{SV} := 5$ $sSV := 6in$

Diameter & Cross-Sectional Area of Bars: $dbSV := d_{b_{Size_{SV}}} = 0.63 \cdot in$ $AbSV := A_{b_{Size_{SV}}} = 0.31 \cdot in^2$

Area of Tension Reinforcement: $A_{sSV} := AbSV \cdot \frac{12in}{sSV} = 0.62 \cdot in^2$

$m := (t_{key.T} - t_{key.B}) \div H_{key} = 0.00$

Section Depth at a Height 'y' from the base: $t_{key}(y) := t_{key.B} + m \cdot y$

Depth to Centroid of Reinforcement: $d_{SV}(y) := t_{key}(y) - clrS - dbSH - 0.5 \cdot dbSV$

Reinforcement Ratio: $\rho_{SV}(y) := A_{sSV} \div (b_w d_{SV}(y))$

Key Strength Capacity

Key Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
ACI 318-14, §22.5.5.1)

$$V_c(y) := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{SV}(y)$$

$$V_c(H_{key}) = 32.30 \cdot kip$$

Design Shear Capacity:

$$\phi V_n(y) := \phi_V \cdot (V_c(y))$$

$$\phi V_n(0ft) = 24.23 \cdot kip$$

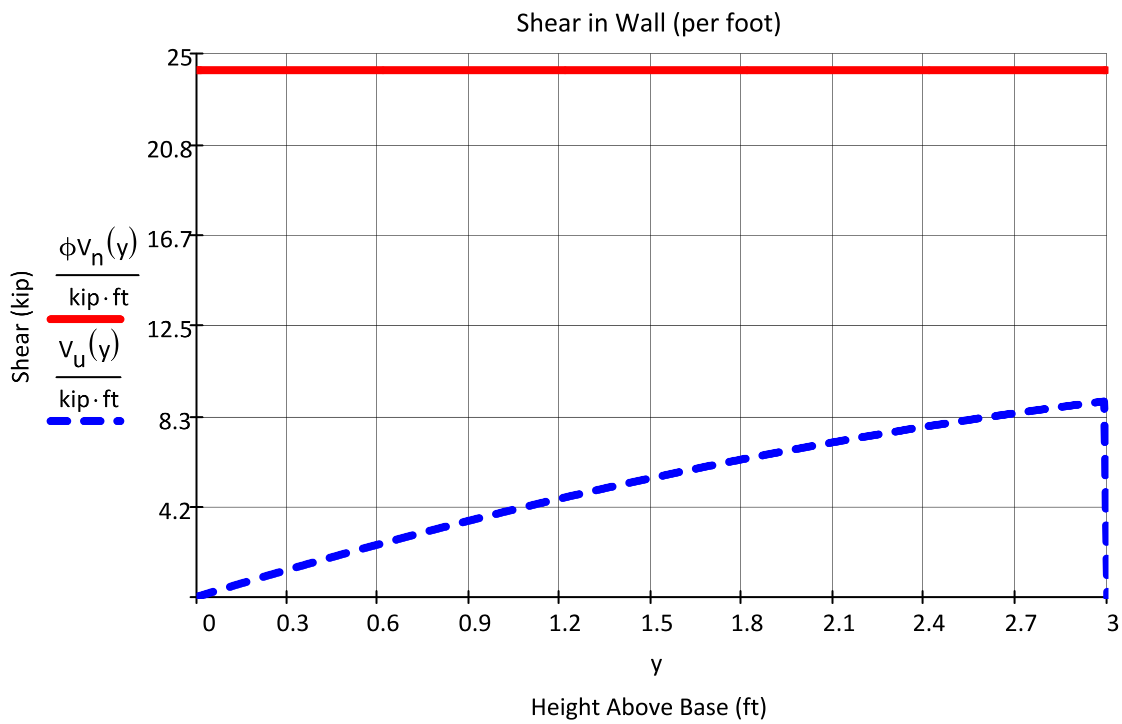
$$V_u(0ft) = 0.00 \cdot kip$$

Demand Capacity Ratio:

$$DCR_V(y) := \frac{V_u(y)}{\phi V_n(y)}$$

$$DCR_V(H_{key}) = 0.37$$

< 1.0 therefore section is okay for shear strength



Key Strength Capacity Cont'd

Key Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(ACI 318-14)

$$M_n(y) := \rho_{SV}(y) \cdot f_y \cdot b_w \cdot d_{SV}(y)^2 \cdot \left(1 - 0.59 \cdot \rho_{SV}(y) \cdot \frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n(y) := \phi_t \cdot M_n(y)$$

$$\phi M_n(H_{key}) = 54.84\, ft \cdot kip$$

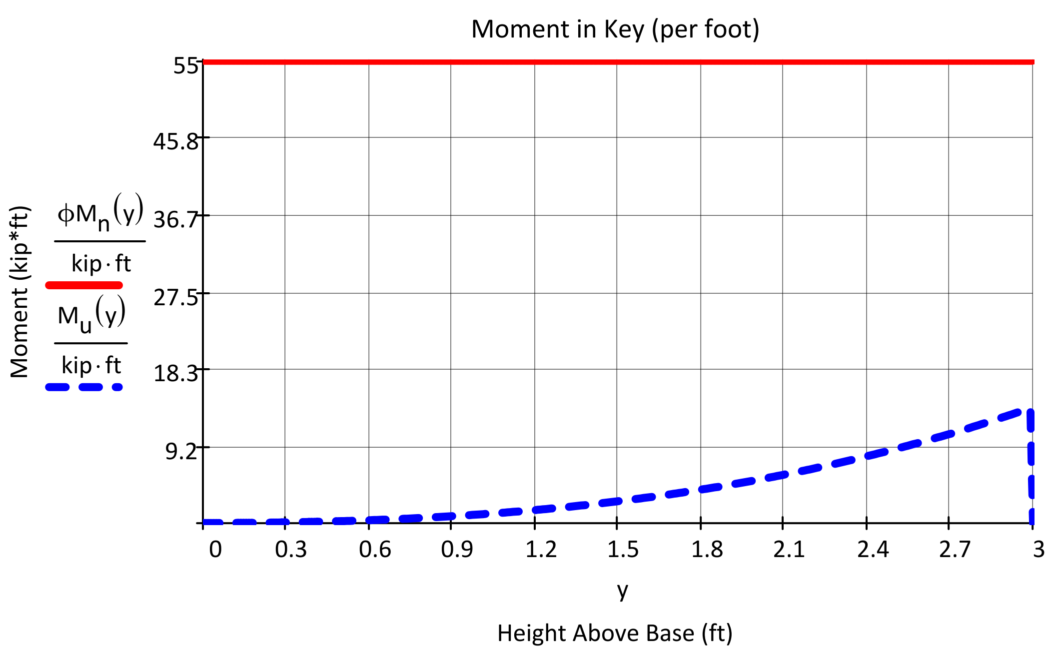
$$M_u(H_{key}) = 13.73 \cdot kip \cdot ft$$

Demand Capacity Ratio:

$$DCR_M(y) := \frac{M_u(y)}{\phi M_n(y)}$$

$$DCR_M(H_{key}) = 0.25$$

< 1.0 therefore section is okay for flexural strength



Key Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in, 5 \cdot 0.5\left(t_{key.B} + t_{key.T}\right), 12in\right] = 12.00 \cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{cases} 0.0020 & \text{if } f_y < 60ksi \\ \max\left[\left(0.0018 \cdot 60ksi \div f_y\right), 0.0014\right] & \text{otherwise} \end{cases} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0040$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0040$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min} := \rho_{max} \cdot b_w \cdot 0.5 \cdot \left(t_{key.B} + t_{key.T}\right) = 1.15 \cdot in^2$$

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Vertical Reinforcement:

Bar Size and Spacing of Reinforcement: $Size_{SV} = 5.00$ $\quad sSV = 6.00 \cdot in$

Diameter & Cross-Sectional Area of Bars: $dbSV = 0.63 \cdot in$ $\quad AbSV = 0.31 \cdot in^2$

Area of Reinforcement: $A_{sSV} = 0.62 \cdot in^2$

$$\boxed{\frac{A_{s.min}}{2A_{sSV}} = 0.93}$$

< 1.0 therefore okay

Horizontal Reinforcement:

Bar Size and Spacing of Reinforcement: $Size_{SH} = 5.00$ $\quad sSH = 6.00 \cdot in$

Diameter & Cross-Sectional Area of Bars: $dbSH = 0.63 \cdot in$ $\quad AbSH = 0.31 \cdot in^2$

Area of Reinforcement: $A_{sSH} = 0.62 \cdot in^2$

$$\boxed{\frac{A_{s.min}}{2 \cdot A_{sSH}} = 0.93}$$

< 1.0 therefore okay

Key Rebar Design Cont'd

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Reinforcement Ratio Provided:
(Vertical)

$$\rho_{pV} := \frac{A_{sSV}}{d_{SV}(H_{key}) \cdot b_w} = 0.0026$$

Reinforcement Ratio Provided:
(Horizontal)

$$\rho_{pH} := \frac{A_{sSH}}{d_{SH}(H_{key}) \cdot b_w} = 0.0025$$

Depth to Neutral Axis:
(Vertical)

$$cV := \frac{\rho_{pV} \cdot f_y \cdot d_{SV}(H_{key})}{0.85 \cdot \beta_1 \cdot f'_c} = 0.98 \cdot in$$

Depth to Neutral Axis:
(Horizontal)

$$cH := \frac{\rho_{pH} \cdot f_y \cdot d_{SH}(H_{key})}{0.85 \cdot \beta_1 \cdot f'_c} = 0.98 \cdot in$$

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Calculated Strain in Tension
Steel at Nominal Strength:
(Vertical)

$$\varepsilon_{tV} := 0.003 \cdot \left(\frac{d_{SV}(H_{key})}{cV} - 1 \right) = 0.06$$

(Horizontal)

$$\varepsilon_{tH} := 0.003 \cdot \left(\frac{d_{SH}(0ft)}{cH} - 1 \right) = 0.06$$

$\boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tV}} = 0.07}$ $\boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tH}} = 0.07}$ < 1.0 therefore okay

Spillway Wall (Inclined) - Upper Black Creek Reservoir

Codes References

1) American Concrete Institute (ACI). Building Code Requirements for Structural Concrete (ACI 318-14), (2014).
2) ACI (American Concrete Institute), 2006. Code Requirements for Environmental Engineering Concrete Structures (ACI 350-06).
3) American Society of Civil Engineers (ASCE). Minimum Design Loads for Buildings and Other Structures (ASCE 7-10), (2013).
4) U.S. Army Corps of Engineers (USACE). Engineering and Design: Retaining and Flood Walls, (1989). EM 1110-2-2502.
5) U.S. Army Corps of Engineers (USACE). 2003 "Strength Design for Reinforced-Concrete Hydraulic Structures." EM 1110-2-2104-Appendix E: Table E-1.
6) Agusti, G. C. and Sitar. 2013. UCB GT 13-02 "Seismic Earth Pressures on Retaining Structures with Cohesive Backfills." UCB GT 13-02.

Structural Design

The following design calculates the applied loads and design strength for the Upper Black Creek Reservoir spillway wall. The wall is designed as a cantilevered retaining wall with soil and seismic loads. The strength/capacity checks are performed using the one foot strip method and code requirements in ACI 318-14, ACI 350-06 and USACE EM 1110-2-2104.

Design Loads (Forces)

The applied loads and load cases for the wall are calculated using guidelines from the documents referenced above. USACE and ACI strength/service load combinations are applied and the resulting combinations are shown below.

Wall Reinforcement Summary

Vertical Reinforcement

Tension Face = #7 at 12" O.C.

Compression Face = #7 at 12" O.C.

Horizontal Reinforcement

Channel = #5 at 12" O.C.

Embankment = #5 at 12" O.C.

Foundation Dowels

Full height bars, mechanical splices or slab tension splices will be allowed. Bars must be developed at wall/foundation joint.

Counterforts

None

Footing Reinforcement Summary

Perpendicular to Wall

Tension Face = #9 at 12" O.C.

Compression Face = #9 at 12" O.C.

Parallel to Wall

Top = #7 at 12" O.C.

Bottom = #7 at 12" O.C.

Wall Design Properties:

Top of Wall Elevation:	$El_{top} := 8756ft$
Bottom of Wall Elevation:	$El_{bot} := 8746.5ft$
Wall Height:	$H_{wall} := El_{top} - El_{bot} = 9.50\,ft$
Bottom Wall Thickness:	$t_{wall.B} := 1ft + 6in$
Top Wall Thickness:	$t_{wall.T} := 1ft + 0in$
Footing width:	$B := 8.0ft$
Toe projection:	$Toe := 3.5ft$
Heel projection:	$Heel := B - Toe - t_{wall.B} = 3.00\,ft$
Footing thickness:	$t_{base} := 2.0ft$
Slab thickness:	$t_{slab} := 1.0ft$
Design Width:	$b_w := 12in$
Unit weight of concrete:	$\gamma_{conc} := 150 \cdot pcf$
Distance between Control Joints:	$L_{CJ} := 25ft$
Specified Compressive Strength of Concrete:	$f'_c := 4500psi$
Minimum Tensile Strength of Concrete:	$f_t := 0.1 \cdot f'_c = 450\,psi$ *[ACI R22.2.2.2]*

Water Surface Elevations and Properties:

Elevation of normal water surface: (Assumed at base of structure)	$El_{nws} := El_{bot} - t_{base} + 0ft = 8744.50\,ft$
Water Unit Weight:	$\gamma_w := 62.4pcf$

Fill Elevations and Properties:

Reference:

Elevation of fill material beside channel: $El_{fill} := 8755ft$

Fill Properties (Soil type: SM)

Angle of internal friction: $\phi_f := 30 \cdot deg$

Unit weight: $\gamma_f := 130pcf$

Angle of inclined backfill: $\alpha_f := 0deg$

At-rest earth pressure coefficient: $k_0 := 1 - \sin(\phi_f) = 0.50$

Active earth pressure coefficient: $k_a := \tan(45deg - 0.5 \cdot \phi_f)^2 = 0.33$

Passive earth pressure coefficient: $k_p := \tan(45deg + 0.5 \cdot \phi_f)^2 = 3.00$

Friction factor for mass concrete on sound rock: $\delta_1 := 0.7$ *[NAVFAC p7.2-63]*

Seismic Properties:

10,000yr Horizontal Seismic Coefficient: $k_h := 0.4972$ [USGS Unified Hazard Tool]

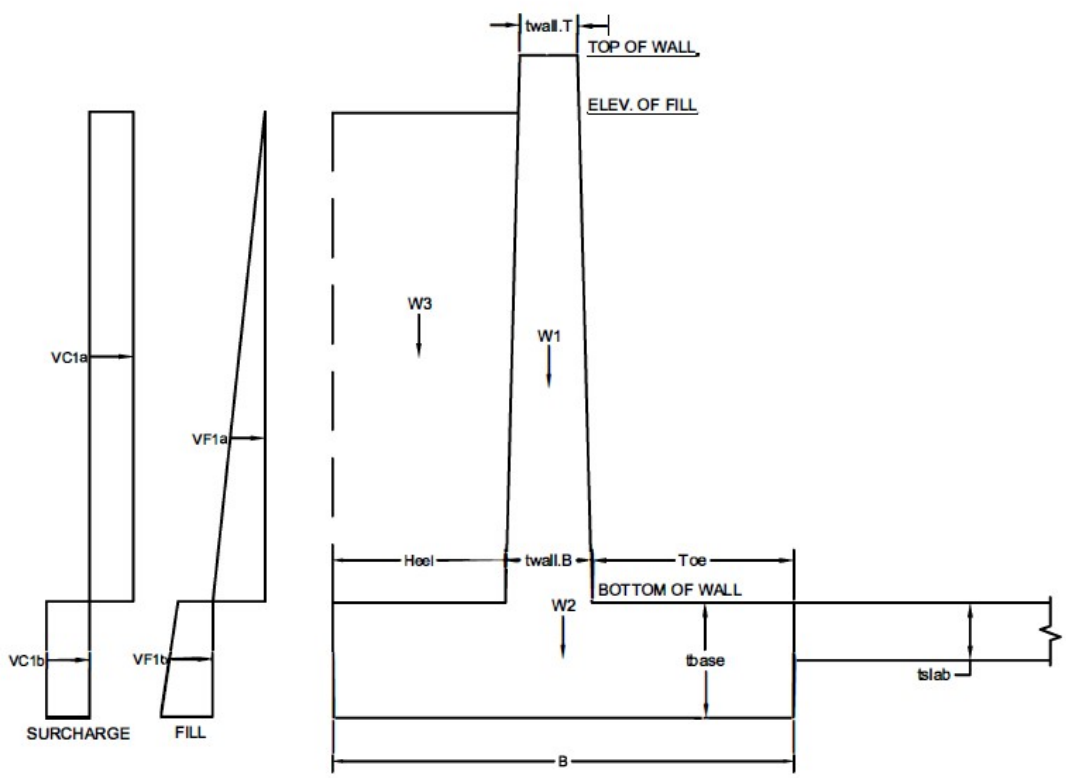
Earth Pressure Coefficient: $K_{ae} := 0.42 \cdot k_h = 0.21$ *[Augusti: UCB GT 13-02: Eq. 4.3]*

Other Properties:

Construction Surcharge: $q_c := 200psf$

1.) LOAD CASE 1 (Usual Condition) - Spillway Wall Loading:

Note: The retaining wall stem is analyzed as a cantilever beam with fixity provided at the footing. Beam is subject to lateral forces due to at-rest earth forces due to fill and construction live loads. Groundwater table is assumed below the wall section.



1.) LOAD CASE 1 (Usual Condition) - Spillway Wall Loading:

LOAD CASE 1-Applied Loads: Static

Active Earth Pressure

Base Shear:

$$V_{F1a} := 0.5 \cdot k_a \gamma_f \cdot \left(El_{fill} - El_{bot}\right)^2 b_w = 1.57 \cdot kip$$

Footing Shear:

$$V_{F1b} := \left[k_a \gamma_f \cdot \left(El_{fill} - El_{bot}\right) \cdot t_{base} + 0.5 \cdot k_a \cdot \gamma_f \cdot t_{base}{}^2\right] \cdot b_w = 0.82 \cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_{F.1}(y) := \begin{cases} \left[0.5 \cdot k_a \cdot \gamma_f \cdot \left(El_{fill} - El_{bot} - y\right)^2 \cdot b_w\right] & \text{if } \ y < \left(El_{fill} - El_{bot}\right) \\ 0 & \text{otherwise} \end{cases}$$

$V_{F.1}\left(El_{nws} - El_{bot}\right) = 2.39 \cdot kip$ $\qquad$ $V_{F.1}(0ft) = 1.57 \cdot kip$

Base Moment:

$$M_{F.1.b} := V_{F1a} \cdot \left[0.33\left(El_{fill} - El_{nws}\right) + \left(El_{nws} - El_{bot}\right)\right] = 2.29\, ft \cdot kip$$

Moment as at a Distance 'y' From Base of Wall:

$$M_{F.1}(y) := 0.5 \cdot k_a \cdot \gamma_f \cdot \left(El_{fill} - El_{bot} - y\right)^2 \cdot b_w 0.33\left(El_{fill} - El_{bot} - y\right)$$

$M_{F.1}\left(El_{nws} - El_{bot}\right) = 8.28 \cdot kip \cdot ft$ $\qquad$ $M_{F.1}(0ft) = 4.4 \cdot kip \cdot ft$

Passive Earth Pressure

Rock to Interior:

$$V_{F4} := 0.5 \cdot k_p \cdot \gamma_f \cdot \left(t_{base} - t_{slab}\right)^2 \cdot 1ft = 0.20 \cdot kip$$

$$V_{F5} := k_p \cdot \gamma_{conc} \cdot t_{slab} \cdot \left(t_{base} - t_{slab}\right) \cdot 1ft = 0.45 \cdot kip$$

$$F_{F.T2} := V_{F4} + V_{F5} = 0.64 \cdot kip$$

Vertical Forces

Weight of stem: $W_1 := \gamma_{conc} \cdot \left(El_{top} - El_{bot}\right) \cdot 0.5\left(t_{wall.B} + t_{wall.T}\right) \cdot 1ft = 1.78 \cdot kip$

Weight of footing: $W_2 := \gamma_{conc} \cdot B \cdot t_{base} \cdot 1ft = 2.40 \cdot kip$

Weight of fill over heel: $W_3 := \gamma_f \cdot \left[Heel + 0.5 \cdot \left(t_{wall.B} - t_{wall.T}\right)\right] \cdot \left(El_{fill} - El_{bot}\right) \cdot 1ft = 3.59 \cdot kip$

1.) LOAD CASE 1 (Usual Condition) Cont'd

LOAD CASE 1-Applied Loads: Static

Construction Surcharge

Base Shear: $V_{C1a} := k_a \cdot q_c \cdot (El_{fill} - El_{bot}) \cdot b_w = 0.57 \cdot kip$

Shear as at a Distance 'y' From Base of Wall:

$$V_C(y) := \begin{cases} V_{C1a} \cdot \dfrac{(El_{fill} - El_{bot} - y)}{(El_{fill} - El_{bot})} & \text{if } y < (El_{fill} - El_{bot}) \\ 0 & \text{otherwise} \end{cases}$$

$V_C(El_{nws} - El_{bot}) = 0.70 \cdot kip$ $\quad$ $V_C(0ft) = 0.57 \cdot kip$

Footing Shear: $V_{C1b} := k_a \cdot q_c \cdot t_{base} \cdot b_w = 0.13 \cdot kip$

Base Moment: $M_C := 0.5 \cdot V_{C1a} \cdot (El_{fill} - El_{bot}) = 2.41 \, ft \cdot kip$

Moment as at a Distance 'y' From Base of Wall: $M_C(y) := 0.5 \cdot V_C(y) \cdot \dfrac{(El_{fill} - El_{bot} - y)^2}{(El_{fill} - El_{bot})}$

1.) LOAD CASE 1: (Usual Condition) Cont'd - Global Stability Checks

CHECK SLIDING STABILITY

Assume structure is constrained from transverse sliding. Due to connection with the slab, the spillway wall is not allowed to move freely and is assumed to not slide.

CHECK OVERTURNING STABILITY

Moment Calculations

Horizontal force moment arms:

$$y_{1a} := \left(El_{fill} - El_{bot}\right) \div 3 + t_{base} = 4.83\,ft$$

$$y_{1b} := 0.5 \cdot t_{base} = 1.00\,ft$$

$$y_{c1a} := \left(El_{fill} - El_{bot}\right) \div 2 + t_{base} = 6.25\,ft$$

$$y_{c1b} := 0.5 \cdot t_{base} = 1.00\,ft$$

$$y_{FFT2} := 0.33\left(t_{base} - t_{slab}\right) = 0.33\,ft$$

Vertical force moment arms:

$$x_1 := Toe + t_{wall.T} + 0.33 \cdot \left(t_{wall.B} - t_{wall.T}\right) = 4.66\,ft$$

$$x_2 := B \div 2 = 4.00\,ft$$

$$x_3 := B - Heel \div 2 = 6.50\,ft$$

Summation of vertical forces:

$$V_{sum} := W_1 + W_2 + W_3 = 7.77 \cdot kip$$

Overturning Moment:

$$M_{o.1} := V_{F1a} \cdot y_{1a} + V_{F1b} \cdot y_{1b} + V_{C1a} \cdot y_{c1a} \ldots + V_{C1b} \cdot y_{c1b} = 12.06 \cdot kip \cdot ft$$

Resisting Moment:

$$M_r := W_1 \cdot x_1 + W_2 \cdot x_2 + W_3 \cdot x_3 + F_{F.T2} \cdot y_{FFT2} = 41.47 \cdot kip \cdot ft$$

Location of resultant:

$$x_{bar.1} := \frac{M_r - M_{o.1}}{V_{sum}} = 3.78\,ft$$

Eccentricity of the resultant:

$$ecc_1 := \frac{B}{2} - x_{bar.1} = 0.22\,ft$$

Overturning check:

$$Check_{ot.1} := \begin{cases} \text{"LC1 Overturning Stability OK"} & \text{if } \frac{1}{3} \cdot B \le x_{bar.1} \le \frac{2}{3} B \\ \text{"STOP - LC1 Overturning Stability Unacceptable"} & \text{otherwise} \end{cases}$$

$$Check_{ot.1} = \text{"LC1 Overturning Stability OK"} \quad \frac{1}{3} \cdot B = 2.67\,ft \quad \frac{2}{3} \cdot B = 5.33\,ft$$

1.) LOAD CASE 1 (Usual Condition) Cont'd

LOAD CASE 1-Loading Conditions & Combinations:

Unfactored Shear At Base of Wall: $V_{LC1}(y) := V_{F.1}(y) + V_C(y)$

Ultimate Shear At Base of Wall:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L) *[Table E-9, Load Case 1A]*

$$V_{uLC1.1}(y) := 2.2 \cdot \left(V_{F.1}(y) + V_C(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$V_{uLC1.2}(y) := 1.6 \cdot V_{F.1}(y)$$

ASCE Load Combination 2: 1.2 (D + F) + 1.6 (L) + 1.6 (H)

$$V_{uLC1.3}(y) := 1.6 \cdot V_C(y) + 1.6 \cdot V_{F.1}(y)$$

Controlling Case:

$$V_{uLC1}(y) := \max\left(V_{uLC1.1}(y), V_{uLC1.2}(y), V_{uLC1.3}(y)\right)$$

$$V_{uLC1}(0ft) = 4.69 \cdot kip \qquad \boxed{V_{uLC1.1}(0ft) = 4.69 \cdot kip}$$

Unfactored Moment At Base of Wall: $M_{LC1}(y) := M_{F.1}(y) + M_C(y)$

Ultimate Moment At Base of Wall:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L)

$$M_{uLC1.1}(y) := 2.2 \cdot \left(M_{F.1}(y) + M_C(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$M_{uLC1.2}(y) := 1.6 \cdot M_{F.1}(y)$$

ASCE Load Combination 2: 1.2 (D + F) + 1.6 (L) + 1.6 (H)

$$M_{uLC1.3}(y) := 1.6 \cdot M_C(y) + 1.6 \cdot M_{F.1}(y)$$

Controlling Case:

$$M_{uLC1}(y) := \max\left(M_{uLC1.1}(y), M_{uLC1.2}(y), M_{uLC1.3}(y)\right)$$

$$M_{uLC1}(0ft) = 14.96 \cdot kip \cdot ft \qquad \boxed{M_{uLC1.1}(0ft) = 14.96\, ft \cdot kip}$$

1.) LOAD CASE 1 (Usual Condition) Cont'd

Environmental Durability Factor (ACI 350-06)

Note: Required for Tension Controlled structure to encourage durability and liquid-tightness. Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads. The durability factor is applied to service loads only.

Specified Yield Strength of Reinforcement: $f_y := 60\text{ksi}$

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4 $f_{s.ss} := 24000\text{psi}$

$$S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$$

Flexure in normal conditions: R10.6.4
- One way element, Bar at 6" spacing

$f_{s.f} := 34000\text{psi}$

$$S_{d.f} := \max\left(\frac{f_y}{f_{s.f}}, 1.0\right) = 1.76$$

Shear in Wall:

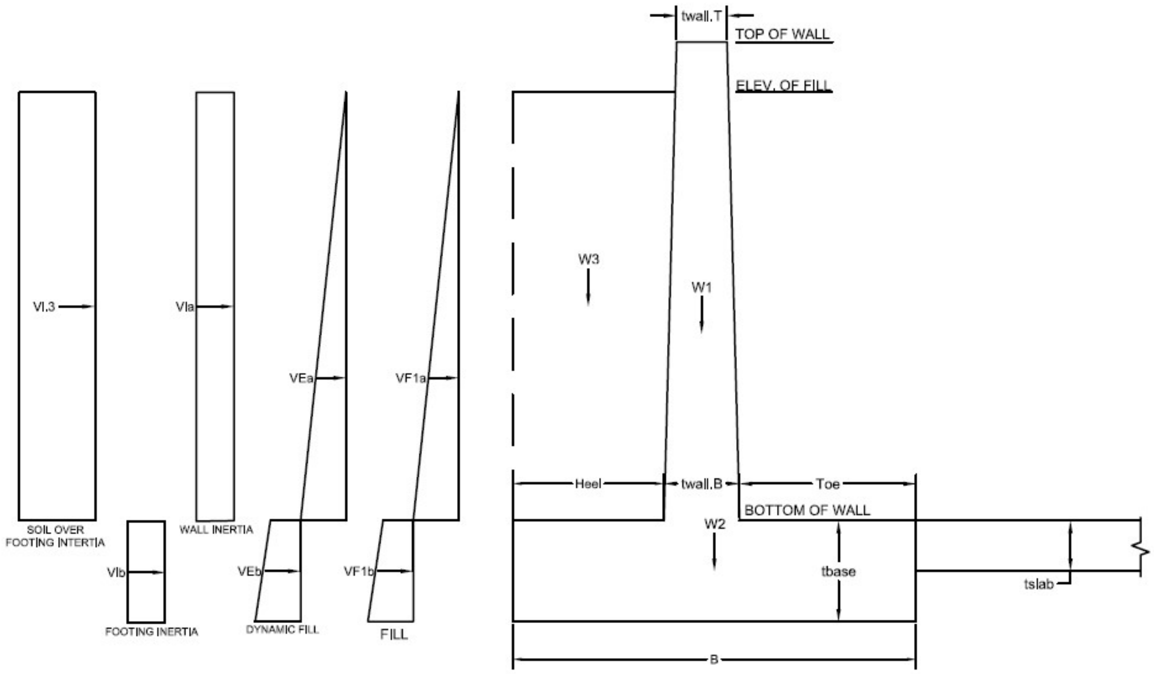
$$\boxed{V_{LC1.ED}(y) := S_{d.ss} \cdot \left(V_{F.1}(y) + V_C(y)\right)} \quad \boxed{V_{LC1.ED}(0\text{ft}) = 5 \cdot \text{kip}} \quad > \quad \boxed{V_{uLC1.1}(0\text{ft}) = 5 \cdot \text{kip}}$$

Moment in Wall:

$$\boxed{M_{LC1.ED}(y) := S_{d.f} \cdot \left(M_{F.1}(y) + M_C(y)\right)} \quad \boxed{M_{LC1.ED}(0\text{ft}) = 12 \cdot \text{kip} \cdot \text{ft}} \quad < \quad \boxed{M_{uLC1.1}(0\text{ft}) = 15 \cdot \text{kip} \cdot \text{ft}}$$

2.) LOAD CASE 2 (Extreme Condition) - Spillway Wall Loading:

Note: The retaining wall stem is analyzed as a cantilever beam with fixity provided at the footing. Beam is subject to lateral forces due to seismic inertial forces due to self-weight and dynamic fill loads. Groundwater table is assumed below the wall section.



2.) LOAD CASE 2 (Extreme Condition) - Spillway Wall Loading:

LOAD CASE 2-Applied Loads: Seismic

Seismic Incremental Force of Earth Pressure

Base Shear:

$$V_{Ea} := 0.5K_{ae}\cdot b_w\cdot\left[\gamma_f\cdot\left(El_{fill} - El_{bot}\right)^2\right] = 0.98\cdot kip$$

$$V_{Eb} := \left[K_{ae}\gamma_f\cdot\left(El_{fill} - El_{bot}\right)\cdot t_{base} + 0.5\cdot K_{ae}\cdot\gamma_f\cdot t_{base}{}^2\right]\cdot b_w = 0.52\cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_E(y) := 0.5\cdot K_{ae}\cdot\gamma_f\cdot\left(El_{fill} - El_{bot} - y\right)^2\cdot b_w \qquad V_E(0ft) = 1.0\cdot kip$$

Base Moment:

$$M_E := V_{Ea}\cdot\left[0.33\left(El_{fill} - El_{bot}\right)\right] = 2.75\,ft\cdot kip$$

Moment at a Distance 'y' From Base of Wall:

$$M_E(y) := 0.5\cdot K_{ae}\cdot\gamma_f\cdot\left(El_{fill} - El_{bot} - y\right)^2\cdot b_w 0.33\left(El_{fill} - El_{bot} - y\right)$$

$$M_E(0ft) = 2.8\cdot kip\cdot ft$$

Inertial Forces

Wall Base Shear:

$$V_{Ia} := k_h\cdot W_1 = 0.89\cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_I(y) := k_h\cdot\gamma_{conc}\cdot\left[t_{wall.T} + \frac{0.5\cdot\left(t_{wall.B} - t_{wall.T}\right)\cdot\left(H_{wall} - y\right)}{\left(H_{wall}\right)}\right]\cdot\left(H_{wall} - y\right)\cdot b_w$$

Base Moment:

$$M_I := V_{Ia}\cdot\left[\frac{H_{wall}\cdot\left(2t_{wall.T} + t_{wall.B}\right)}{3\left(t_{wall.B} + t_{wall.T}\right)}\right] = 3.93\cdot kip\cdot ft$$

Moment at a Distance 'y' From Base of Wall:

$$M_I(y) := V_I(y)\cdot\frac{\left(H_{wall} - y\right)\cdot\left[2t_{wall.T} + \left[t_{wall.T} + \frac{\left(t_{wall.B} - t_{wall.T}\right)\cdot\left(H_{wall} - y\right)}{H_{wall}}\right]\right]}{3\left[t_{wall.T} + \frac{\left(t_{wall.B} - t_{wall.T}\right)\cdot\left(H_{wall} - y\right)}{H_{wall}} + t_{wall.T}\right]}$$

Footing Shear:

$$V_{Ib} := k_h\cdot W_2 = 1.19\cdot kip$$

Soil over Footing Shear:

$$V_{I.3} := k_h\cdot W_3 = 1.79\cdot kip$$

2.) LOAD CASE 2 (Extreme Condition) Cont'd -Global Stability Checks

CHECK SLIDING STABILITY

Assume structure is constrained from transverse sliding. Due to connection with the slab, the spillway wall is not allowed to move freely and is assumed to not slide.

CHECK OVERTURNING STABILITY

Moment Calculations

Horizontal force moment arms:

$$y_{Ia} := \left(El_{fill} - El_{bot}\right) \div 2 + t_{base} = 6.25\,ft$$

$$y_{Ib} := 0.5 \cdot t_{base} = 1.00\,ft$$

$$y_{I3} := y_{Ia} = 6.25\,ft$$

[Augusti: UCB GT 13-02]

$$y_{ea} := 0.4\left(El_{fill} - El_{bot}\right) + t_{base} = 5.40\,ft$$

$$y_{eb} := 0.4 \cdot t_{base} = 0.80\,ft$$

Summation of vertical forces:

$$V_{sum} := W_1 + W_2 + W_3 = 7.77 \cdot kip$$

Overturning Moment:

$$M_{o.2} := V_{F1a} \cdot y_{1a} + V_{F1b} \cdot y_{1b} \cdots + V_{Ea} \cdot y_{ea} + V_{Eb} \cdot y_{eb} \cdots + V_{Ia} \cdot y_{Ia} + V_{Ib} \cdot y_{Ib} + V_{I.3} \cdot y_{I3} = 31.99 \cdot kip \cdot ft$$

Resisting Moment:

$$M_r := W_1 \cdot x_1 + W_2 \cdot x_2 + W_3 \cdot x_3 + F_{F.T2} \cdot y_{FFT2} = 41.47 \cdot kip \cdot ft$$

Location of resultant:

$$x_{bar.2} := \frac{M_r - M_{o.2}}{V_{sum}} = 1.22\,ft$$

Eccentricity of the resultant:

$$ecc_2 := \frac{B}{2} - x_{bar.2} = 2.78\,ft$$

Overturning check:

$$Check_{ot.2} := \begin{cases} \text{"LC2 Overturning Stability OK"} & \text{if } 0 \le x_{bar.2} \le B \\ \text{"STOP - LC2 Overturning Stability Unacceptable"} & \text{otherwise} \end{cases}$$

$Check_{ot.2}$ = "LC2 Overturning Stability OK"

$B = 8.00\,ft$

2.) LOAD CASE 2 (Extreme Condition) Cont'd

LOAD CASE 2-Loading Conditions & Combinations:

Unfactored Shear At Base of Wall: $V_{LC2}(y) := V_{F.1}(y) + V_E(y) + V_I(y)$

Ultimate Shear At Base of Wall:

EM 1110-2-2104 Strength Load Combination: 1.0 EH + 1.0 Hs + 1.0 EQ *[Table E-9, Load Case 1A]*

$$V_{uLC2.1}(y) := V_{F.1}(y) + V_E(y) + V_I(y)$$

ASCE Load Combination 5: 1.2 (D + F) + 1.0 E + 1.6 H

$$V_{uLC2.2}(y) := \left(V_E(y) + V_I(y)\right) + 1.6 \cdot V_{F.1}(y)$$

Controlling Case: $V_{uLC2}(y) := \max\left(V_{uLC2.1}(y), V_{uLC2.2}(y)\right)$

$$V_{uLC2}(0ft) = 4.37 \cdot kip \qquad \boxed{V_{uLC2.2}(0ft) = 4.37 \cdot kip}$$

Unfactored Moment At Base of Wall: $M_{LC3}(y) := M_{F.1}(y) + M_E(y) + M_I(y)$

Ultimate Moment At Base of Wall:

EM 1110-2-2014 Strength Load Combination: 1.0 EH + 1.0 Hs + 1.0 (EQ)

$$M_{uLC2.1}(y) := M_{F.1}(y) + M_E(y) + M_I(y)$$

ASCE Load Combination 5: 1.2 (D + F) + 1.0 E + 1.6 H

$$M_{uLC2.2}(y) := \left(M_E(y) + M_I(y)\right) + 1.6 \cdot M_{F.1}(y)$$

Controlling Case: $M_{uLC2}(y) := \max\left(M_{uLC2.1}(y), M_{uLC2.2}(y)\right)$

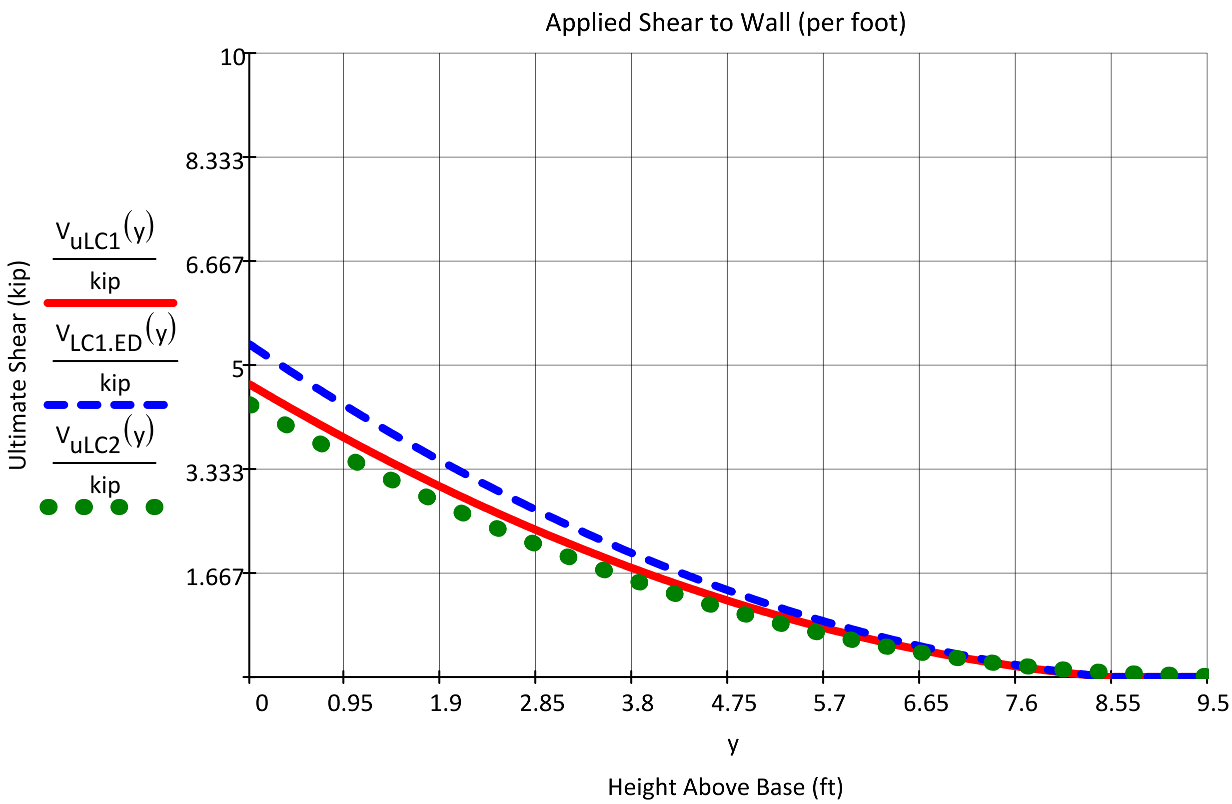
$$M_{uLC2}(0ft) = 13.70 \cdot kip \cdot ft \qquad \boxed{M_{uLC2.2}(0ft) = 13.70\, ft \cdot kip}$$

3.) Load Case Summary Analysis

Controlling Applied Load:

Ultimate Shear at Base of Wall: $V_u(y) := \max\left(V_{uLC1}(y), V_{LC1.ED}(y), V_{uLC2}(y)\right)$

$V_u(0ft) = 5.33 \cdot kip$ $V_{LC1.ED}(0ft) = 5.33 \cdot kip$

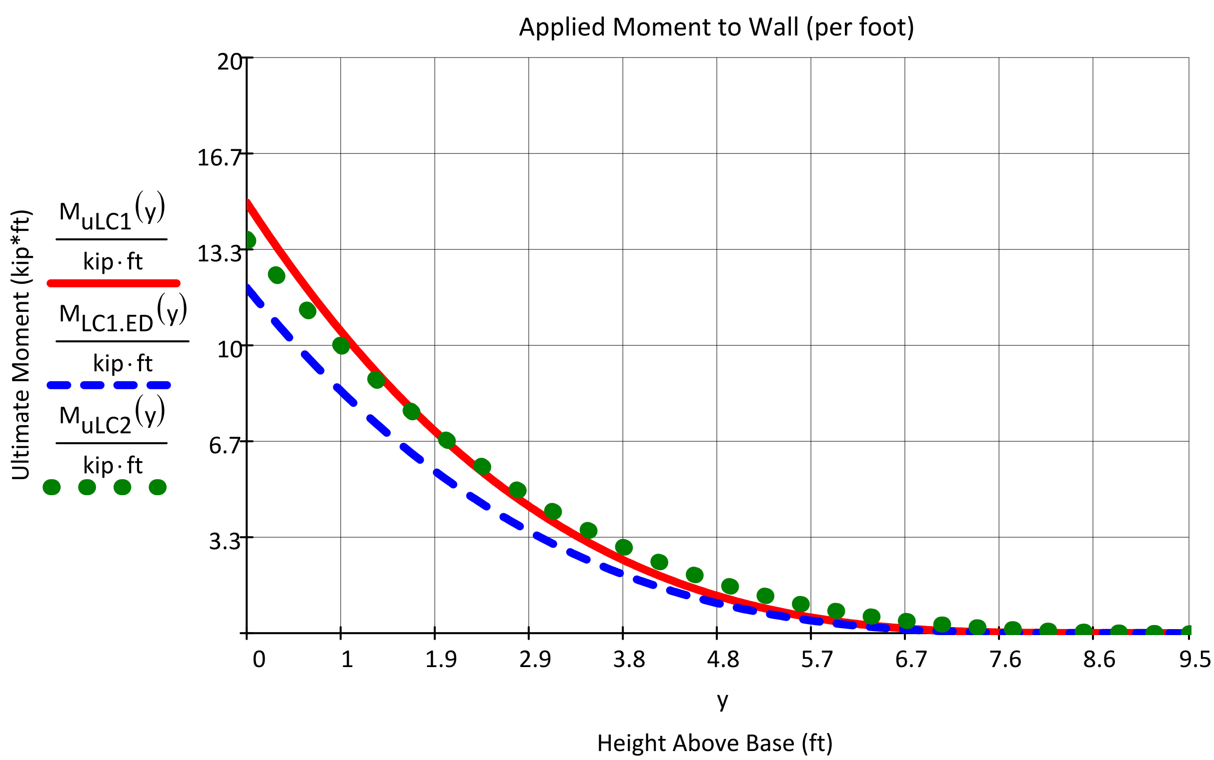


3.) Load Case Summary Analysis Cont'd

Controlling Applied Load:

Ultimate Moment at Base of Wall: $M_u(y) := \max\left(M_{uLC1}(y), M_{LC1.ED}(y), M_{uLC2}(y)\right)$

$M_u(0ft) = 14.96 \cdot kip \cdot ft$ $\quad M_{LC1.ED}(0ft) = 12.00 \cdot kip \cdot ft$



4.) Wall Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05 \cdot \dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Horizontal Reinforcement:

Bar Size and Spacing: (Channel & Embankment) $Size_{SH} := 5$ $sSH := 12in$

Reinforcement Clear Cover: $clrS := 3in$

Diameter & Cross-Sectional Area of Bars: $dbSH := d_{b_{Size_{SH}}} = 0.63 \cdot in$ $AbSH := A_{b_{Size_{SH}}} = 0.31 \cdot in^2$

Area of Reinforcement:

$$A_{sSH} := AbSH \cdot \frac{12in}{sSH} = 0.31 \cdot in^2$$

$$m := \left(t_{wall.T} - t_{wall.B}\right) \div H_{wall} = -0.05$$

Section Depth at a Height 'y' from the base: $t_{wall}(y) := t_{wall.B} + m \cdot y$

Depth to Centroid of Embankment Reinf: $d_{SH}(y) := t_{wall}(y) - clrS - 0.5 \cdot dbSH$

Reinforcement Ratio: $\rho_{SH}(y) := A_{sSH} \div \left(b_w\, d_{SH}(y)\right)$

4.) Wall Concrete & Rebar Strength Design Cont'd

Vertical Reinforcement:

Bar Size and Spacing: (Embankment) $Size_{SE} := 7$ $sSE := 12in$

Diameter & Cross-Sectional Area of Bars: $dbSE := d_{b_{Size_{SE}}} = 0.88 \cdot in$ $AbSE := A_{b_{Size_{SE}}} = 0.60 \cdot in^2$

Area of Tension Reinforcement: $A_{sSE} := AbSE \cdot \dfrac{12in}{sSE} = 0.60 \cdot in^2$

$m := (t_{wall.T} - t_{wall.B}) \div H_{wall} = -0.05$

Section Depth at a Height 'y' from the base: $t_{wall}(y) := t_{wall.B} + m \cdot y$

Depth to Centroid of Reinforcement: $d_{SE}(y) := t_{wall}(y) - clrS - dbSH - 0.5 \cdot dbSE$

Reinforcement Ratio: $\rho_{SE}(y) := A_{sSE} \div (b_w d_{SE}(y))$

Bar Size and Spacing: (Channel) $Size_{SC} := 7$ $sSC := 12in$

Diameter & Cross-Sectional Area of Bars: $dbSC := d_{b_{Size_{SC}}} = 0.88 \cdot in$ $AbSC := A_{b_{Size_{SC}}} = 0.60 \cdot in^2$

Area of Tension Reinforcement: $A_{sSC} := AbSC \cdot \dfrac{12in}{sSC} = 0.60 \cdot in^2$

Depth to Centroid of Reinforcement: $d_{SC}(y) := t_{wall}(y) - clrS - dbSH - 0.5 \cdot dbSC$

Reinforcement Ratio: $\rho_{SC}(y) := A_{sSC} \div (b_w d_{SC}(y))$

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GEI Consultants

Project No.	1801834	Document No.	N/A
Subject	Design of Spillway - Inclined Portion		

5.) Wall Strength Capacity

Wall Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
ACI 318-14, §22.5.5.1)

$$V_c(y) := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot \min\left(d_{SE}(y), d_{SC}(y)\right)$$

$$V_c(0ft) = 22.44 \cdot kip$$

Design Shear Capacity:

$$\phi V_n(y) := \phi_v \cdot \left(V_c(y)\right)$$

$$\phi V_n(0ft) = 16.83 \cdot kip$$

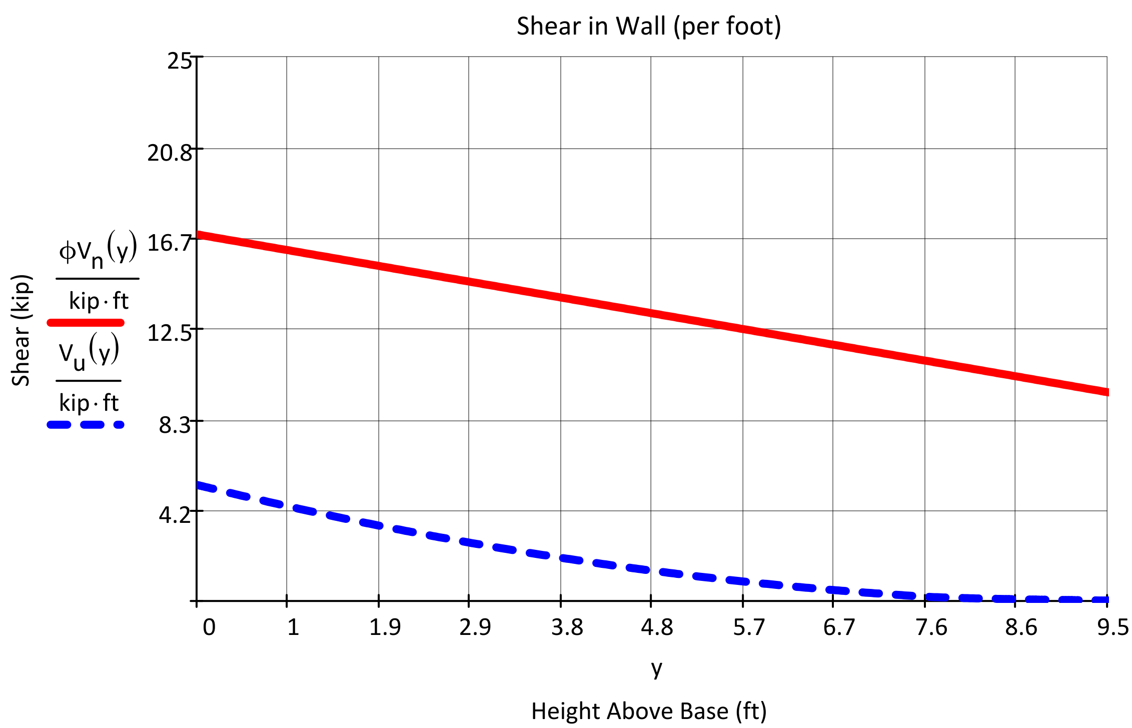
$$V_u(0ft) = 5.33 \cdot kip$$

Demand Capacity Ratio:

$$DCR_V(y) := \frac{V_u(y)}{\phi V_n(y)}$$

$$DCR_V(0ft) = 0.32$$

< 1.0 therefore section is okay for shear strength



5.) Wall Strength Capacity Cont'd

Wall Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(ACI 318-14)

$$M_n(y) := \rho_{SE}(y) \cdot f_y \cdot b_w \cdot d_{SE}(y)^2 \cdot \left(1 - 0.59 \cdot \rho_{SE}(y) \cdot \frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n(y) := \phi_t \cdot M_n(y)$$

$$\phi M_n(0) = 36.57\,\text{ft} \cdot \text{kip}$$

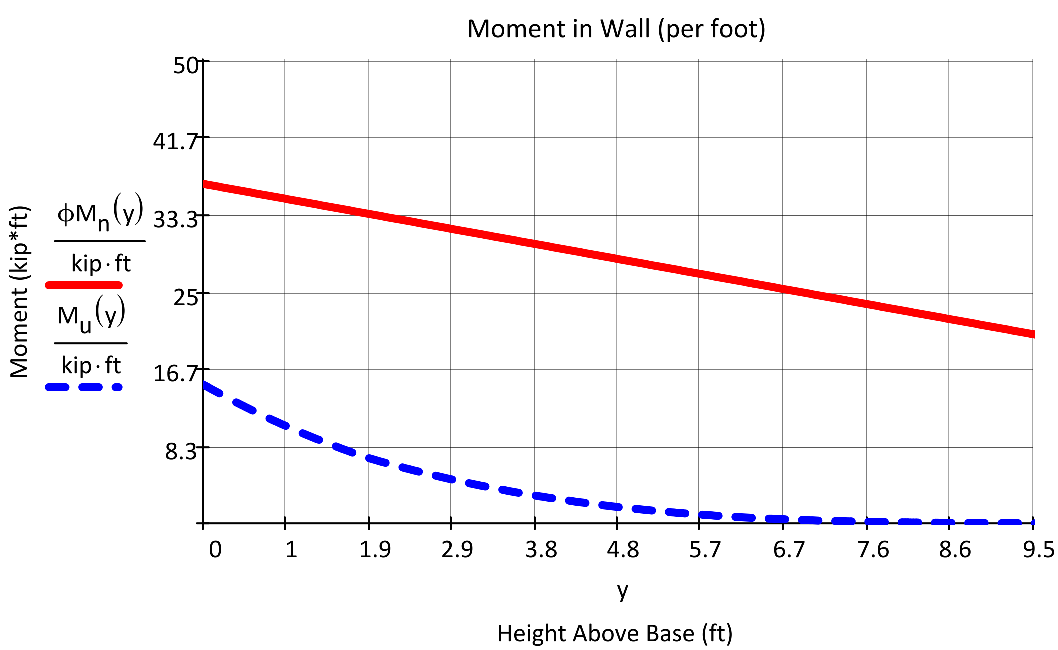
$$M_u(0) = 14.96 \cdot \text{kip} \cdot \text{ft}$$

Demand Capacity Ratio:

$$DCR_M(y) := \frac{M_u(y)}{\phi M_n(y)}$$

$$DCR_M(0ft) = 0.41$$

< 1.0 therefore section is okay for flexural strength



6.) Wall Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in,\ 5\cdot 0.5\left(t_{wall.B} + t_{wall.T}\right),\ 12in\right] = 12.00\cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{cases} 0.0020 & \text{if } f_y < 60ksi \\ \max\left[\left(0.0018\cdot 60ksi \div f_y\right),\ 0.0014\right] & \text{otherwise} \end{cases} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0030$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI},\ \rho_{USACE}\right) = 0.0030$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min} := \rho_{max}\cdot b_w\cdot 0.5\cdot\left(t_{wall.B} + t_{wall.T}\right) = 0.54\cdot in^2$$

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Vertical Reinforcement:

Bar Size and Spacing of Reinforcement: (Embankment) — $Size_{SE} = 7.00$ — $sSE = 12.00\cdot in$

Diameter & Cross-Sectional Area of Bars: — $dbSE = 0.88\cdot in$ — $AbSE = 0.60\cdot in^2$

Area of Reinforcement: — $A_{sSE} = 0.60\cdot in^2$

Bar Size and Spacing of Reinforcement: (Channel) — $Size_{SC} = 7.00$ — $sSC = 12.00\cdot in$

Diameter & Cross-Sectional Area of Bars: — $dbSC = 0.88\cdot in$ — $AbSC = 0.60\cdot in^2$

Area of Reinforcement: — $A_{sSC} = 0.60\cdot in^2$

$$\boxed{\frac{A_{s.min}}{A_{sSC} + A_{sSE}} = 0.45}$$

< 1.0 therefore okay

6.) Wall Rebar Design Cont'd

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Horizontal Reinforcement:

Wall is reinforced with two sections. The top 20-feet from the top of the wall use reinforcing at half the spacing of that after 20-feet.

Bar Size and Spacing of Reinforcement: (Embankment & Channel) $\quad Size_{SH} = 5.00 \quad sSH = 12.00 \cdot in$

Diameter & Cross-Sectional Area of Bars: $\quad dbSH = 0.63 \cdot in \quad AbSH = 0.31 \cdot in^2$

Area of Reinforcement: $\quad A_{sSH} = 0.31 \cdot in^2$

$$\boxed{\frac{A_{s.min}}{2 \cdot A_{sSH}} = 0.87}$$

< 1.0 therefore okay

Minimum Flexural Steel: (ACI 318-14, §9.6.1.2)

Vertical Reinforcement:

$$A_{sV.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot \min\left(d_{SE}(0ft), d_{SC}(0ft)\right), \frac{200 \cdot psi \cdot b_w \cdot \min\left(d_{SE}(0ft), d_{SC}(0ft)\right)}{f_y}\right) = 0.56 \cdot in^2$$

$$\boxed{\frac{A_{sV.min}}{A_{sSE}} = 0.93}$$

< 1.0 therefore okay

6.) Wall Rebar Design Cont'd

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Reinforcement Ratio Provided:
(Vertical)

Embankment: $$\rho_{pVE} := \frac{A_{sSE}}{d_{SE}(0ft)\cdot b_w} = 0.0036$$

Channel: $$\rho_{pVC} := \frac{A_{sSE}}{d_{SE}(0ft)\cdot b_w} = 0.0036$$

Reinforcement Ratio Provided:
(Horizontal)

$$\rho_{pH} := \frac{A_{sSH}}{d_{SH}(0ft)\cdot b_w} = 0.0018$$

Depth to Neutral Axis:
(Vertical)

$$cVE := \frac{\rho_{pVE}\cdot f_y\cdot d_{SE}(0ft)}{0.85\cdot\beta_1\cdot f'_c} = 0.95\cdot in$$

$$cVC := \frac{\rho_{pVC}\cdot f_y\cdot d_{SC}(0ft)}{0.85\cdot\beta_1\cdot f'_c} = 0.95\cdot in$$

Depth to Neutral Axis:
(Horizontal)

$$cH := \frac{\rho_{pH}\cdot f_y\cdot d_{SH}(0ft)}{0.85\cdot\beta_1\cdot f'_c} = 0.49\cdot in$$

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Calculated Strain in Tension Steel at Nominal Strength:
(Vertical)

$$\varepsilon_{tVE} := 0.003\cdot\left(\frac{d_{SE}(0ft)}{cVE} - 1\right) = 0.04 \qquad \varepsilon_{tVC} := 0.003\cdot\left(\frac{d_{SC}(0ft)}{cVC} - 1\right) = 0.04$$

(Horizontal)

$$\varepsilon_{tH} := 0.003\cdot\left(\frac{d_{SH}(0ft)}{cH} - 1\right) = 0.09$$

$$\boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tVE}} = 0.10} \quad \boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tVC}} = 0.10} \quad \boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tH}} = 0.05}$$

< 1.0 therefore okay

7.) Footing Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1)*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05 \cdot \dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Reinforcement Parallel to Wall:

Heel Reinforcement:

Bar Size and Spacing (Top): $Size_{HTP} := 7$ $sHTP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbHTP := d_{b_{Size_{HTP}}} = 0.88 \cdot in$

$AbHTP := A_{b_{Size_{HTP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sHTP} := AbHTP \cdot \dfrac{12in}{sHTP} = 0.60 \cdot in^2$

Depth to Centroid of Tension Reinforcement: $d_{HTP} := t_{base} - 0.5 \cdot dbHTP = 1.96\,ft$

Reinforcement Ratio: $\rho_{HTP} := A_{sHTP} \div \left(b_w d_{HTP}\right) = 0.00212$

Bar Size and Spacing (Bottom): $Size_{HBP} := 7$ $sHBP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbHBP := d_{b_{Size_{HBP}}} = 0.88 \cdot in$

$AbHBP := A_{b_{Size_{HBP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sHBP} := AbHBP \cdot \dfrac{12in}{sHBP} = 0.60 \cdot in^2$

7.) Footing Concrete & Rebar Strength Design Cont'd

Reinforcement Parallel to Wall:

Toe Reinforcement:

Bar Size and Spacing (Top): $Size_{TTP} := 7$ $sTTP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbTTP := d_{b_{Size_{TTP}}} = 0.88 \cdot in$

$AbTTP := A_{b_{Size_{TTP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sTTP} := AbTTP \cdot \frac{12in}{sTTP} = 0.60 \cdot in^2$

Bar Size and Spacing (Bottom): $Size_{TBP} := 7$ $sTBP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbTBP := d_{b_{Size_{TBP}}} = 0.88 \cdot in$

$AbTBP := A_{b_{Size_{TBP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sTBP} := AbTBP \cdot \frac{12in}{sTBP}$

Reinforcement Perpendicular to Wall:

Heel Reinforcement:

Bar Size and Spacing (Top): $Size_{HT} := 9$ $sHT := 12in$

Reinforcement Clear Cover: $clrHT := 3in$

Diameter & Cross-Sectional Area of Bars: $dbHT := d_{b_{Size_{HT}}} = 1.13 \cdot in$ $AbHT := A_{b_{Size_{HT}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sHT} := AbHT \cdot \frac{12in}{sHT} = 1.00 \cdot in^2$

Section Depth: $t_{foot} := t_{base} = 2.00\,ft$

Depth to Centroid of Tension Reinforcement: $d_{HT} := t_{foot} - clrHT - (0.5 \cdot dbHT) = 1.70\,ft$

Reinforcement Ratio: $\rho_{HT} := A_{sHT} \div \left(b_w\, d_{HT}\right) = 0.00408$

7.) Footing Concrete & Rebar Strength Design Cont'd

Reinforcement Perpendicular to Wall:

Heel Reinforcement:

Bar Size and Spacing (Bottom): $Size_{HB} := 9$ $sHB := 12in$

Reinforcement Clear Cover: $clrHB := 4in$

Diameter & Cross-Sectional Area of Bars: $dbHB := d_{b_{Size_{HB}}} = 1.13 \cdot in$ $AbHB := A_{b_{Size_{HB}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sHB} := AbHB \cdot \frac{12in}{sHB} = 1.00 \cdot in^2$

Depth to Centroid of Compression Reinforcement: $d_{HB} := t_{foot} - clrHB - 0.5 \cdot dbHB = 1.62\,ft$

Toe Reinforcement:

Bar Size and Spacing (Top): $Size_{TT} := 9$ $sTT := 12in$

Reinforcement Clear Cover: $clrTT := 3in$

Diameter & Cross-Sectional Area of Bars: $dbTT := d_{b_{Size_{TT}}} = 1.13 \cdot in$ $AbTT := A_{b_{Size_{TT}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sTT} := AbTT \cdot \frac{12in}{sTT} = 1.00 \cdot in^2$

Depth to Centroid of Tension Reinforcement: $d_{TT} := t_{foot} - clrTT - (0.5 \cdot dbTT) = 1.70\,ft$

Reinforcement Ratio: $\rho_{TT} := A_{sTT} \div (b_w d_{TT}) = 0.00408$

Bar Size and Spacing (Bottom): $Size_{TB} := 9$ $sTB := 12in$

Reinforcement Clear Cover: $clrTB := 4in$

Diameter & Cross-Sectional Area of Bars: $dbTB := d_{b_{Size_{TB}}} = 1.13 \cdot in$ $AbTB := A_{b_{Size_{TB}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sTB} := AbTB \cdot \frac{12in}{sTB} = 1.00 \cdot in^2$

Depth to Centroid of Compression Reinforcement: $d_{TB} := t_{foot} - clrTB - 0.5 \cdot dbTB = 1.62\,ft$

Reinforcement Ratio: $\rho_{TB} := A_{sTB} \div (b_w d_{TB}) = 0.00429$

8.) Footing Strength Capacity

Bearing Pressure:

Maximum Eccentricity: $ecc := \max(ecc_1, ecc_2) = 2.78\,ft$

Unfactored Shear: $V := (W_1 + W_2 + W_3) = 7.77 \cdot kip$

Factored Shear:
[ASCE 7-10, 2.3.2 Eqtn 2:
DL & pressure of bulk materials factor]

$V_u := 1.2 \cdot (W_1 + W_2) + 1.6 \cdot W_3 = 10.76 \cdot kip$

Unfactored bearing pressure at heel:

$$q_{heel.ED} := \frac{V}{B \cdot b_w} \cdot \left[1 - \frac{(6 \cdot ecc_1)}{B}\right] = 0.81 \cdot \frac{kip}{ft^2}$$

Unfactored bearing pressure at toe:

$$q_{toe.ED} := \frac{V}{B \cdot b_w} \cdot \left[1 + \frac{(6 \cdot ecc_1)}{B}\right] = 1.13 \cdot \frac{kip}{ft^2}$$

Factored bearing pressure at heel:

$$q_{u.heel} := \frac{V_u}{B \cdot b_w} \cdot \left[1 - \frac{(6 \cdot ecc)}{B}\right] = -1.46 \cdot \frac{kip}{ft^2}$$

Factored bearing pressure at toe:

$$q_{u.toe} := \frac{V_u}{B \cdot b_w} \cdot \left[1 + \frac{(6 \cdot ecc)}{B}\right] = 4.15 \cdot \frac{kip}{ft^2}$$

$$q_{u.max} := \max(q_{u.heel}, q_{u.toe}) = 4.15 \cdot ksf$$

8.) Footing Strength Capacity Cont'd

Note: The heel slab is designed as a cantilever beam with downward pressure from soil + slab and upward pressure from bearing. The toe slab is designed as a cantilever beam with downward pressure from slab and upward pressure from bearing.

Net Forces on Footings:

Downward weight of heel slab:
(unfactored)

$$W_{heel} := \gamma_{conc} \cdot t_{base} \cdot 1ft = 0.30 \cdot \frac{kip}{ft}$$

Downward weight of toe slab:
(unfactored)

$$W_{toe} := \gamma_{conc} \cdot t_{base} \cdot 1ft = 0.30 \cdot \frac{kip}{ft}$$

Downward weight of fill over heel slab:
(unfactored)

$$W_{heelfill} := \gamma_f \cdot \left(El_{fill} - El_{bot}\right) \cdot 1ft = 1.11 \cdot \frac{kip}{ft}$$

Downward weight of heel slab:

$$W_{u.heel} := 1.2W_{heel} = 0.36 \cdot \frac{kip}{ft}$$

Downward weight of toe slab:

$$W_{u.toe} := 1.2W_{toe} = 0.36 \cdot \frac{kip}{ft}$$

Downward weight of fill over heel slab:

$$W_{u.heelfill} := 1.6W_{heelfill} = 1.77 \cdot \frac{kip}{ft}$$

Net downward force on heel (Unfactored-Load Case 1):

$$p_{h.ED1} := W_{heel} + W_{heelfill} - b_w \cdot \left[q_{heel.ED} + 0.5 \cdot \left(\frac{q_{toe.ED} - q_{heel.ED}}{B}\right) \cdot \left(Heel - \frac{d_{HT}}{2}\right)\right] = 0.55 \cdot \frac{kip}{ft}$$

Tension on Top

Net downward force on toe (Unfactored-Load Case 1):

$$p_{h.ED2} := W_{toe} - b_w \cdot \left[q_{toe.ED} + 0.5 \cdot \left(\frac{q_{heel.ED} - q_{toe.ED}}{B}\right) \cdot \left(Toe - \frac{d_{TT}}{2}\right)\right] = -0.78 \cdot \frac{kip}{ft}$$

Tension on Bottom

Net downward force on heel (Factored-Max):

$$p_h := W_{u.heel} + W_{u.heelfill} - b_w \cdot \left[q_{u.heel} + 0.5 \cdot \left(\frac{q_{u.toe} - q_{u.heel}}{B}\right) \cdot \left(Heel - \frac{d_{HT}}{2}\right)\right] = 2.83 \cdot \frac{kip}{ft}$$

Tension on Top

Net downward force on toe (Factored-Max):

$$p_{h2} := W_{u.toe} - b_w \cdot \left[q_{u.toe} + 0.5 \cdot \left(\frac{q_{u.heel} - q_{u.toe}}{B}\right) \cdot \left(Toe - \frac{d_{TT}}{2}\right)\right] = -2.86 \cdot \frac{kip}{ft}$$

Tension on Bottom

8.) Footing Strength Capacity Cont'd

Environmental Durability Factor (ACI 350-06)

Note: Required for Tension Controlled structure to encourage durability and liquid-tightness.
Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads. The durability factor is applied to service loads only.

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4

$f_{s.ss} := 24000psi$

$$S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$$

Flexure in normal conditions: R10.6.4
- One way element, bar ar 6" spacing

$f_{s.f} := 34000psi$

$$S_{d.f} := \max\left(\frac{f_y}{f_{s.f}}, 1.0\right) = 1.76$$

EDF Shear:

$$V_{heel.ED} := S_{d.ss} \left|p_{h.ED1}\right| \cdot Heel = 4.12 \cdot kip$$

$$V_{toe.ED} := S_{d.ss} \left|p_{h.ED2}\right| \cdot Toe = 6.80 \cdot kip$$

EDF Moment:

$$M_{heel.ED} := S_{d.f} \left|p_{h.ED1}\right| \cdot Heel^2 \div 2 = 4.36 \cdot kip \cdot ft$$

$$M_{toe.ED} := S_{d.f} \left|p_{h.ED2}\right| \cdot Toe^2 \div 2 = 8.40 \cdot kip \cdot ft$$

Ultimate Shear-Cantilevered Section:

$$V_{u.heel} := \left|p_h\right| \cdot Heel = 8.50 \cdot kip$$

$$V_{u.toe} := \left|p_{h2}\right| \cdot Toe = 10.02 \cdot kip$$

Ultimate Moment-Cantilevered Section:

$$M_{u.heel} := \left|p_h\right| \cdot Heel^2 \div 2 = 12.76 \cdot kip \cdot ft$$

$$M_{u.toe} := \left|p_{h2}\right| \cdot Toe^2 \div 2 = 17.53 \cdot kip \cdot ft$$

8.) Footing Strength Capacity Cont'd

Footing Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
(*ACI 318-14, §22.5.5.1)*

$$V_c := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{HT} = 32.90 \cdot kip$$

Design Shear Capacity:

$$\phi V_n := \phi_v \cdot (V_c) = 24.68 \cdot kip$$

$V_{u.heel} = 8.50 \cdot kip$ $\quad V_{heel.ED} = 4.12 \cdot kip$

Demand Capacity Ratio-Heel:

$$DCR_H := \frac{\max(V_{u.heel}, V_{heel.ED})}{\phi V_n} = 0.34$$

< 1.0 therefore section is okay for shear strength

$V_{u.toe} = 10.02 \cdot kip$ $\quad V_{toe.ED} = 6.80 \cdot kip$

Demand Capacity Ratio-Toe:

$$DCR_{F.T} := \frac{\max(V_{u.toe}, V_{toe.ED})}{\phi V_n} = 0.41$$

< 1.0 therefore section is okay for shear strength

Footing Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(*ACI 318-14)*

$$M_n := \rho_{HT} \cdot f_y \cdot b_w \cdot d_{HT}^{\,2} \cdot \left(1 - 0.59 \cdot \rho_{HT} \cdot \frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n := \phi_t \cdot M_n = 89.01\, ft \cdot kip$$

$M_{u.heel} = 12.76\, ft \cdot kip$ $\quad M_{heel.ED} = 4.36\, ft \cdot kip$

Demand Capacity Ratio-Heel:

$$\boxed{DCR_H := \frac{\max(M_{u.heel}, M_{heel.ED})}{\phi M_n} = 0.14}$$

< 1.0 therefore section is okay for flexural strength

$M_{u.toe} = 17.53\, ft \cdot kip$ $\quad M_{toe.ED} = 8.40\, ft \cdot kip$

Demand Capacity Ratio-Toe:

$$\boxed{DCR_T := \frac{\max(M_{u.toe}, M_{toe.ED})}{\phi M_n} = 0.20}$$

< 1.0 therefore section is okay for flexural strength

9.) Footing Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in, 5 \cdot 0.5\left(t_{wall.B} + t_{wall.T}\right), 12in\right] = 12.00 \cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{cases} 0.0020 & \text{if } f_y < 60ksi \\ \max\left[\left(0.0018 \cdot 60ksi \div f_y\right), 0.0014\right] & \text{otherwise} \end{cases} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0030$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0030$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min.toe} := \rho_{max} \cdot b_w \cdot t_{base} = 0.86 \cdot in^2$$

$$A_{s.min.heel} := \rho_{max} \cdot b_w \cdot t_{base} = 0.86 \cdot in^2$$

Reinforcement perpendicular to Wall:

Bar Size and Spacing of Reinforcement:

$Size_{TT} = 9.00$ $sTT = 1.00\,ft$

$Size_{TB} = 9.00$ $sTB = 1.00\,ft$

Area of Reinforcement:

$A_{sTT} = 1.00 \cdot in^2$ $A_{sTB} = 1.00 \cdot in^2$

$$\boxed{\frac{A_{s.min.toe}}{A_{sTT} + A_{sTB}} = 0.43}$$

< 1.0 therefore okay

Bar Size and Spacing of Reinforcement:

$Size_{HT} = 9.00$ $sHT = 1.00\,ft$

$Size_{HB} = 9.00$ $sHB = 1.00\,ft$

Area of Reinforcement:

$A_{sHT} = 1.00 \cdot in^2$ $A_{sHB} = 1.00 \cdot in^2$

$$\boxed{\frac{A_{s.min.heel}}{A_{sHT} + A_{sHB}} = 0.43}$$

< 1.0 therefore okay

9.) Footing Rebar Design Cont'd

Reinforcement parallel to Wall:

Toe:

Bar Size and Spacing of Reinforcement: $Size_{TTP} = 7.00$ $sTTP = 1.00\,ft$

$Size_{TBP} = 7.00$ $sTBP = 1.00\,ft$

Area of Reinforcement: $A_{sTTP} = 0.60 \cdot in^2$ $A_{sTBP} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min.toe}}{A_{sTTP} + A_{sTBP}} = 0.72}$$ < 1.0 therefore okay

Heel:

Bar Size and Spacing of Reinforcement: $Size_{HTP} = 7.00$ $sHTP = 1.00\,ft$

$Size_{HBP} = 7.00$ $sHBP = 1.00\,ft$

Area of Reinforcement: $A_{sHTP} = 0.60 \cdot in^2$ $A_{sHBP} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min.heel}}{A_{sHTP} + A_{sHBP}} = 0.72}$$ < 1.0 therefore okay

Minimum Flexural Steel: (ACI 318-14, §9.6.1.2)

Perpendicular to Wall

Heel:

$$A_{s.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d_{HT}, \frac{200 \cdot psi \cdot b_w \cdot d_{HT}}{f_y}\right) = 0.82 \cdot in^2$$

Toe:

$$A_{s.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d_{TB}, \frac{200 \cdot psi \cdot b_w \cdot d_{TB}}{f_y}\right) = 0.78 \cdot in^2$$

$$\boxed{\frac{A_{s.min}}{A_{sHT}} = 0.78} \quad \boxed{\frac{A_{s.min}}{A_{sTB}} = 0.78}$$ < 1.0 therefore okay

9.) Footing Rebar Design Cont'd

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Perpendicular to Wall

Reinforcement Ratio Provided: $$\rho_p := \frac{AbHT}{d_{HT} \cdot b_w} = 0.0041$$

Depth to Neutral Axis: $$c := \frac{\rho_p \cdot f_y \cdot d_{HT}}{0.85 \cdot \beta_1 \cdot f'_c} = 1.58 \cdot in$$

Calculated Strain in Tension Steel at Nominal Strength: $$\varepsilon_t := 0.003 \cdot \left(\frac{d_{HT}}{c} - 1\right) = 0.04$$

$$\boxed{\frac{\varepsilon_{t.min}}{\varepsilon_t} = 0.11}$$

< 1.0 Therefore Okay

Spillway Slab (Stilling Basin) - Upper Black Creek Reservoir

<u>Problem statement</u>: Check shear and moment capacity across the slab. Assume the slab is pinned at the two longitudinal joints spaced at 25 ft apart.

References

-ACI 318-14, Building Code Requirements for Structural Concrete
-EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures

Constants

Unit Weight of Water: $\gamma_w := 62.4\text{pcf}$ Unit Weight of Concrete: $\gamma_{conc} := 150\text{pcf}$

Specificied Compressive Strength of Concrete: $f'_c := 4500\text{psi}$

Specified Yield Strenght of Reinforcement: $f_y := 60\text{ksi}$

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *ACI 318-14, §21.2.1*

Strength Reduction Factor of Moment: $\phi_m := 0.9$ *ACI 318-14, §21.2.1*

Load Factor for Strength Design: $U_1 := 1.6$ *EM 1110-2-2104, Table E-9, Load Case 1B*

Dimensions & Reinforcement

Elevation of PWS during IDF: $EL_{PWS} := 8748\text{ft}$

Top of Slab Elevation: $TOS := 8745\text{ft}$

Thickness of Slab $t_{slab} := 12\text{in}$

Assume 1' wide strip of slab: $b_w := 12\text{in}$

Maximum distance between control joints: $L_{CJ} := 31\text{ft}$

Distance across slab: $Dist := 5\text{ft}$

Concrete Cover: $Cov := 4\text{in}$
Assume Rebar Spacing: $sp := 12\text{in}$

Assume a Bar Size: $Barsize := 7$ $Bar := d_{b_{Barsize}} = 0.88\cdot\text{in}$ $A_b := A_{b_{Barsize}} = 0.60\cdot\text{in}^2$

	Client	Blue Lake Reservoir Company			Page	80
GEI Consultants	Project	Upper Black Creek Reservoir			Pg. Rev.	
	By	C. Diebold	Chk.	M. Provencher	App.	C. Masching
	Date	11/26/2019	Date	11/26/2020	Date	11/26/2020

Project No.	1801834	Document No.	N/A
Subject	Design of Spillway - Inclined Portion		

Calculations

Height of Water Column for uplift: $H_{max} := EL_{PWS} - TOS + t_{slab} = 4.00\,ft$

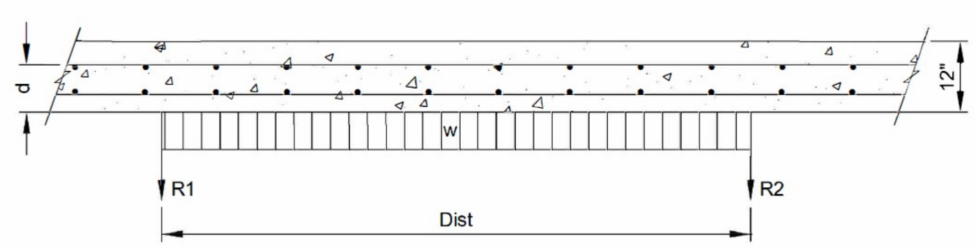
Max case = IDF conditions, vertical seismic does not control

Hydrostratic uplift per unit slab width: $p_{max} := H_{max} \cdot b_w \cdot \gamma_w = 0.25 \cdot klf$

Dead Load of slab per unit slab width: $w_d := t_{slab} \cdot b_w \cdot \gamma_{conc} = 0.15 \cdot klf$

Depth to Centroid of Reinforcement: $d_{rebar} := t_{slab} - Cov - Bar - \frac{Bar}{2} = 6.69 \cdot in$

Free Body Diagram of Slab Across Spillway



Calculate Reaction Forces R1 and R2

Sum Forces Vertically to find reaction anchor Loads R1 and R2 $\left(p_{max} - w_d\right) \cdot Dist - R_1 - R_2 = ■ \cdot 0$

Reaction Force 1: $R_1 := \frac{\left[\left(p_{max} - w_d\right) \cdot Dist\right]}{2}$ $R_1 = 0.25 \cdot kip$

Reaction Force 1: $R_2 := R_1$ $R_2 = 0.25 \cdot kip$

Model Slab as a simple beam and define locations along x-axis for Shear & Moment Diagrams

Beginning of Beam: $x_{min} := 0ft$

End of Beam: $x_{max} := Dist = 5.00\,ft$

Location of Reaction Anchor 1: $x_a := x_{min} = 0.00 \cdot ft$

Location of Reaction Anchor 2: $x_b := x_{max} = 5.00\,ft$

Define Range Variable "x" and "Step Size" of Range variable

Vector Step Size: $d_t := 0.01\text{ft}$

Define x: $x := x_{min}, (x_{min} + d_t) .. x_{max}$

Define Shear Loading for each Condition along Beam as a function of x

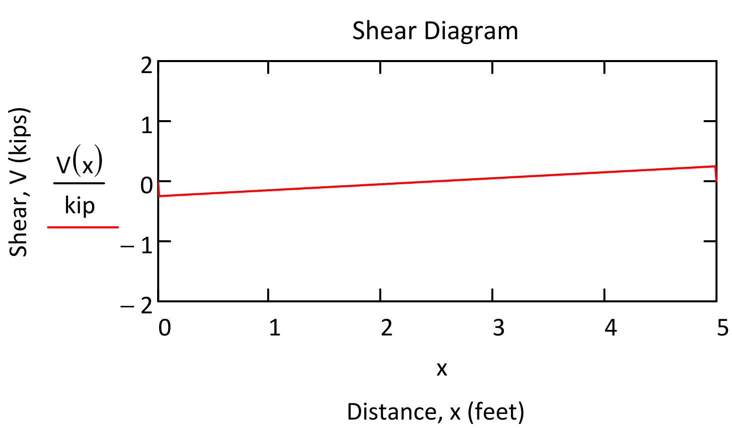
From x_{min} to x_a: $v_1(x) := (p_{max} - w_d) \cdot x$

From x_a to x_b: $v_2(x) := (p_{max} - w_d) \cdot x - R_1$

From x_b to x_{max}: $v_3(x) := (p_{max} - w_d) \cdot x - R_1 - R_2$

Conditional Shear Eqn:

$$V(x) := \begin{cases} v_1(x) & \text{if } x_{min} \le x \le x_a \\ v_2(x) & \text{if } x_a < x \le x_b \\ v_3(x) & \text{if } x_b < x \le x_{max} \\ 0 & \text{otherwise} \end{cases}$$



Check for Max Shear

$\text{Check1} := V(x_a + d_t) = -0.25 \cdot \text{kip}$

$\text{Check2} := V(x_a - d_t) = 0.00 \cdot \text{kip}$

$\text{Check3} := V(x_a + \text{Dist} - d_t) = 0.25 \cdot \text{kip}$

$\text{Check4} := V(x_a + \text{Dist} + d_t) = 0.00 \cdot \text{kip}$

$V_{max} := \max(\text{Check1}, \text{Check2}, \text{Check3}, \text{Check4})$

$\boxed{V_{max} = 0.25 \cdot \text{kip}}$

Define Moment for each Condition along Beam as a function of x

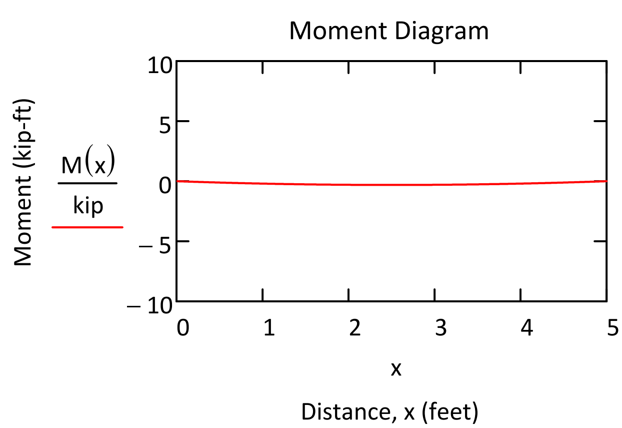
From x_{min} to x_a: $$m_1(x) := (p_{max} - w_d) \cdot x \cdot \frac{1}{2} x$$

From x_a to x_b: $$m_2(x) := (p_{max} - w_d) \cdot x \cdot \frac{1}{2} x - R_1 \cdot (x - x_a)$$

From x_b to x_{max}: $$m_3(x) := (p_{max} - w_d) \cdot x \cdot \frac{1}{2} x - R_1 \cdot (x - x_a) - R_2 \cdot (x - x_a - Dist)$$

Calculate Max Moment and Plot Moment Diagram

Conditional Moment Eqn: $$M(x) := \begin{cases} m_1(x) & \text{if } x_{min} \le x \le x_a \\ m_2(x) & \text{if } x_a < x \le x_b \\ m_3(x) & \text{if } x_b < x \le x_{max} \\ 0 & \text{otherwise} \end{cases}$$



$$Check1 := M\left(\frac{Dist}{2}\right) = -0.31\, ft \cdot kip$$

$$M_{max} := |Check1| = 0.31\, ft \cdot kip$$

Controlling Applied Load:

Environmental Durability Factor (ACI 350-06):

Required for Tension Controlled structure to encourage durability and liquid-tightness.
Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads.

Specified Yield Strength of Reinforcement: $f_y = 60 \cdot ksi$

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4 $f_{s.ss} := 24000psi$

$$S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$$

Flexure in normal conditions: R10.6.4
- One way element, bar ar 12" spacing

$f_{s.f.12} := 21000psi$

$$S_{d.f.12} := \max\left(\frac{f_y}{f_{s.f.12}}, 1.0\right) = 2.86$$

Check Shear Capacity Across Slab

Factored shear load: $V_u := \max\left(U_1 \cdot V_{max}, S_{d.ss} \cdot V_{max}\right) = 0.62 \cdot kip$

Allowable Shear Load: $V_{all} := \left(\phi_v \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{rebar}\right) = 8.1\, ft \cdot \frac{kip}{ft}$ *ACI 318-14, §22.5.5.1*

Find the Demand/Capacity Ratio:

$$\boxed{DC_{vratio} := \frac{V_u}{V_{all}} = 0.08}$$

$$check := \begin{vmatrix} \text{"Shear Capacity > Demand, Shear Design Okay"} & \text{if } V_u < V_{all} \\ \text{"Shear Capacity < Demand, Redesign"} & \text{if } V_u > V_{all} \end{vmatrix}$$

check = "Shear Capacity > Demand, Shear Design Okay"

Check Moment Capacity Across Slab

Factored Moment: $M_u := \max\left(U_1 \cdot M_{max}, S_{d.f.12} \cdot M_{max}\right) = 0.89 \cdot kip \cdot ft$

Tension Reinforcement Area: $A_s := A_b \div sp \cdot b_w = 0.60 \cdot in^2$

Find the equivalent Rectangular Compression Block: $a_c := \dfrac{A_s \cdot f_y}{\left(0.85 \cdot f'_c \cdot b_w\right)} = 0.78 \cdot in$

Allowable Moment: $M_{all} := \phi_m \cdot A_s \cdot f_y \cdot \left(d_{rebar} - \dfrac{a_c}{2}\right) = 17 \cdot ft \cdot kip$

Find the Demand/Capacity Ratio: $\boxed{DC_{mratio} := \dfrac{M_u}{M_{all}} = 0.05}$

check := | "Flexural Capacity > Demand, Moment Design Okay" if $M_u < M_{all}$
"Flexural Capacity < Demand, Redesign" if $M_u > M_{all}$

check = "Flexural Capacity > Demand, Moment Design Okay"

Check Shearing of Dowels Between Wall and Slab:

Compression Face Shear Strength:
(ACI 318-11, §11.4)

Note: Where shear reinforcement perpendicular to axis of member is used. Additional design will be conducted with future submittal.

$$V_C := \frac{A_s \cdot f_y \cdot d_{rebar}}{sp} = 20.06 \cdot kip$$

Design Shear Capacity: $\phi V_n := \phi_v \cdot V_C = 15.05 \cdot kip$

Demand Capacity Ratio: $DCR_V := \frac{V_u}{\phi V_n} = 0.04$ $\boxed{< 1.0 \text{ therefore okay}}$

Shrinkage and Temperature Reinforcement (S&T): ACI 318-11, §7.12 & USACE §2-8

Maximum Spacing:
(ACI 318-11, §7.12.2.2)
$s := \min\left[18in, 5 \cdot 0.5\left(t_{slab}\right), 12in\right] = 12.00 \cdot in$

Minimum Area of Steel Ratio:
(ACI §7.12.2.1)
$\rho_{ACI} := 0.0018$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0040$$

Minimum Area of Steel Ratio: $\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0040$

Minimum Steel Area:
(USACE §2-8)
$A_{s.min} := \rho_{max} \cdot b_w \cdot t_{slab} = 0.58 \cdot in^2$

Bar Size and Spacing of Reinforcement: $Barsize = 7$ $sp = 12.00 \cdot in$

Area of Reinforcement: $A_s := A_b \cdot \frac{12in}{s} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min}}{2 \cdot A_s} = 0.48}$$ $\boxed{< 1.0 \text{ therefore okay}}$

Spillway Wall (Stilling Basin) - Upper Black Creek Reservoir

Codes References

1) American Concrete Institute (ACI). Building Code Requirements for Structural Concrete (ACI 318-14), (2014).
2) ACI (American Concrete Institute), 2006. Code Requirements for Environmental Engineering Concrete Structures (ACI 350-06).
3) American Society of Civil Engineers (ASCE). Minimum Design Loads for Buildings and Other Structures (ASCE 7-10), (2013).
4) U.S. Army Corps of Engineers (USACE). Engineering and Design: Retaining and Flood Walls, (1989). EM 1110-2-2502.
5) U.S. Army Corps of Engineers (USACE). 2003 "Strength Design for Reinforced-Concrete Hydraulic Structures." EM 1110-2-2104-Appendix E: Table E-1.
6) Agusti, G. C. and Sitar. 2013. UCB GT 13-02 "Seismic Earth Pressures on Retaining Structures with Cohesive Backfills." UCB GT 13-02.

Structural Design

The following design calculates the applied loads and design strength for the Upper Black Creek Reservoir spillway wall. The wall is designed as a cantilevered retaining wall with soil and seismic loads. The strength/capacity checks are performed using the one foot strip method and code requirements in ACI 318-14, ACI 350-06 and USACE EM 1110-2-2104.

Design Loads (Forces)

The applied loads and load cases for the wall are calculated using guidelines from the documents referenced above. USACE and ACI strength/service load combinations are applied and the resulting combinations are shown below.

Wall Reinforcement Summary

Vertical Reinforcement

Tension Face = #7 at 12" O.C.

Compression Face = #7 at 12" O.C.

Horizontal Reinforcement

Channel = #5 at 12" O.C.

Embankment = #5 at 12" O.C.

Foundation Dowels

Full height bars, mechanical splices or slab tension splices will be allowed. Bars must be developed at wall/foundation joint.

Counterforts

None

Footing Reinforcement Summary

Perpendicular to Wall

Tension Face = #9 at 12" O.C.

Compression Face = #9 at 12" O.C.

Parallel to Wall

Top = #7 at 12" O.C.

Bottom = #7 at 12" O.C.

Wall Design Properties:

Top of Wall Elevation:	$El_{top} := 8737ft$
Bottom of Wall Elevation:	$El_{bot} := 8724ft$
Wall Height:	$H_{wall} := El_{top} - El_{bot} = 13.00\,ft$
Bottom Wall Thickness:	$t_{wall.B} := 1ft + 6in$
Top Wall Thickness:	$t_{wall.T} := 1ft + 0in$
Footing width:	$B := 8ft$
Toe projection:	$Toe := 3.5ft$
Heel projection:	$Heel := B - Toe - t_{wall.B} = 3.00\,ft$
Footing thickness:	$t_{base} := 2.0ft$
Slab thickness:	$t_{slab} := 2.0ft$
Design Width:	$b_w := 12in$
Unit weight of concrete:	$\gamma_{conc} := 150 \cdot pcf$
Distance between Control Joints:	$L_{CJ} := 25ft$
Specified Compressive Strength of Concrete:	$f'_c := 4500psi$
Minimum Tensile Strength of Concrete:	$f_t := 0.1 \cdot f'_c = 450\,psi$ *[ACI R22.2.2.2]*

Water Surface Elevations and Properties:

Elevation of normal water surface: (Assumed at base of structure)	$El_{nws} := El_{bot} - t_{base} + 0ft = 8722.00\,ft$
Water Unit Weight:	$\gamma_w := 62.4pcf$

Fill Elevations and Properties:

Reference:

Elevation of fill material beside channel: $El_{fill} := 8732ft$

Fill Properties (Soil type: SM)

Angle of internal friction: $\phi_f := 30 \cdot deg$

Unit weight: $\gamma_f := 130pcf$

Angle of inclined backfill: $\alpha_f := 0deg$

At-rest earth pressure coefficient: $k_0 := 1 - \sin(\phi_f) = 0.50$

Active earth pressure coefficient: $k_a := \tan(45deg - 0.5 \cdot \phi_f)^2 = 0.33$

Passive earth pressure coefficient: $k_p := \tan(45deg + 0.5 \cdot \phi_f)^2 = 3.00$

Friction factor for mass concrete on sound rock: $\delta_1 := 0.7$ *[NAVFAC p7.2-63]*

Seismic Properties:

10,000yr Horizontal Seismic Coefficient: $k_h := 0.4972$ [USGS Unified Hazard Tool]

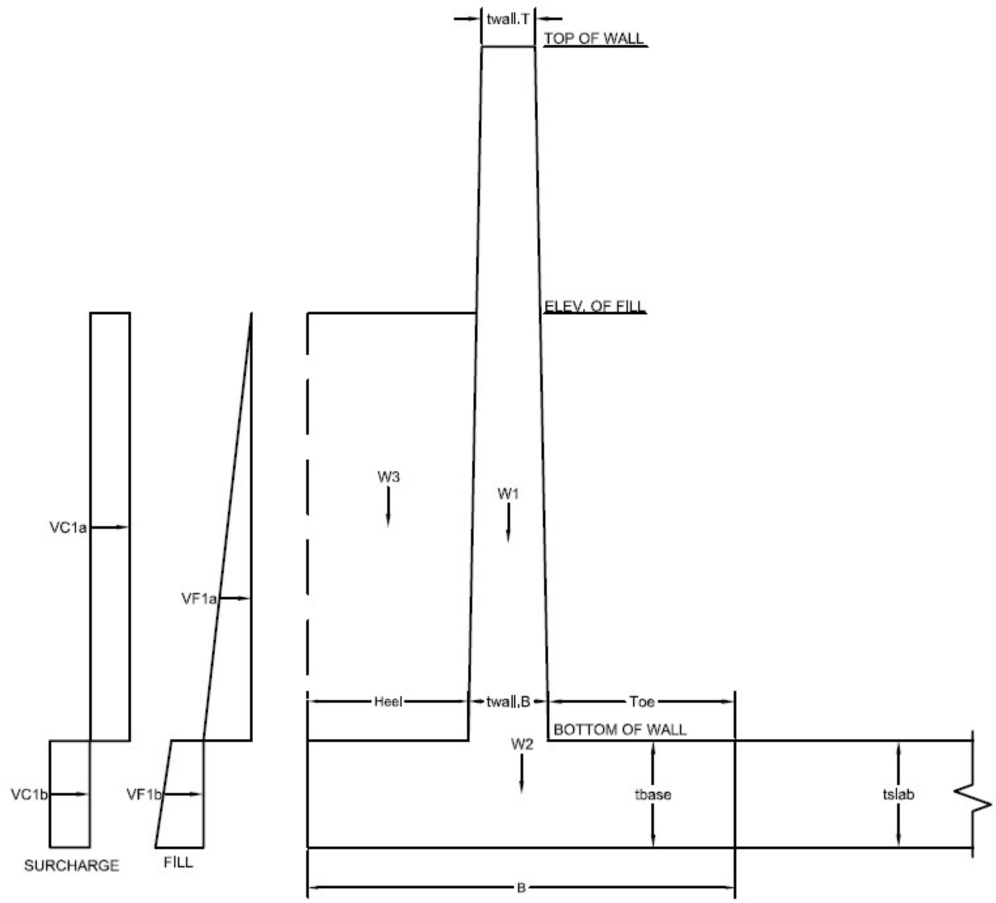
Earth Pressure Coefficient: $K_{ae} := 0.42 \cdot k_h = 0.21$ *[Augusti: UCB GT 13-02: Eq. 4.3]*

Other Properties:

Construction Surcharge: $q_c := 200psf$

1.) LOAD CASE 1 (Usual Condition) - Spillway Wall Loading:

Note: The retaining wall stem is analyzed as a cantilever beam with fixity provided at the footing. Beam is subject to lateral forces due to at-rest earth forces due to fill and construction live loads. Groundwater table is assumed below the wall section.



1.) LOAD CASE 1 (Usual Condition) - Spillway Wall Loading:

LOAD CASE 1-Applied Loads: Static

Active Earth Pressure

Base Shear:
$$V_{F1a} := 0.5 \cdot k_a \gamma_f \cdot (El_{fill} - El_{bot})^2 b_w = 1.39 \cdot kip$$

Footing Shear:
$$V_{F1b} := \left[k_a \gamma_f \cdot (El_{fill} - El_{bot}) \cdot t_{base} + 0.5 \cdot k_a \cdot \gamma_f \cdot t_{base}{}^2\right] \cdot b_w = 0.78 \cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_{F.1}(y) := \begin{cases} \left[0.5 \cdot k_a \cdot \gamma_f \cdot (El_{fill} - El_{bot} - y)^2 \cdot b_w\right] & \text{if } \; y < (El_{fill} - El_{bot}) \\ 0 & \text{otherwise} \end{cases}$$

$V_{F.1}(El_{nws} - El_{bot}) = 2.17 \cdot kip$ $\qquad V_{F.1}(0ft) = 1.39 \cdot kip$

Base Moment:
$$M_{F.1.b} := V_{F1a} \cdot \left[0.33(El_{fill} - El_{nws}) + (El_{nws} - El_{bot})\right] = 1.80\,ft \cdot kip$$

Moment as at a Distance 'y' From Base of Wall:

$$M_{F.1}(y) := 0.5 \cdot k_a \cdot \gamma_f \cdot (El_{fill} - El_{bot} - y)^2 \cdot b_w 0.33(El_{fill} - El_{bot} - y)$$

$M_{F.1}(El_{nws} - El_{bot}) = 7.15 \cdot kip \cdot ft$ $\qquad M_{F.1}(0ft) = 3.7 \cdot kip \cdot ft$

Passive Earth Pressure

Rock to Interior:
$$V_{F4} := 0.5 \cdot k_p \cdot \gamma_f \cdot (t_{base} - t_{slab})^2 \cdot 1ft = 0.00 \cdot kip$$
$$V_{F5} := k_p \cdot \gamma_{conc} \cdot t_{slab} \cdot (t_{base} - t_{slab}) \cdot 1ft = 0.00 \cdot kip$$
$$F_{F.T2} := V_{F4} + V_{F5} = 0.00 \cdot kip$$

Vertical Forces

Weight of stem: $W_1 := \gamma_{conc} \cdot (El_{top} - El_{bot}) \cdot 0.5(t_{wall.B} + t_{wall.T}) \cdot 1ft = 2.44 \cdot kip$

Weight of footing: $W_2 := \gamma_{conc} \cdot B \cdot t_{base} \cdot 1ft = 2.40 \cdot kip$

Weight of fill over heel: $W_3 := \gamma_f \cdot \left[Heel + 0.5 \cdot (t_{wall.B} - t_{wall.T})\right] \cdot (El_{fill} - El_{bot}) \cdot 1ft = 3.38 \cdot kip$

1.) LOAD CASE 1 (Usual Condition) Cont'd

LOAD CASE 1-Applied Loads: Static

Construction Surcharge

Base Shear: $V_{C1a} := k_a \cdot q_c \cdot (El_{fill} - El_{bot}) \cdot b_w = 0.53 \cdot kip$

Shear as at a Distance 'y' From Base of Wall:

$$V_C(y) := \begin{cases} V_{C1a} \cdot \dfrac{(El_{fill} - El_{bot} - y)}{(El_{fill} - El_{bot})} & \text{if} \quad y < (El_{fill} - El_{bot}) \\ 0 & \text{otherwise} \end{cases}$$

$V_C(El_{nws} - El_{bot}) = 0.67 \cdot kip$ $\qquad$ $V_C(0ft) = 0.53 \cdot kip$

Footing Shear: $V_{C1b} := k_a \cdot q_c \cdot t_{base} \cdot b_w = 0.13 \cdot kip$

Base Moment: $M_C := 0.5 \cdot V_{C1a} \cdot (El_{fill} - El_{bot}) = 2.13 \, ft \cdot kip$

Moment as at a Distance 'y' From Base of Wall: $M_C(y) := 0.5 \cdot V_C(y) \cdot \dfrac{(El_{fill} - El_{bot} - y)^2}{(El_{fill} - El_{bot})}$

1.) LOAD CASE 1: (Usual Condition) Cont'd - Global Stability Checks

CHECK SLIDING STABILITY

Assume structure is constrained from transverse sliding. Due to connection with the slab the spillway wall is not allowed to move freely and is assumed to not slide.

CHECK OVERTURNING STABILITY

Moment Calculations

Horizontal force moment arms:

$$y_{1a} := \left(El_{fill} - El_{bot}\right) \div 3 + t_{base} = 4.67\,ft$$

$$y_{1b} := 0.5 \cdot t_{base} = 1.00\,ft$$

$$y_{c1a} := \left(El_{fill} - El_{bot}\right) \div 2 + t_{base} = 6.00\,ft$$

$$y_{c1b} := 0.5 \cdot t_{base} = 1.00\,ft$$

$$y_{FFT2} := 0.33\left(t_{base} - t_{slab}\right) = 0.00\,ft$$

Vertical force moment arms:

$$x_1 := Toe + t_{wall.T} + 0.33 \cdot \left(t_{wall.B} - t_{wall.T}\right) = 4.66\,ft$$

$$x_2 := B \div 2 = 4.00\,ft$$

$$x_3 := B - Heel \div 2 = 6.50\,ft$$

Summation of vertical forces:

$$V_{sum} := W_1 + W_2 + W_3 = 8.22 \cdot kip$$

Overturning Moment:

$$M_{o.1} := V_{F1a} \cdot y_{1a} + V_{F1b} \cdot y_{1b} + V_{C1a} \cdot y_{c1a} \; ... \; + V_{C1b} \cdot y_{c1b} = 10.58 \cdot kip \cdot ft$$

Resisting Moment:

$$M_r := W_1 \cdot x_1 + W_2 \cdot x_2 + W_3 \cdot x_3 + F_{F.T2} \cdot y_{FFT2} = 42.94 \cdot kip \cdot ft$$

Location of resultant:

$$x_{bar.1} := \frac{M_r - M_{o.1}}{V_{sum}} = 3.94\,ft$$

Eccentricity of the resultant:

$$ecc_1 := \frac{B}{2} - x_{bar.1} = 0.06\,ft$$

Overturning check:

$$Check_{ot.1} := \begin{cases} \text{"LC1 Overturning Stability OK"} & \text{if } \frac{1}{3} \cdot B \le x_{bar.1} \le \frac{2}{3}B \\ \text{"STOP - LC1 Overturning Stability Unacceptable"} & \text{otherwise} \end{cases}$$

$$Check_{ot.1} = \text{"LC1 Overturning Stability OK"} \quad \frac{1}{3} \cdot B = 2.67\,ft \quad \frac{2}{3} \cdot B = 5.33\,ft$$

1.) LOAD CASE 1 (Usual Condition) Cont'd

LOAD CASE 1-Loading Conditions & Combinations:

Unfactored Shear At Base of Wall: $V_{LC1}(y) := V_{F.1}(y) + V_C(y)$

Ultimate Shear At Base of Wall:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L) *[Table E-9, Load Case 1A]*

$$V_{uLC1.1}(y) := 2.2 \cdot \left(V_{F.1}(y) + V_C(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$V_{uLC1.2}(y) := 1.6 \cdot V_{F.1}(y)$$

ASCE Load Combination 2: 1.2 (D + F) + 1.6 (L) + 1.6 (H)

$$V_{uLC1.3}(y) := 1.6 \cdot V_C(y) + 1.6 \cdot V_{F.1}(y)$$

Controlling Case:

$$V_{uLC1}(y) := \max\left(V_{uLC1.1}(y), V_{uLC1.2}(y), V_{uLC1.3}(y)\right)$$

$$V_{uLC1}(0ft) = 4.22 \cdot kip \qquad \boxed{V_{uLC1.1}(0ft) = 4.22 \cdot kip}$$

Unfactored Moment At Base of Wall: $M_{LC1}(y) := M_{F.1}(y) + M_C(y)$

Ultimate Moment At Base of Wall:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L)

$$M_{uLC1.1}(y) := 2.2 \cdot \left(M_{F.1}(y) + M_C(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$M_{uLC1.2}(y) := 1.6 \cdot M_{F.1}(y)$$

ASCE Load Combination 2: 1.2 (D + F) + 1.6 (L) + 1.6 (H)

$$M_{uLC1.3}(y) := 1.6 \cdot M_C(y) + 1.6 \cdot M_{F.1}(y)$$

Controlling Case:

$$M_{uLC1}(y) := \max\left(M_{uLC1.1}(y), M_{uLC1.2}(y), M_{uLC1.3}(y)\right)$$

$$M_{uLC1}(0ft) = 12.75 \cdot kip \cdot ft \qquad \boxed{M_{uLC1.1}(0ft) = 12.75\, ft \cdot kip}$$

1.) LOAD CASE 1 (Usual Condition) Cont'd

Environmental Durability Factor (ACI 350-06)

Note: Required for Tension Controlled structure to encourage durability and liquid-tightness.
Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads. The durability factor is applied to service loads only.

Specified Yield Strength of Reinforcement: $f_y := 60\text{ksi}$

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4 $f_{s.ss} := 24000\text{psi}$

$$S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$$

Flexure in normal conditions: R10.6.4
- One way element, Bar at 6" spacing

$f_{s.f} := 34000\text{psi}$

$$S_{d.f} := \max\left(\frac{f_y}{f_{s.f}}, 1.0\right) = 1.76$$

Shear in Wall:

$\boxed{V_{LC1.ED}(y) := S_{d.ss} \cdot \left(V_{F.1}(y) + V_C(y)\right)}$ $\boxed{V_{LC1.ED}(0ft) = 5 \cdot kip}$ $>$ $\boxed{V_{uLC1.1}(0ft) = 4 \cdot kip}$

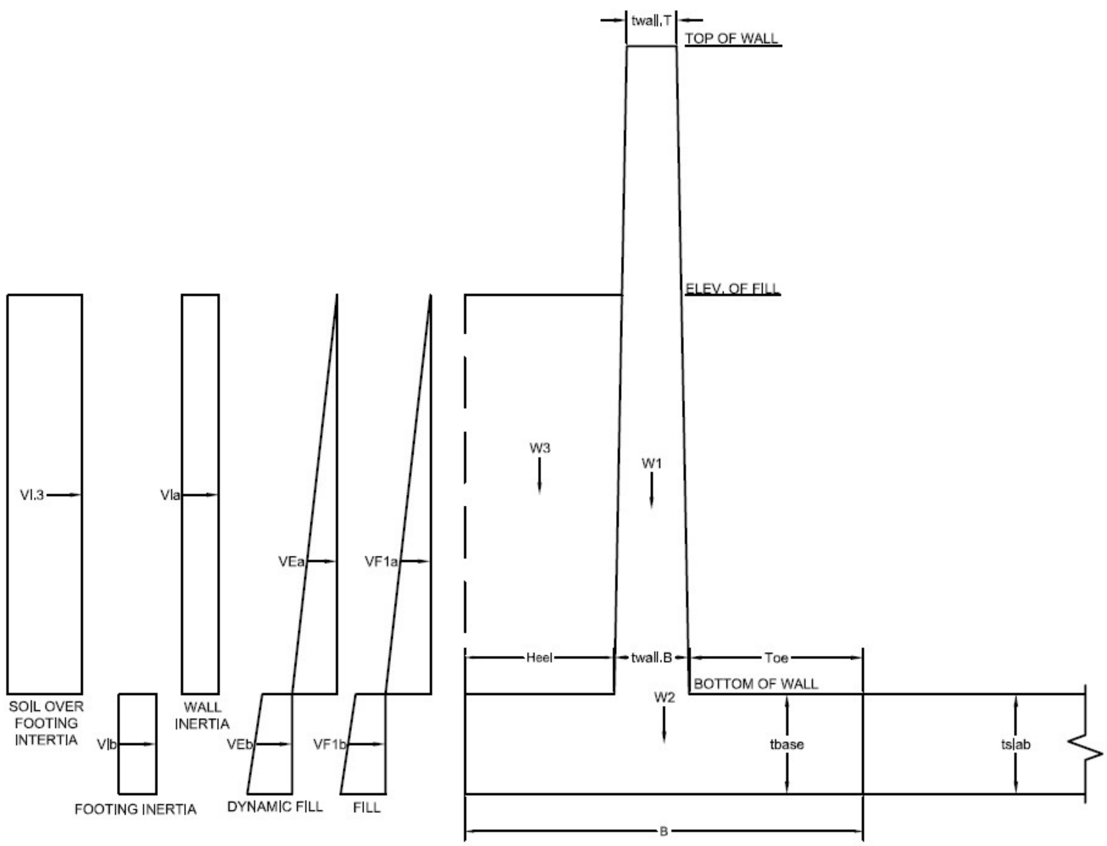
Moment in Wall:

$\boxed{M_{LC1.ED}(y) := S_{d.f} \cdot \left(M_{F.1}(y) + M_C(y)\right)}$ $\boxed{M_{LC1.ED}(0ft) = 10 \cdot kip \cdot ft}$ $<$ $\boxed{M_{uLC1.1}(0ft) = 13 \cdot kip \cdot ft}$

	Client	Blue Lake Reservoir Company			Page	95
GEI Consultants	Project	Upper Black Creek Reservoir			Pg. Rev.	
	By	C. Diebold	Chk.	M. Provencher	App.	C. Masching
	Date	11/27/2019	Date	11/27/2019	Date	11/27/2019
Project No.	1801834	Document No.	N/A			
Subject	Design of Spillway - Stilling Basin					

2.) LOAD CASE 2 (Extreme Condition) - Spillway Wall Loading:

Note: The retaining wall stem is analyzed as a cantilever beam with fixity provided at the footing. Beam is subject to lateral forces due to seismic inertial forces due to self-weight and dynamic fill loads. Groundwater table is assumed below the wall section.



2.) LOAD CASE 2 (Extreme Condition) - Spillway Wall Loading:

LOAD CASE 2-Applied Loads: Seismic

Seismic Incremental Force of Earth Pressure

Base Shear:

$$V_{Ea} := 0.5K_{ae} \cdot b_w \cdot \left[\gamma_f \cdot \left(El_{fill} - El_{bot}\right)^2\right] = 0.87 \cdot kip$$

$$V_{Eb} := \left[K_{ae}\gamma_f \cdot \left(El_{fill} - El_{bot}\right) \cdot t_{base} + 0.5 \cdot K_{ae} \cdot \gamma_f \cdot t_{base}{}^2\right] \cdot b_w = 0.49 \cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_E(y) := 0.5 \cdot K_{ae} \cdot \gamma_f \cdot \left(El_{fill} - El_{bot} - y\right)^2 \cdot b_w \qquad V_E(0ft) = 0.9 \cdot kip$$

Base Moment:

$$M_E := V_{Ea} \cdot \left[0.33\left(El_{fill} - El_{bot}\right)\right] = 2.29\,ft \cdot kip$$

Moment at a Distance 'y' From Base of Wall:

$$M_E(y) := 0.5 \cdot K_{ae} \cdot \gamma_f \cdot \left(El_{fill} - El_{bot} - y\right)^2 \cdot b_w 0.33\left(El_{fill} - El_{bot} - y\right)$$

$$M_E(0ft) = 2.3 \cdot kip \cdot ft$$

Inertial Forces

Wall Base Shear:

$$V_{Ia} := k_h \cdot W_1 = 1.21 \cdot kip$$

Shear as at a Distance 'y' From Base of Wall:

$$V_I(y) := k_h \cdot \gamma_{conc} \cdot \left[t_{wall.T} + \frac{0.5 \cdot \left(t_{wall.B} - t_{wall.T}\right) \cdot \left(H_{wall} - y\right)}{\left(H_{wall}\right)}\right] \cdot \left(H_{wall} - y\right) \cdot b_w$$

Base Moment:

$$M_I := V_{Ia} \cdot \left[\frac{H_{wall} \cdot \left(2t_{wall.T} + t_{wall.B}\right)}{3\left(t_{wall.B} + t_{wall.T}\right)}\right] = 7.35 \cdot kip \cdot ft$$

Moment at a Distance 'y' From Base of Wall:

$$M_I(y) := V_I(y) \cdot \frac{\left(H_{wall} - y\right) \cdot \left[2t_{wall.T} + \left[t_{wall.T} + \frac{\left(t_{wall.B} - t_{wall.T}\right) \cdot \left(H_{wall} - y\right)}{H_{wall}}\right]\right]}{3\left[t_{wall.T} + \frac{\left(t_{wall.B} - t_{wall.T}\right) \cdot \left(H_{wall} - y\right)}{H_{wall}} + t_{wall.T}\right]}$$

Footing Shear:

$$V_{Ib} := k_h \cdot W_2 = 1.19 \cdot kip$$

Soil over Footing Shear:

$$V_{I.3} := k_h \cdot W_3 = 1.68 \cdot kip$$

2.) LOAD CASE 2 (Extreme Condition) Cont'd -Global Stability Checks

CHECK SLIDING STABILITY

Assume structure is constrained from transverse sliding. Due to connection with the slab, the spillway wall is not allowed to move freely and is assumed to not slide.

CHECK OVERTURNING STABILITY

Moment Calculations

Horizontal force moment arms:

$$y_{Ia} := \left(El_{fill} - El_{bot}\right) \div 2 + t_{base} = 6.00\,ft$$

$$y_{Ib} := 0.5 \cdot t_{base} = 1.00\,ft$$

$$y_{I3} := y_{Ia} = 6.00\,ft$$

[Augusti: UCB GT 13-02]

$$y_{ea} := 0.4\left(El_{fill} - El_{bot}\right) + t_{base} = 5.20\,ft$$

$$y_{eb} := 0.4 \cdot t_{base} = 0.80\,ft$$

Summation of vertical forces:

$$V_{sum} := W_1 + W_2 + W_3 = 8.22 \cdot kip$$

Overturning Moment:

$$M_{o.2} := V_{F1a} \cdot y_{1a} + V_{F1b} \cdot y_{1b} \cdots + V_{Ea} \cdot y_{ea} + V_{Eb} \cdot y_{eb} \cdots + V_{Ia} \cdot y_{Ia} + V_{Ib} \cdot y_{Ib} + V_{I.3} \cdot y_{I3} = 30.71 \cdot kip \cdot ft$$

Resisting Moment:

$$M_r := W_1 \cdot x_1 + W_2 \cdot x_2 + W_3 \cdot x_3 + F_{F.T2} \cdot y_{FFT2} = 42.94 \cdot kip \cdot ft$$

Location of resultant:

$$x_{bar.2} := \frac{M_r - M_{o.2}}{V_{sum}} = 1.49\,ft$$

Eccentricity of the resultant:

$$ecc_2 := \frac{B}{2} - x_{bar.2} = 2.51\,ft$$

Overturning check:

$$Check_{ot.2} := \begin{cases} \text{"LC2 Overturning Stability OK"} & \text{if } 0 \le x_{bar.2} \le B \\ \text{"STOP - LC2 Overturning Stability Unacceptable"} & \text{otherwise} \end{cases}$$

$Check_{ot.2}$ = "LC2 Overturning Stability OK"

$B = 8.00\,ft$

2.) LOAD CASE 2 (Extreme Condition) Cont'd

LOAD CASE 2-Loading Conditions & Combinations:

Unfactored Shear At Base of Wall: $V_{LC2}(y) := V_{F.1}(y) + V_E(y) + V_I(y)$

Ultimate Shear At Base of Wall:

EM 1110-2-2104 Strength Load Combination: 1.0 EH + 1.0 Hs + 1.0 EQ *[Table E-9, Load Case 1A]*

$$V_{uLC2.1}(y) := V_{F.1}(y) + V_E(y) + V_I(y)$$

ASCE Load Combination 5: 1.2 (D + F) + 1.0 E + 1.6 H

$$V_{uLC2.2}(y) := \left(V_E(y) + V_I(y)\right) + 1.6 \cdot V_{F.1}(y)$$

Controlling Case: $V_{uLC2}(y) := \max\left(V_{uLC2.1}(y), V_{uLC2.2}(y)\right)$

$V_{uLC2}(0ft) = 4.30 \cdot kip$ $\boxed{V_{uLC2.2}(0ft) = 4.30 \cdot kip}$

Unfactored Moment At Base of Wall: $M_{LC3}(y) := M_{F.1}(y) + M_E(y) + M_I(y)$

Ultimate Moment At Base of Wall:

EM 1110-2-2014 Strength Load Combination: 1.0 EH + 1.0 Hs + 1.0 (EQ)

$$M_{uLC2.1}(y) := M_{F.1}(y) + M_E(y) + M_I(y)$$

ASCE Load Combination 5: 1.2 (D + F) + 1.0 E + 1.6 H

$$M_{uLC2.2}(y) := \left(M_E(y) + M_I(y)\right) + 1.6 \cdot M_{F.1}(y)$$

Controlling Case: $M_{uLC2}(y) := \max\left(M_{uLC2.1}(y), M_{uLC2.2}(y)\right)$

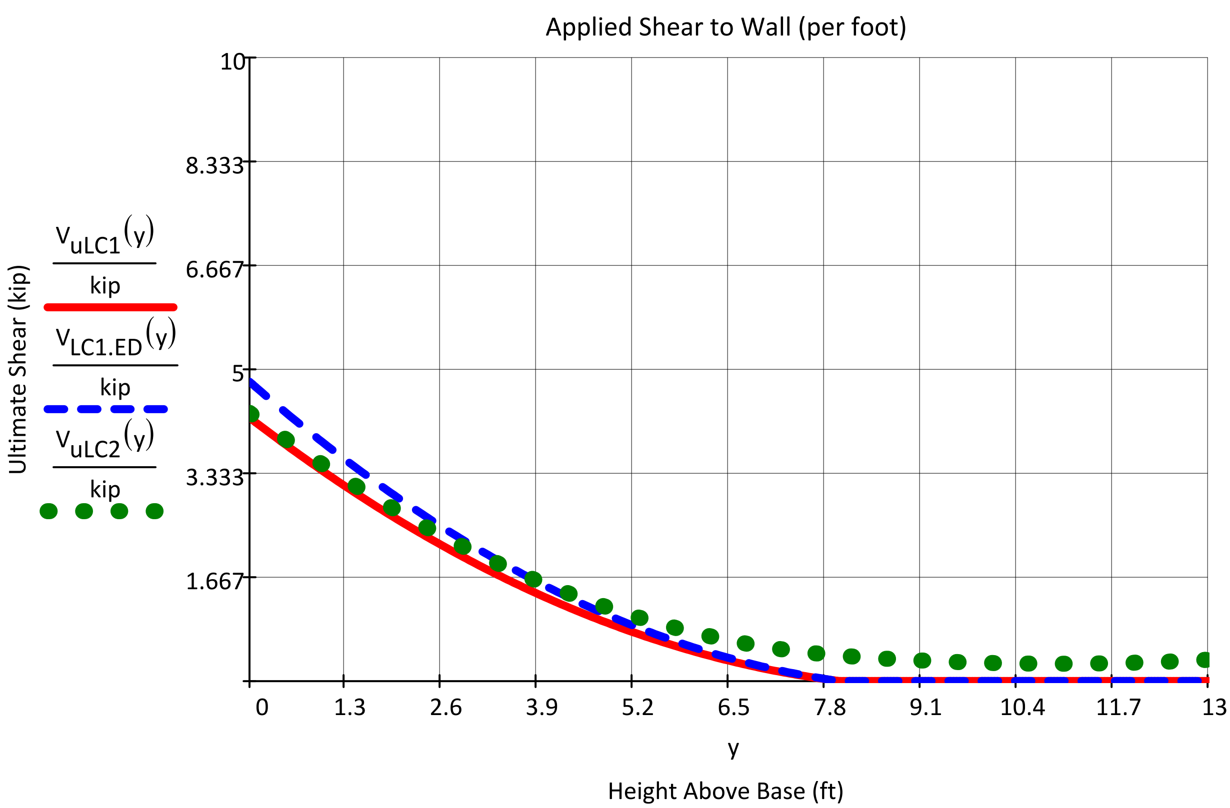
$M_{uLC2}(0ft) = 15.50 \cdot kip \cdot ft$ $\boxed{M_{uLC2.2}(0ft) = 15.50\, ft \cdot kip}$

3.) Load Case Summary Analysis

Controlling Applied Load:

Ultimate Shear at Base of Wall: $V_u(y) := \max\left(V_{uLC1}(y), V_{LC1.ED}(y), V_{uLC2}(y)\right)$

$V_u(0ft) = 4.80 \cdot kip$ $\qquad V_{LC1.ED}(0ft) = 4.80 \cdot kip$

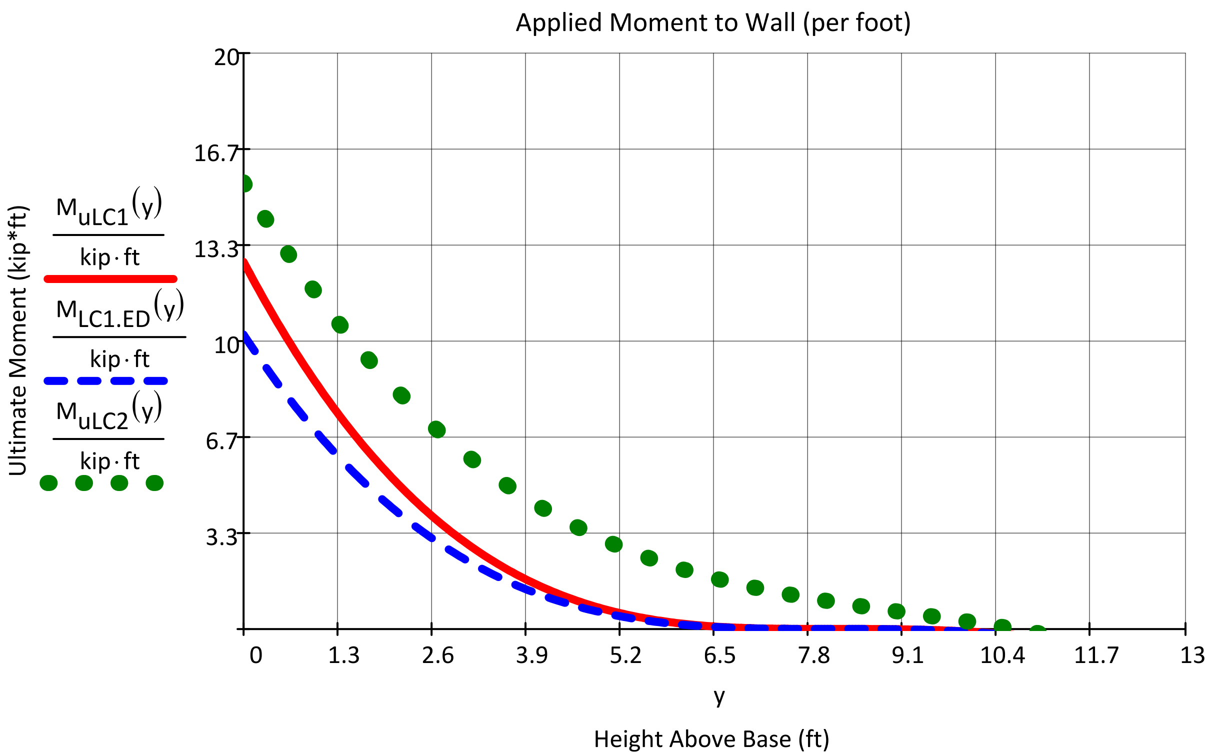


3.) Load Case Summary Analysis Cont'd

Controlling Applied Load:

Ultimate Moment at Base of Wall: $M_u(y) := \max\left(M_{uLC1}(y), M_{LC1.ED}(y), M_{uLC2}(y)\right)$

$M_u(0ft) = 15.50 \cdot kip \cdot ft$ $\qquad M_{LC1.ED}(0ft) = 10.22 \cdot kip \cdot ft$



4.) Wall Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1)*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05\cdot\dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Horizontal Reinforcement:

Bar Size and Spacing: $Size_{SH} := 5$ $sSH := 12in$
(Channel & Embankment)

Reinforcement Clear Cover: $clrS := 3in$

Diameter & Cross-Sectional Area of Bars: $dbSH := d_{b_{Size_{SH}}} = 0.63\cdot in$ $AbSH := A_{b_{Size_{SH}}} = 0.31\cdot in^2$

Area of Reinforcement: $A_{sSH} := AbSH\cdot\dfrac{12in}{sSH} = 0.31\cdot in^2$

$m := \left(t_{wall.T} - t_{wall.B}\right) \div H_{wall} = -0.04$

Section Depth at a Height 'y' from the base: $t_{wall}(y) := t_{wall.B} + m\cdot y$

Depth to Centroid of Embankment Reinf: $d_{SH}(y) := t_{wall}(y) - clrS - 0.5\cdot dbSH$

Reinforcement Ratio: $\rho_{SH}(y) := A_{sSH} \div \left(b_w\, d_{SH}(y)\right)$

4.) Wall Concrete & Rebar Strength Design Cont'd

Vertical Reinforcement:

Bar Size and Spacing: (Embankment) $Size_{SE} := 7$ $sSE := 12in$

Diameter & Cross-Sectional Area of Bars: $dbSE := d_{b_{Size_{SE}}} = 0.88 \cdot in$ $AbSE := A_{b_{Size_{SE}}} = 0.60 \cdot in^2$

Area of Tension Reinforcement: $A_{sSE} := AbSE \cdot \frac{12in}{sSE} = 0.60 \cdot in^2$

$m := (t_{wall.T} - t_{wall.B}) \div H_{wall} = -0.04$

Section Depth at a Height 'y' from the base: $t_{wall}(y) := t_{wall.B} + m \cdot y$

Depth to Centroid of Reinforcement: $d_{SE}(y) := t_{wall}(y) - clrS - dbSH - 0.5 \cdot dbSE$

Reinforcement Ratio: $\rho_{SE}(y) := A_{sSE} \div (b_w d_{SE}(y))$

Bar Size and Spacing: (Channel) $Size_{SC} := 7$ $sSC := 12in$

Diameter & Cross-Sectional Area of Bars: $dbSC := d_{b_{Size_{SC}}} = 0.88 \cdot in$ $AbSC := A_{b_{Size_{SC}}} = 0.60 \cdot in^2$

Area of Tension Reinforcement: $A_{sSC} := AbSC \cdot \frac{12in}{sSC} = 0.60 \cdot in^2$

Depth to Centroid of Reinforcement: $d_{SC}(y) := t_{wall}(y) - clrS - dbSH - 0.5 \cdot dbSC$

Reinforcement Ratio: $\rho_{SC}(y) := A_{sSC} \div (b_w d_{SC}(y))$

5.) Wall Strength Capacity

Wall Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
ACI 318-14, §22.5.5.1)

$V_c(y) := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot \min\left(d_{SE}(y), d_{SC}(y)\right)$

$V_c(0ft) = 22.44 \cdot kip$

Design Shear Capacity:

$\phi V_n(y) := \phi_V \cdot \left(V_c(y)\right)$

$\phi V_n(0ft) = 16.83 \cdot kip$

$V_u(0ft) = 4.80 \cdot kip$

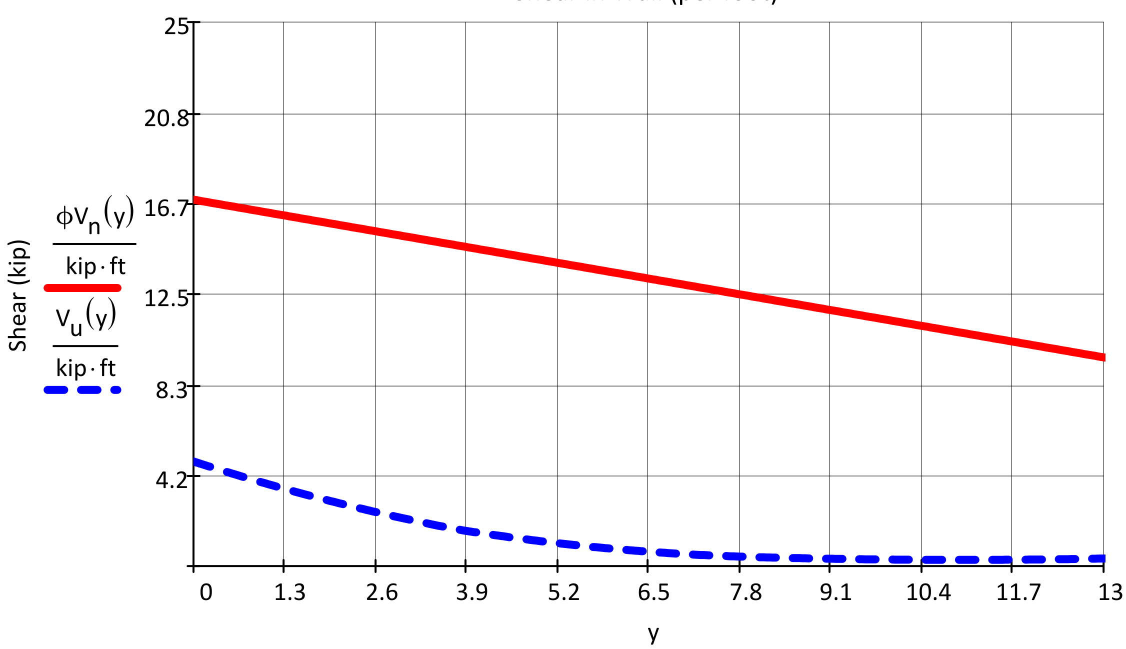
Demand Capacity Ratio:

$DCR_V(y) := \dfrac{V_u(y)}{\phi V_n(y)}$

$DCR_V(0ft) = 0.29$

< 1.0 therefore section is okay for shear strength

Shear in Wall (per foot)



Height Above Base (ft)

5.) Wall Strength Capacity Cont'd

Wall Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(ACI 318-14)

$$M_n(y) := \rho_{SE}(y)\cdot f_y\cdot b_w\cdot d_{SE}(y)^2\cdot\left(1-0.59\cdot\rho_{SE}(y)\cdot\frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n(y) := \phi_t\cdot M_n(y)$$

$$\phi M_n(0) = 36.57\,ft\cdot kip$$

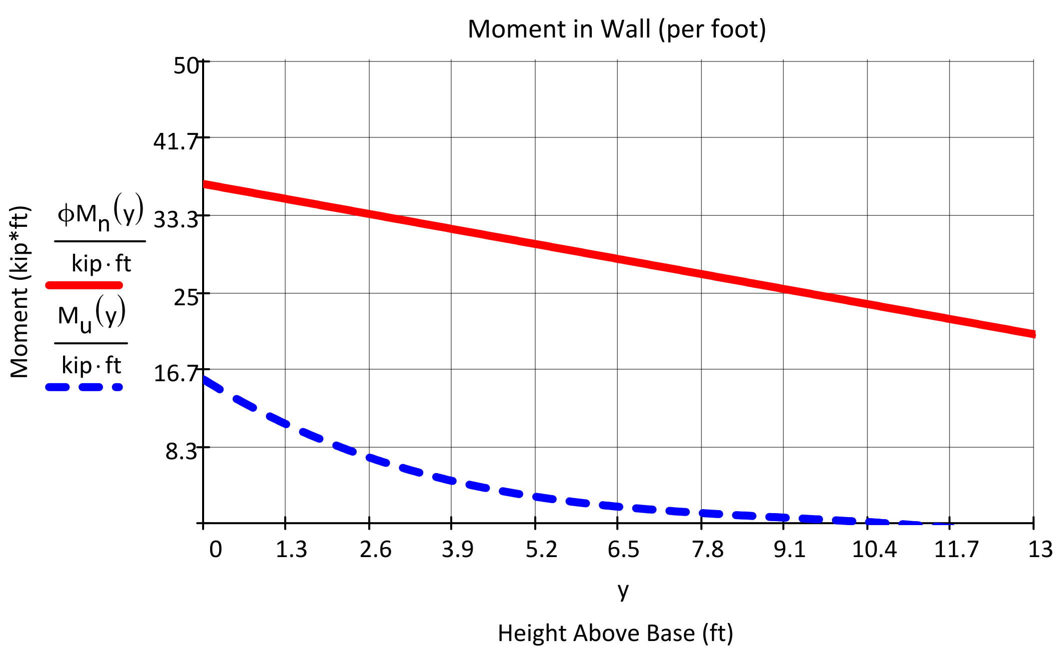
$$M_u(0) = 15.50\cdot kip\cdot ft$$

Demand Capacity Ratio:

$$DCR_M(y) := \frac{M_u(y)}{\phi M_n(y)}$$

$$DCR_M(0ft) = 0.42$$

< 1.0 therefore section is okay for flexural strength



6.) Wall Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in, 5\cdot 0.5\left(t_{wall.B} + t_{wall.T}\right), 12in\right] = 12.00\cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{cases} 0.0020 & \text{if } f_y < 60ksi \\ \max\left[\left(0.0018\cdot 60ksi \div f_y\right), 0.0014\right] & \text{otherwise} \end{cases} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0030$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0030$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min} := \rho_{max}\cdot b_w\cdot 0.5\cdot\left(t_{wall.B} + t_{wall.T}\right) = 0.54\cdot in^2$$

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Vertical Reinforcement:

Bar Size and Spacing of Reinforcement:
(Embankment)

$Size_{SE} = 7.00$ $\quad sSE = 12.00\cdot in$

Diameter & Cross-Sectional Area of Bars:

$dbSE = 0.88\cdot in$ $\quad AbSE = 0.60\cdot in^2$

Area of Reinforcement:

$A_{sSE} = 0.60\cdot in^2$

Bar Size and Spacing of Reinforcement:
(Channel)

$Size_{SC} = 7.00$ $\quad sSC = 12.00\cdot in$

Diameter & Cross-Sectional Area of Bars:

$dbSC = 0.88\cdot in$ $\quad AbSC = 0.60\cdot in^2$

Area of Reinforcement:

$A_{sSC} = 0.60\cdot in^2$

$$\boxed{\frac{A_{s.min}}{A_{sSC} + A_{sSE}} = 0.45}$$

< 1.0 therefore okay

6.) Wall Rebar Design Cont'd

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Horizontal Reinforcement:

Wall is reinforced with two sections. The top 20-feet from the top of the wall use reinforcing at half the spacing of that after 20-feet.

Bar Size and Spacing of Reinforcement: (Embankment & Channel) $Size_{SH} = 5.00$ $sSH = 12.00 \cdot in$

Diameter & Cross-Sectional Area of Bars: $dbSH = 0.63 \cdot in$ $AbSH = 0.31 \cdot in^2$

Area of Reinforcement: $A_{sSH} = 0.31 \cdot in^2$

$$\boxed{\frac{A_{s.min}}{2 \cdot A_{sSH}} = 0.87}$$ < 1.0 therefore okay

Minimum Flexural Steel: (ACI 318-14, §9.6.1.2)

Vertical Reinforcement:

$$A_{sV.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot \min\left(d_{SE}(0ft), d_{SC}(0ft)\right), \frac{200 \cdot psi \cdot b_w \cdot \min\left(d_{SE}(0ft), d_{SC}(0ft)\right)}{f_y}\right) = 0.56 \cdot in^2$$

$$\boxed{\frac{A_{sV.min}}{A_{sSE}} = 0.93}$$ < 1.0 therefore okay

6.) Wall Rebar Design Cont'd

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Reinforcement Ratio Provided: (Vertical)

Embankment: $\rho_{pVE} := \dfrac{A_{sSE}}{d_{SE}(0ft) \cdot b_w} = 0.0036$

Channel: $\rho_{pVC} := \dfrac{A_{sSE}}{d_{SE}(0ft) \cdot b_w} = 0.0036$

Reinforcement Ratio Provided: (Horizontal)

$\rho_{pH} := \dfrac{A_{sSH}}{d_{SH}(0ft) \cdot b_w} = 0.0018$

Depth to Neutral Axis: (Vertical)

$cVE := \dfrac{\rho_{pVE} \cdot f_y \cdot d_{SE}(0ft)}{0.85 \cdot \beta_1 \cdot f'_c} = 0.95 \cdot in$

$cVC := \dfrac{\rho_{pVC} \cdot f_y \cdot d_{SC}(0ft)}{0.85 \cdot \beta_1 \cdot f'_c} = 0.95 \cdot in$

Depth to Neutral Axis: (Horizontal)

$cH := \dfrac{\rho_{pH} \cdot f_y \cdot d_{SH}(0ft)}{0.85 \cdot \beta_1 \cdot f'_c} = 0.49 \cdot in$

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Calculated Strain in Tension Steel at Nominal Strength: (Vertical)

$\varepsilon_{tVE} := 0.003 \cdot \left(\dfrac{d_{SE}(0ft)}{cVE} - 1\right) = 0.04$ $\quad \varepsilon_{tVC} := 0.003 \cdot \left(\dfrac{d_{SC}(0ft)}{cVC} - 1\right) = 0.04$

(Horizontal) $\varepsilon_{tH} := 0.003 \cdot \left(\dfrac{d_{SH}(0ft)}{cH} - 1\right) = 0.09$

$\boxed{\dfrac{\varepsilon_{t.min}}{\varepsilon_{tVE}} = 0.10}$ $\boxed{\dfrac{\varepsilon_{t.min}}{\varepsilon_{tVC}} = 0.10}$ $\boxed{\dfrac{\varepsilon_{t.min}}{\varepsilon_{tH}} = 0.05}$ $\boxed{< 1.0 \text{ therefore okay}}$

7.) Footing Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1)*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05 \cdot \dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Reinforcement Parallel to Wall:

Heel Reinforcement:

Bar Size and Spacing (Top): $Size_{HTP} := 7$ $sHTP := 12in$

Diameter & Cross-Sectional Area of Bars:

$dbHTP := d_{b_{Size_{HTP}}} = 0.88 \cdot in$

$AbHTP := A_{b_{Size_{HTP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sHTP} := AbHTP \cdot \dfrac{12in}{sHTP} = 0.60 \cdot in^2$

Depth to Centroid of Tension Reinforcement: $d_{HTP} := t_{base} - 0.5 \cdot dbHTP = 1.96\,ft$

Reinforcement Ratio: $\rho_{HTP} := A_{sHTP} \div \left(b_w\, d_{HTP}\right) = 0.00212$

Bar Size and Spacing (Bottom): $Size_{HBP} := 7$ $sHBP := 12in$

Diameter & Cross-Sectional Area of Bars:

$dbHBP := d_{b_{Size_{HBP}}} = 0.88 \cdot in$

$AbHBP := A_{b_{Size_{HBP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sHBP} := AbHBP \cdot \dfrac{12in}{sHBP} = 0.60 \cdot in^2$

7.) Footing Concrete & Rebar Strength Design Cont'd

Reinforcement Parallel to Wall:

Toe Reinforcement:

Bar Size and Spacing (Top): $Size_{TTP} := 7$ $sTTP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbTTP := d_{b_{Size_{TTP}}} = 0.88 \cdot in$

$AbTTP := A_{b_{Size_{TTP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sTTP} := AbTTP \cdot \frac{12in}{sTTP} = 0.60 \cdot in^2$

Bar Size and Spacing (Bottom): $Size_{TBP} := 7$ $sTBP := 12in$

Diameter & Cross-Sectional Area of Bars: $dbTBP := d_{b_{Size_{TBP}}} = 0.88 \cdot in$

$AbTBP := A_{b_{Size_{TBP}}} = 0.60 \cdot in^2$

Area of Reinforcement: $A_{sTBP} := AbTBP \cdot \frac{12in}{sTBP}$

Reinforcement Perpendicular to Wall:

Heel Reinforcement:

Bar Size and Spacing (Top): $Size_{HT} := 9$ $sHT := 12in$

Reinforcement Clear Cover: $clrHT := 3in$

Diameter & Cross-Sectional Area of Bars: $dbHT := d_{b_{Size_{HT}}} = 1.13 \cdot in$ $AbHT := A_{b_{Size_{HT}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sHT} := AbHT \cdot \frac{12in}{sHT} = 1.00 \cdot in^2$

Section Depth: $t_{foot} := t_{base} = 2.00\,ft$

Depth to Centroid of Tension Reinforcement: $d_{HT} := t_{foot} - clrHT - (0.5 \cdot dbHT) = 1.70\,ft$

Reinforcement Ratio: $\rho_{HT} := A_{sHT} \div \left(b_w\, d_{HT}\right) = 0.00408$

7.) Footing Concrete & Rebar Strength Design Cont'd

Reinforcement Perpendicular to Wall:

Heel Reinforcement:

Bar Size and Spacing (Bottom): $Size_{HB} := 9$ $sHB := 12in$

Reinforcement Clear Cover: $clrHB := 4in$

Diameter & Cross-Sectional Area of Bars: $dbHB := d_{b_{Size_{HB}}} = 1.13 \cdot in$ $AbHB := A_{b_{Size_{HB}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sHB} := AbHB \cdot \frac{12in}{sHB} = 1.00 \cdot in^2$

Depth to Centroid of Compression Reinforcement: $d_{HB} := t_{foot} - clrHB - 0.5 \cdot dbHB = 1.62\,ft$

Toe Reinforcement:

Bar Size and Spacing (Top): $Size_{TT} := 9$ $sTT := 12in$

Reinforcement Clear Cover: $clrTT := 3in$

Diameter & Cross-Sectional Area of Bars: $dbTT := d_{b_{Size_{TT}}} = 1.13 \cdot in$ $AbTT := A_{b_{Size_{TT}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sTT} := AbTT \cdot \frac{12in}{sTT} = 1.00 \cdot in^2$

Depth to Centroid of Tension Reinforcement: $d_{TT} := t_{foot} - clrTT - (0.5 \cdot dbTT) = 1.70\,ft$

Reinforcement Ratio: $\rho_{TT} := A_{sTT} \div \left(b_w d_{TT}\right) = 0.00408$

Bar Size and Spacing (Bottom): $Size_{TB} := 9$ $sTB := 12in$

Reinforcement Clear Cover: $clrTB := 4in$

Diameter & Cross-Sectional Area of Bars: $dbTB := d_{b_{Size_{TB}}} = 1.13 \cdot in$ $AbTB := A_{b_{Size_{TB}}} = 1.00 \cdot in^2$

Area of Tension Reinforcement: $A_{sTB} := AbTB \cdot \frac{12in}{sTB} = 1.00 \cdot in^2$

Depth to Centroid of Compression Reinforcement: $d_{TB} := t_{foot} - clrTB - 0.5 \cdot dbTB = 1.62\,ft$

Reinforcement Ratio: $\rho_{TB} := A_{sTB} \div \left(b_w d_{TB}\right) = 0.00429$

8.) Footing Strength Capacity

Bearing Pressure:

Maximum Eccentricity: $ecc := \max(ecc_1, ecc_2) = 2.51\,ft$

Unfactored Shear: $V := (W_1 + W_2 + W_3) = 8.22 \cdot kip$

Factored Shear:
[ASCE 7-10, 2.3.2 Eqtn 2:
DL & pressure of bulk materials factor]

$V_u := 1.2 \cdot (W_1 + W_2) + 1.6 \cdot W_3 = 11.21 \cdot kip$

Unfactored bearing pressure at heel:

$$q_{heel.ED} := \frac{V}{B \cdot b_w} \cdot \left[1 - \frac{(6 \cdot ecc_1)}{B}\right] = 0.98 \cdot \frac{kip}{ft^2}$$

Unfactored bearing pressure at toe:

$$q_{toe.ED} := \frac{V}{B \cdot b_w} \cdot \left[1 + \frac{(6 \cdot ecc_1)}{B}\right] = 1.08 \cdot \frac{kip}{ft^2}$$

Factored bearing pressure at heel:

$$q_{u.heel} := \frac{V_u}{B \cdot b_w} \cdot \left[1 - \frac{(6 \cdot ecc)}{B}\right] = -1.24 \cdot \frac{kip}{ft^2}$$

Factored bearing pressure at toe:

$$q_{u.toe} := \frac{V_u}{B \cdot b_w} \cdot \left[1 + \frac{(6 \cdot ecc)}{B}\right] = 4.04 \cdot \frac{kip}{ft^2}$$

$$q_{u.max} := \max(q_{u.heel}, q_{u.toe}) = 4.04 \cdot ksf$$

8.) Footing Strength Capacity Cont'd

Note: The heel slab is designed as a cantilever beam with downward pressure from soil + slab and upward pressure from bearing. The toe slab is designed as a cantilever beam with downward pressure from slab and upward pressure from bearing.

Net Forces on Footings:

Downward weight of heel slab: (unfactored)

$$W_{heel} := \gamma_{conc} \cdot t_{base} \cdot 1ft = 0.30 \cdot \frac{kip}{ft}$$

Downward weight of toe slab: (unfactored)

$$W_{toe} := \gamma_{conc} \cdot t_{base} \cdot 1ft = 0.30 \cdot \frac{kip}{ft}$$

Downward weight of fill over heel slab: (unfactored)

$$W_{heelfill} := \gamma_f \cdot \left(El_{fill} - El_{bot}\right) \cdot 1ft = 1.04 \cdot \frac{kip}{ft}$$

Downward weight of heel slab:

$$W_{u.heel} := 1.2W_{heel} = 0.36 \cdot \frac{kip}{ft}$$

Downward weight of toe slab:

$$W_{u.toe} := 1.2W_{toe} = 0.36 \cdot \frac{kip}{ft}$$

Downward weight of fill over heel slab:

$$W_{u.heelfill} := 1.6W_{heelfill} = 1.66 \cdot \frac{kip}{ft}$$

Net downward force on heel (Unfactored-Load Case 1):

$$p_{h.ED1} := W_{heel} + W_{heelfill} - b_w \cdot \left[q_{heel.ED} + 0.5 \cdot \left(\frac{q_{toe.ED} - q_{heel.ED}}{B}\right) \cdot \left(Heel - \frac{d_{HT}}{2}\right)\right] = 0.35 \cdot \frac{kip}{ft}$$

Tension on Top

Net downward force on toe (Unfactored-Load Case 1):

$$p_{h.ED2} := W_{toe} - b_w \cdot \left[q_{toe.ED} + 0.5 \cdot \left(\frac{q_{heel.ED} - q_{toe.ED}}{B}\right) \cdot \left(Toe - \frac{d_{TT}}{2}\right)\right] = -0.76 \cdot \frac{kip}{ft}$$

Tension on Bottom

Net downward force on heel (Factored-Max):

$$p_h := W_{u.heel} + W_{u.heelfill} - b_w \cdot \left[q_{u.heel} + 0.5 \cdot \left(\frac{q_{u.toe} - q_{u.heel}}{B}\right) \cdot \left(Heel - \frac{d_{HT}}{2}\right)\right] = 2.55 \cdot \frac{kip}{ft}$$

Tension on Top

Net downward force on toe (Factored-Max):

$$p_{h2} := W_{u.toe} - b_w \cdot \left[q_{u.toe} + 0.5 \cdot \left(\frac{q_{u.heel} - q_{u.toe}}{B}\right) \cdot \left(Toe - \frac{d_{TT}}{2}\right)\right] = -2.81 \cdot \frac{kip}{ft}$$

Tension on Bottom

8.) Footing Strength Capacity Cont'd

Environmental Durability Factor (ACI 350-06)

Note: Required for Tension Controlled structure to encourage durability and liquid-tightness.
Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads. The durability factor is applied to service loads only.

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4 $f_{s.ss} := 24000psi$

$$S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$$

Flexure in normal conditions: R10.6.4
- One way element, bar ar 6" spacing

$f_{s.f} := 34000psi$

$$S_{d.f} := \max\left(\frac{f_y}{f_{s.f}}, 1.0\right) = 1.76$$

EDF Shear:

$$V_{heel.ED} := S_{d.ss}\left|p_{h.ED1}\right|\cdot Heel = 2.61\cdot kip$$
$$V_{toe.ED} := S_{d.ss}\left|p_{h.ED2}\right|\cdot Toe = 6.64\cdot kip$$

EDF Moment:

$$M_{heel.ED} := S_{d.f}\left|p_{h.ED1}\right|\cdot Heel^2 \div 2 = 2.76\cdot kip\cdot ft$$
$$M_{toe.ED} := S_{d.f}\left|p_{h.ED2}\right|\cdot Toe^2 \div 2 = 8.21\cdot kip\cdot ft$$

Ultimate Shear-Cantilevered Section:

$$V_{u.heel} := \left|p_h\right|\cdot Heel = 7.66\cdot kip$$
$$V_{u.toe} := \left|p_{h2}\right|\cdot Toe = 9.83\cdot kip$$

Ultimate Moment-Cantilevered Section:

$$M_{u.heel} := \left|p_h\right|\cdot Heel^2 \div 2 = 11.49\cdot kip\cdot ft$$
$$M_{u.toe} := \left|p_{h2}\right|\cdot Toe^2 \div 2 = 17.20\cdot kip\cdot ft$$

8.) Footing Strength Capacity Cont'd

Footing Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
(*ACI 318-14, §22.5.5.1)*

$$V_c := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{HT} = 32.90 \cdot kip$$

Design Shear Capacity:

$$\phi V_n := \phi_v \cdot (V_c) = 24.68 \cdot kip$$

$V_{u.heel} = 7.66 \cdot kip$ $\quad V_{heel.ED} = 2.61 \cdot kip$

Demand Capacity Ratio-Heel:

$$DCR_H := \frac{\max(V_{u.heel}, V_{heel.ED})}{\phi V_n} = 0.31$$

< 1.0 therefore section is okay for shear strength

$V_{u.toe} = 9.83 \cdot kip$ $\quad V_{toe.ED} = 6.64 \cdot kip$

Demand Capacity Ratio-Toe:

$$DCR_{F.T} := \frac{\max(V_{u.toe}, V_{toe.ED})}{\phi V_n} = 0.40$$

< 1.0 therefore section is okay for shear strength

Footing Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(*ACI 318-14)*

$$M_n := \rho_{HT} \cdot f_y \cdot b_w \cdot d_{HT}^2 \cdot \left(1 - 0.59 \cdot \rho_{HT} \cdot \frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n := \phi_t \cdot M_n = 89.01\, ft \cdot kip$$

$M_{u.heel} = 11.49\, ft \cdot kip$ $\quad M_{heel.ED} = 2.76\, ft \cdot kip$

Demand Capacity Ratio-Heel:

$$\boxed{DCR_H := \frac{\max(M_{u.heel}, M_{heel.ED})}{\phi M_n} = 0.13}$$

< 1.0 therefore section is okay for flexural strength

$M_{u.toe} = 17.20\, ft \cdot kip$ $\quad M_{toe.ED} = 8.21\, ft \cdot kip$

Demand Capacity Ratio-Toe:

$$\boxed{DCR_T := \frac{\max(M_{u.toe}, M_{toe.ED})}{\phi M_n} = 0.19}$$

< 1.0 therefore section is okay for flexural strength

9.) Footing Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in, 5 \cdot 0.5\left(t_{wall.B} + t_{wall.T}\right), 12in\right] = 12.00 \cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{vmatrix} 0.0020 \quad \text{if} \quad f_y < 60ksi \\ \max\left[\left(0.0018 \cdot 60ksi \div f_y\right), 0.0014\right] \quad \text{otherwise} \end{vmatrix} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{vmatrix} 0.003 \quad \text{if} \quad L_{CJ} < 30ft \\ 0.004 \quad \text{if} \quad 30ft \le L_{CJ} \le 40ft \\ 0.005 \quad \text{otherwise} \end{vmatrix} = 0.0030$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0030$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min.toe} := \rho_{max} \cdot b_w \cdot t_{base} = 0.86 \cdot in^2$$

$$A_{s.min.heel} := \rho_{max} \cdot b_w \cdot t_{base} = 0.86 \cdot in^2$$

Reinforcement perpendicular to Wall:

Bar Size and Spacing of Reinforcement:

$Size_{TT} = 9.00$ $\quad sTT = 1.00\,ft$

$Size_{TB} = 9.00$ $\quad sTB = 1.00\,ft$

Area of Reinforcement:

$A_{sTT} = 1.00 \cdot in^2$ $\quad A_{sTB} = 1.00 \cdot in^2$

$$\boxed{\frac{A_{s.min.toe}}{A_{sTT} + A_{sTB}} = 0.43}$$

< 1.0 therefore okay

Bar Size and Spacing of Reinforcement:

$Size_{HT} = 9.00$ $\quad sHT = 1.00\,ft$

$Size_{HB} = 9.00$ $\quad sHB = 1.00\,ft$

Area of Reinforcement:

$A_{sHT} = 1.00 \cdot in^2$ $\quad A_{sHB} = 1.00 \cdot in^2$

$$\boxed{\frac{A_{s.min.heel}}{A_{sHT} + A_{sHB}} = 0.43}$$

< 1.0 therefore okay

9.) Footing Rebar Design Cont'd

Reinforcement parallel to Wall:

Toe:

Bar Size and Spacing of Reinforcement: $Size_{TTP} = 7.00$ $sTTP = 1.00\,ft$

$Size_{TBP} = 7.00$ $sTBP = 1.00\,ft$

Area of Reinforcement: $A_{sTTP} = 0.60 \cdot in^2$ $A_{sTBP} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min.toe}}{A_{sTTP} + A_{sTBP}} = 0.72}$$ < 1.0 therefore okay

Heel:

Bar Size and Spacing of Reinforcement: $Size_{HTP} = 7.00$ $sHTP = 1.00\,ft$

$Size_{HBP} = 7.00$ $sHBP = 1.00\,ft$

Area of Reinforcement: $A_{sHTP} = 0.60 \cdot in^2$ $A_{sHBP} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min.heel}}{A_{sHTP} + A_{sHBP}} = 0.72}$$ < 1.0 therefore okay

Minimum Flexural Steel: (ACI 318-14, §9.6.1.2)

Perpendicular to Wall

Heel:

$$A_{s.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d_{HT}, \frac{200 \cdot psi \cdot b_w \cdot d_{HT}}{f_y}\right) = 0.82 \cdot in^2$$

Toe:

$$A_{s.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d_{TB}, \frac{200 \cdot psi \cdot b_w \cdot d_{TB}}{f_y}\right) = 0.78 \cdot in^2$$

$$\boxed{\frac{A_{s.min}}{A_{sHT}} = 0.78}$$ $$\boxed{\frac{A_{s.min}}{A_{sTB}} = 0.78}$$ < 1.0 therefore okay

9.) Footing Rebar Design Cont'd

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Perpendicular to Wall

Reinforcement Ratio Provided: $$\rho_p := \frac{AbHT}{d_{HT} \cdot b_w} = 0.0041$$

Depth to Neutral Axis: $$c := \frac{\rho_p \cdot f_y \cdot d_{HT}}{0.85 \cdot \beta_1 \cdot f'_c} = 1.58 \cdot in$$

Calculated Strain in Tension Steel at Nominal Strength: $$\varepsilon_t := 0.003 \cdot \left(\frac{d_{HT}}{c} - 1\right) = 0.04$$

$$\boxed{\frac{\varepsilon_{t.min}}{\varepsilon_t} = 0.11}$$

< 1.0 Therefore Okay

Spillway Slab (Stilling Basin) - Upper Black Creek Reservoir

Problem statement: Check shear and moment capacity across the slab. Assume the slab is pinned between two longitudinal joints spaced at 25 ft apart.

References

-ACI 318-14, Building Code Requirements for Structural Concrete
-EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures

Constants

Unit Weight of Water: $\gamma_w := 62.4\text{pcf}$ Unit Weight of Concrete: $\gamma_{conc} := 150\text{pcf}$

Specificied Compressive Strength of Concrete: $f'_c := 4500\text{psi}$

Specified Yield Strenght of Reinforcement: $f_y := 60\text{ksi}$

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *ACI 318-14, §21.2.1*

Strength Reduction Factor of Moment: $\phi_m := 0.9$ *ACI 318-14, §21.2.1*

Load Factor for Strength Design: $U_1 := 1.6$ *EM 1110-2-2104, Table E-9, Load Case 1B*

Dimensions & Reinforcement

Elevation of PWS during IDF: $EL_{PWS} := 8734\text{ft}$

Top of Slab Elevation: $TOS := 8724\text{ft}$

Thickness of Slab $t_{slab} := 24\text{in}$

Assume 1' wide strip of slab: $b_w := 12\text{in}$

Maximum distance between control joints: $L_{CJ} := 31\text{ft}$

Distance across slab: $Dist := 5\text{ft}$

Concrete Cover: $Cov := 4\text{in}$

Assume Rebar Spacing: $sp := 12\text{in}$

Assume a Bar Size: $Barsize := 7$ $Bar := d_{b_{Barsize}} = 0.88 \cdot \text{in}$ $A_b := A_{b_{Barsize}} = 0.60 \cdot \text{in}^2$

Calculations

Height of Water Column for uplift: $H_{max} := EL_{PWS} - TOS + t_{slab} = 12.00\,ft$

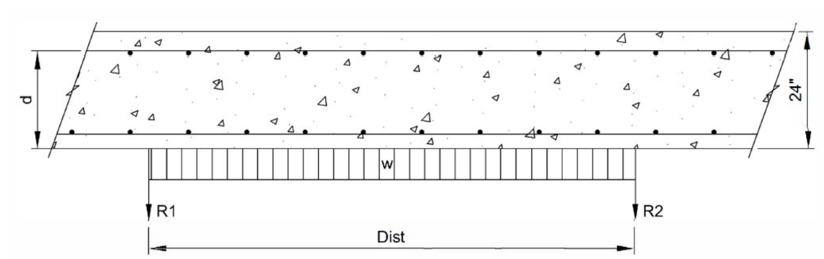
Max case = IDF conditions, vertical seismic does not control

Hydrostratic uplift per unit slab width: $p_{max} := H_{max} \cdot b_w \cdot \gamma_w = 0.75 \cdot klf$

Dead Load of slab per unit slab width: $w_d := t_{slab} \cdot b_w \cdot \gamma_{conc} = 0.30 \cdot klf$

Depth to Centroid of Reinforcement: $d_{rebar} := t_{slab} - Cov - Bar - \frac{Bar}{2} = 18.69 \cdot in$

Free Body Diagram of Slab Across Spillway



Calculate Reaction Anchor Reaction Forces R1 and R2

Sum Forces Vertically to find reaction anchor Loads R1 and R2 $\left(p_{max} - w_d\right) \cdot Dist - R_1 - R_2 = \blacksquare \cdot 0$

Reaction Force 1: $R_1 := \frac{\left[\left(p_{max} - w_d\right) \cdot Dist\right]}{2}$ $R_1 = 1.12 \cdot kip$

Reaction Force 1: $R_2 := R_1$ $R_2 = 1.12 \cdot kip$

Model Slab as a simple beam and define locations along x-axis for Shear & Moment Diagrams

Beginning of Beam: $x_{min} := 0ft$

End of Beam: $x_{max} := Dist = 5.00\,ft$

Location of Reaction Anchor 1: $x_a := x_{min} = 0.00 \cdot ft$

Location of Reaction Anchor 2: $x_b := x_{max} = 5.00\,ft$

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	Date	11/26/2019	Date	11/26/2019	Date	11/26/2019
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Define Range Variable "x" and "Step Size" of Range variable

Vector Step Size: $d_t := 0.01ft$

Define x: $x := x_{min}, (x_{min} + d_t) .. x_{max}$

Define Shear Loading for each Condition along Beam as a function of x

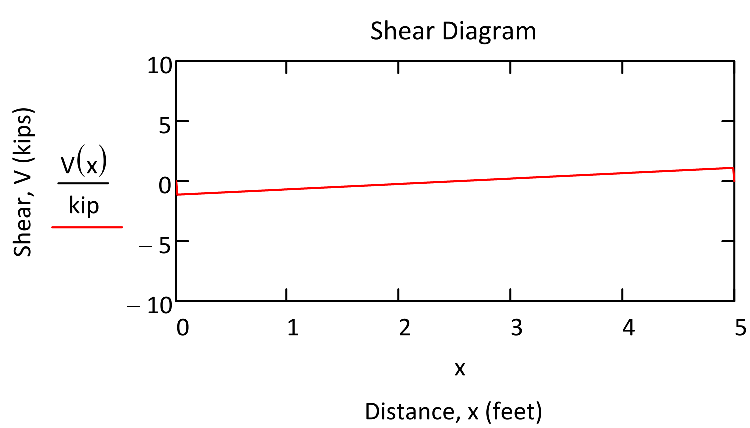
From x_{min} to x_a: $v_1(x) := (p_{max} - w_d) \cdot x$

From x_a to x_b: $v_2(x) := (p_{max} - w_d) \cdot x - R_1$

From x_b to x_{max}: $v_3(x) := (p_{max} - w_d) \cdot x - R_1 - R_2$

Conditional Shear Eqn:

$$V(x) := \begin{cases} v_1(x) & \text{if } x_{min} \le x \le x_a \\ v_2(x) & \text{if } x_a < x \le x_b \\ v_3(x) & \text{if } x_b < x \le x_{max} \\ 0 & \text{otherwise} \end{cases}$$



Check for Max Shear

$Check1 := V(x_a + d_t) = -1.12 \cdot kip$

$Check2 := V(x_a - d_t) = 0.00 \cdot kip$

$Check3 := V(x_a + Dist - d_t) = 1.12 \cdot kip$

$Check4 := V(x_a + Dist + d_t) = 0.00 \cdot kip$

$V_{max} := \max(Check1, Check2, Check3, Check4)$

$\boxed{V_{max} = 1.12 \cdot kip}$

Define Moment for each Condition along Beam as a function of x

From x_{min} to x_a: $$m_1(x) := (p_{max} - w_d)\cdot x \cdot \frac{1}{2} x$$

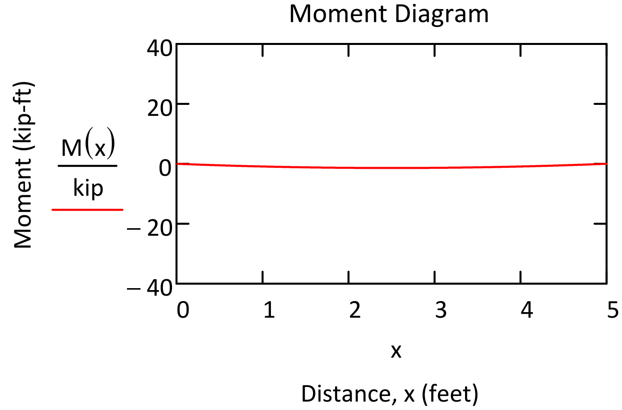
From x_a to x_b: $$m_2(x) := (p_{max} - w_d)\cdot x \cdot \frac{1}{2} x - R_1\cdot(x - x_a)$$

From x_b to x_{max}: $$m_3(x) := (p_{max} - w_d)\cdot x \cdot \frac{1}{2} x - R_1\cdot(x - x_a) - R_2\cdot(x - x_a - Dist)$$

Calculate Max Moment and Plot Moment Diagram

Conditional Moment Eqn: $$M(x) := \begin{cases} m_1(x) & \text{if } x_{min} \le x \le x_a \\ m_2(x) & \text{if } x_a < x \le x_b \\ m_3(x) & \text{if } x_b < x \le x_{max} \\ 0 & \text{otherwise} \end{cases}$$

Moment Diagram



$$Check1 := M\left(\frac{Dist}{2}\right) = -1.40\,ft\cdot kip$$

$$M_{max} := |Check1| = 1.40\,ft\cdot kip$$

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Controlling Applied Load:

Environmental Durability Factor (ACI 350-06):

Required for Tension Controlled structure to encourage durability and liquid-tightness.
Per section 21.2.1.8.a, the environmental durability factor need not be applied to load combinations that include earthquake loads.

Specified Yield Strength of Reinforcement: $f_y = 60 \cdot ksi$

Permissible Tensile Stress:

Shear stress in normal conditions: 9.2.6.4 $f_{s.ss} := 24000psi$

$$S_{d.ss} := \max\left(\frac{f_y}{f_{s.ss}}, 1.0\right) = 2.50$$

Flexure in normal conditions: R10.6.4
- One way element, bar ar 12" spacing

$f_{s.f.12} := 21000psi$

$$S_{d.f.12} := \max\left(\frac{f_y}{f_{s.f.12}}, 1.0\right) = 2.86$$

Check Shear Capacity Across Slab

Factored shear load: $V_u := \max\left(U_1 \cdot V_{max}, S_{d.ss} \cdot V_{max}\right) = 2.79 \cdot kip$

Allowable Shear Load: $V_{all} := \left(\phi_V \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{rebar}\right) = 22.6\,ft \cdot \frac{kip}{ft}$ *ACI 318-14, §22.5.5.1*

Find the Demand/Capacity Ratio:

$$\boxed{DC_{vratio} := \frac{V_u}{V_{all}} = 0.12}$$

check := | "Shear Capacity > Demand, Shear Design Okay" if $V_u < V_{all}$
| "Shear Capacity < Demand, Redesign" if $V_u > V_{all}$

check = "Shear Capacity > Demand, Shear Design Okay"

Check Moment Capacity Across Slab

Factored Moment: $M_u := \max\left(U_1 \cdot M_{max}, S_{d.f.12} \cdot M_{max}\right) = 4.01 \cdot kip \cdot ft$

Tension Reinforcement Area: $A_s := A_b \div sp \cdot b_w = 0.60 \cdot in^2$

Find the equivalent Rectangular Compression Block: $a_c := \dfrac{A_s \cdot f_y}{\left(0.85 \cdot f'_c \cdot b_w\right)} = 0.78 \cdot in$

Allowable Moment: $M_{all} := \phi_m \cdot A_s \cdot f_y \cdot \left(d_{rebar} - \dfrac{a_c}{2}\right) = 49.4 \cdot ft \cdot kip$

Find the Demand/Capacity Ratio: $\boxed{DC_{mratio} := \dfrac{M_u}{M_{all}} = 0.08}$

$$check := \begin{cases} \text{"Flexural Capacity > Demand, Moment Design Okay"} & \text{if } M_u < M_{all} \\ \text{"Flexural Capacity < Demand, Redesign"} & \text{if } M_u > M_{all} \end{cases}$$

check = "Flexural Capacity > Demand, Moment Design Okay"

Check Shearing of Dowels Between Wall and Slab:

Compression Face Shear Strength: *(ACI 318-11, §11.4)*

Note: Where shear reinforcement perpendicular to axis of member is used. Additional design will be conducted with future submittal.

$$V_C := \frac{A_s \cdot f_y \cdot d_{rebar}}{sp} = 56.06 \cdot kip$$

Design Shear Capacity: $\phi V_n := \phi_v \cdot V_C = 42.05 \cdot kip$

Demand Capacity Ratio: $DCR_V := \frac{V_u}{\phi V_n} = 0.07$ $\boxed{< 1.0 \text{ therefore okay}}$

Shrinkage and Temperature Reinforcement (S&T): ACI 318-11, §7.12 & USACE §2-8

Maximum Spacing: *(ACI 318-11, §7.12.2.2)* $s := \min\left[18in, 5 \cdot 0.5\left(t_{slab}\right), 12in\right] = 12.00 \cdot in$

Minimum Area of Steel Ratio: *(ACI §7.12.2.1)* $\rho_{ACI} := 0.0018$

Minimum Area of Steel Ratio: *(USACE §2-8)*

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0040$$

Minimum Area of Steel Ratio: $\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0040$

Minimum Steel Area: *(USACE §2-8)* $A_{s.min} := \rho_{max} \cdot b_w \cdot t_{slab} = 1.15 \cdot in^2$

Bar Size and Spacing of Reinforcement: $Barsize = 7$ $sp = 12.00 \cdot in$

Area of Reinforcement: $A_s := A_b \cdot \frac{12in}{s} = 0.60 \cdot in^2$

$$\boxed{\frac{A_{s.min}}{2 \cdot A_s} = 0.96}$$ $\boxed{< 1.0 \text{ therefore okay}}$

Spillway Stilling Basin Key - Upper Black Creek Reservoir

Codes References

1) American Concrete Institute (ACI). Building Code Requirements for Structural Concrete (ACI 318-14), (2014).
2) ACI (American Concrete Institute), 2006. Code Requirements for Environmental Engineering Concrete Structures (ACI 350-06).
3) U.S. Army Corps of Engineers (USACE). 2003 "Strength Design for Reinforced-Concrete Hydraulic Structures." EM 1110-2-2104-Appendix E: Table E-1.

Structural Design

The following design calculates the design strength for the Upper Black Creek Reservoir Spillway Stilling Basin Key necessary to develop the full passive resistance at the base of the slab. The key is designed as a cantilevered beam with soil loads. The strength/capacity checks are performed using the one foot strip method and code requirements in ACI 318-14, ACI 350-06 and USACE EM 1110-2-2104.

Design Loads (Forces)

The applied loads and load cases for the key are calculated using guidelines from the documents referenced above. USACE and ACI strength/service load combinations are applied and the resulting combinations are shown below.

Key Reinforcement Summary

Vertical Reinforcement Tension Face = #4 at 6" O.C.

Horizontal Reinforcement Channel = #4 at 6" O.C.

Key Design Properties:

Key Height: $H_{key} := 3ft$

Bottom Key Thickness: Minimum $t_{key.B} := 1ft + 0in$

Top Key Thickness: Minimum $t_{key.T} := 1ft + 0in$

Angle of Chute Inclination: $\theta := 26.57deg$

Slab thickness: $t_{slab} := 1.0ft \div \cos(\theta) = 1.12\,ft$

Key depth below slab: $d_{key} := 3.0ft$

Design Width: $b_w := 12in$

Unit weight of concrete: $\gamma_{conc} := 150 \cdot pcf$

Distance between Control Joints: $L_{CJ} := 30ft$

Specified Compressive Strength of Concrete: $f'_c := 4500psi$

Fill Properties:

Fill Properties

Angle of internal friction: $\phi_f := 30 \cdot deg$

Unit weight: $\gamma_f := 130pcf$

Active earth pressure coefficient: $k_a := \tan(45deg - 0.5 \cdot \phi_f)^2 = 0.33$

Passive earth pressure coefficient: $k_p := \tan(45deg + 0.5 \cdot \phi_f)^2 = 3.00$

Applied Loads: Static

Passive Earth Pressure

Active Pressure on the upstream side of key is conservatively ignored.

Shear at Top of Key: $F := k_p \cdot \left[\gamma_{conc} \cdot t_{slab} \cdot H_{key} + 0.5 \cdot \gamma_f \cdot (H_{key})^2\right] \cdot b_w = 3.26 \cdot kip$

Shear as at a Distance 'y' From Base of Key:

$$V_F(y) := \begin{cases} F - k_p \cdot \left[\gamma_{conc} \cdot t_{slab} \cdot (H_{key} - y) + 0.5 \cdot \gamma_f \cdot (H_{key} - y)^2\right] \cdot b_w & \text{if } y \le H_{key} \\ 0 & \text{otherwise} \end{cases}$$

$V_F(H_{key}) = 3.26 \cdot kip$ $\quad V_F(0ft) = 0.00 \cdot kip$

Moment at Top of Key:

$$M_F := (k_p - k_a) \cdot \left[\gamma_{conc} \cdot t_{slab} \cdot 0.5 (H_{key})^2 + 0.5 \cdot \gamma_f \cdot \frac{2}{3} (H_{key})^3\right] \cdot b_w = 5.13 \cdot kip \cdot ft$$

Moment as at a Distance 'y' From Base of Key:

$$M_F(y) := \begin{cases} (k_p - k_a) \cdot \left[\gamma_{conc} \cdot t_{slab} \cdot 0.5 (y)^2 + 0.5 \cdot \gamma_f \cdot \frac{2}{3} (y)^3\right] \cdot b_w & \text{if } y \le H_{key} \\ 0 & \text{otherwise} \end{cases}$$

$M_F(H_{key}) = 5.13 \cdot kip \cdot ft$ $\quad M_F(0ft) = 0.0 \cdot kip \cdot ft$

LOAD CASE 1-Loading Conditions & Combinations:

Unfactored Shear At Top of Key: $V(y) := V_F(y)$

Ultimate Shear At Top of Key:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L) *[Table E-9, Load Case 1A]*

$$V_{u.1}(y) := 2.2 \cdot \left(V_F(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$V_{u.2}(y) := 1.6 \cdot V_F(y)$$

Controlling Case: $V_u(y) := \max\left(V_{u.1}(y), V_{u.2}(y)\right)$

$$V_u\left(H_{key}\right) = 7.18 \cdot kip \qquad \boxed{V_{u.1}\left(H_{key}\right) = 7.18 \cdot kip}$$

Unfactored Moment At Top of Key: $M(y) := M_F(y)$

Ultimate Moment At Top of Key:

EM 1110-2-2104 Serviceability Load Combination: 2.2 (EH + Hs + L)

$$M_{u.1}(y) := 2.2 \cdot \left(M_F(y)\right)$$

ASCE Load Combination 1: 1.4 (D + F) + 1.6 H

$$M_{u.2}(y) := 1.6 \cdot M_F(y)$$

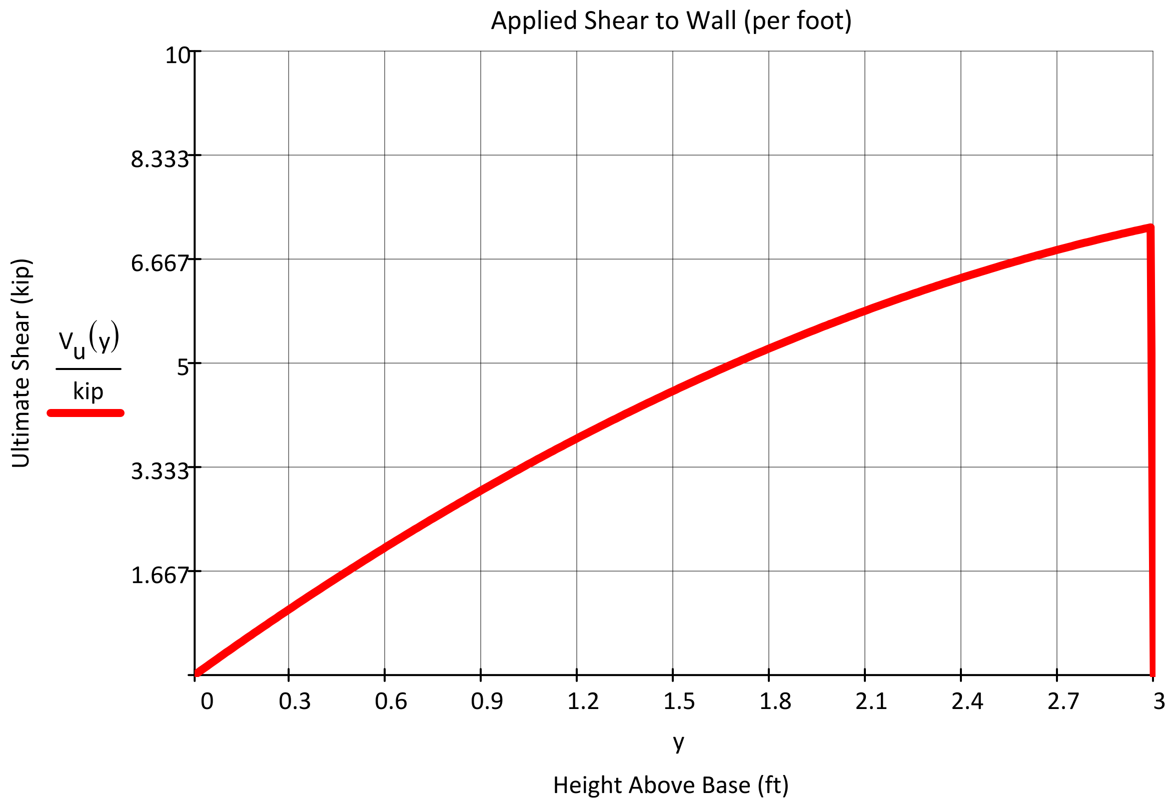
Controlling Case: $M_u(y) := \max\left(M_{u.1}(y), M_{u.2}(y)\right)$

$$M_u\left(H_{key}\right) = 11.29 \cdot kip \cdot ft \qquad \boxed{M_{u.1}\left(H_{key}\right) = 11.29\, ft \cdot kip}$$

Load Summary

Applied Load:

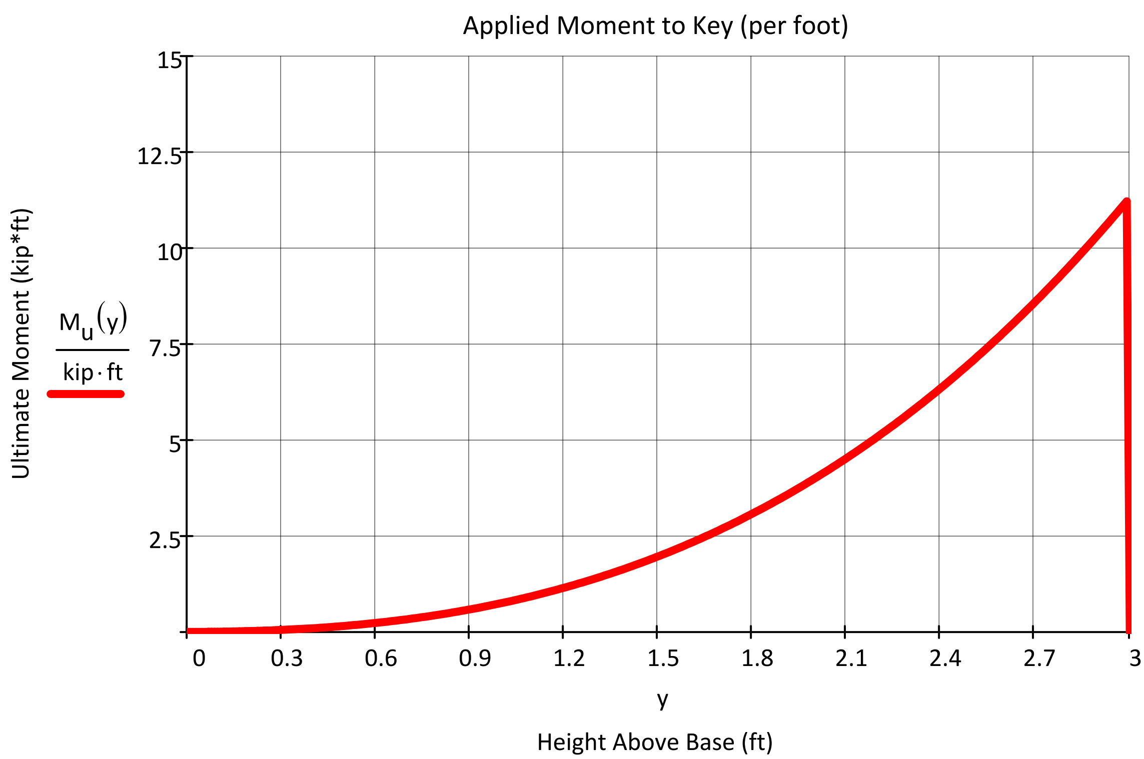
Ultimate Shear at Top of Key: $V_u(H_{key}) = 7.18 \cdot kip$



Load Summary Cont'd

Controlling Applied Load:

Ultimate Moment at Top of Key: $M_u(H_{key}) = 11.29 \cdot kip \cdot ft$



Key Concrete & Rebar Strength Design

Concrete Properties:

Specified Compressive Strength of Concrete: $f'_c = 4500\,psi$

Specified Yield Strength of Reinforcement: $f_y := 60ksi$

Strength Reduction Factor for Tension-Controlled Sections: $\phi_t := 0.90$ *(ACI 318-14, Table 21.2.1*

Strength Reduction Factor for Shear: $\phi_v := 0.75$ *(ACI 318-14, Table 21.2.2)*

Whitney Stress Block Factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ 0.65 & \text{if } f'_c > 8000psi \\ 0.85 - 0.05 \cdot \dfrac{f'_c - 4000psi}{1000psi} & \text{otherwise} \end{cases}$$

$\beta_1 = 0.83$ *(ACI 318-14, Table 22.2.2.4.3)*

Horizontal Reinforcement:

Bar Size and Spacing: $Size_{SH} := 4$ $sSH := 6in$
(Channel & Embankment)

Reinforcement Clear Cover: $clrS := 3in$

Diameter & Cross-Sectional Area of Bars: $dbSH := d_{b_{Size_{SH}}} = 0.50 \cdot in$ $AbSH := A_{b_{Size_{SH}}} = 0.20 \cdot in^2$

Area of Reinforcement:

$$A_{sSH} := AbSH \cdot \frac{12in}{sSH} = 0.40 \cdot in^2$$

$$m := \left(t_{key.B} - t_{key.T}\right) \div H_{key} = 0.00$$

Section Depth at a Height 'y' from the base: $t_{key}(y) := t_{key.T} + m \cdot y$

Depth to Centroid of Embankment Reinf: $d_{SH}(y) := t_{key}(y) - clrS - 0.5 \cdot dbSH$

Reinforcement Ratio: $\rho_{SH}(y) := A_{sSH} \div \left(b_w d_{SH}(y)\right)$

Key Concrete & Rebar Strength Design Cont'd

Vertical Reinforcement:

Bar Size and Spacing: (Embankment) $Size_{SV} := 4$ $sSV := 6in$

Diameter & Cross-Sectional Area of Bars: $dbSV := d_{b_{Size_{SV}}} = 0.50 \cdot in$ $AbSV := A_{b_{Size_{SV}}} = 0.20 \cdot in^2$

Area of Tension Reinforcement: $A_{sSV} := AbSV \cdot \frac{12in}{sSV} = 0.40 \cdot in^2$

$m := \left(t_{key.T} - t_{key.B}\right) \div H_{key} = 0.00$

Section Depth at a Height 'y' from the base: $t_{key}(y) := t_{key.B} + m \cdot y$

Depth to Centroid of Reinforcement: $d_{SV}(y) := t_{key}(y) - clrS - dbSH - 0.5 \cdot dbSV$

Reinforcement Ratio: $\rho_{SV}(y) := A_{sSV} \div \left(b_w\, d_{SV}(y)\right)$

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Key Strength Capacity

Key Capacity (1-foot Design Strip) - SHEAR CAPACITY:

One Way Shear Strength:
ACI 318-14, §22.5.5.1)

$$V_c(y) := 2\sqrt{f'_c \cdot psi} \cdot b_w \cdot d_{SV}(y)$$

$$V_c(0ft) = 13.28 \cdot kip$$

Design Shear Capacity:

$$\phi V_n(y) := \phi_V \cdot \left(V_c(y)\right)$$

$$\phi V_n(0ft) = 9.96 \cdot kip$$

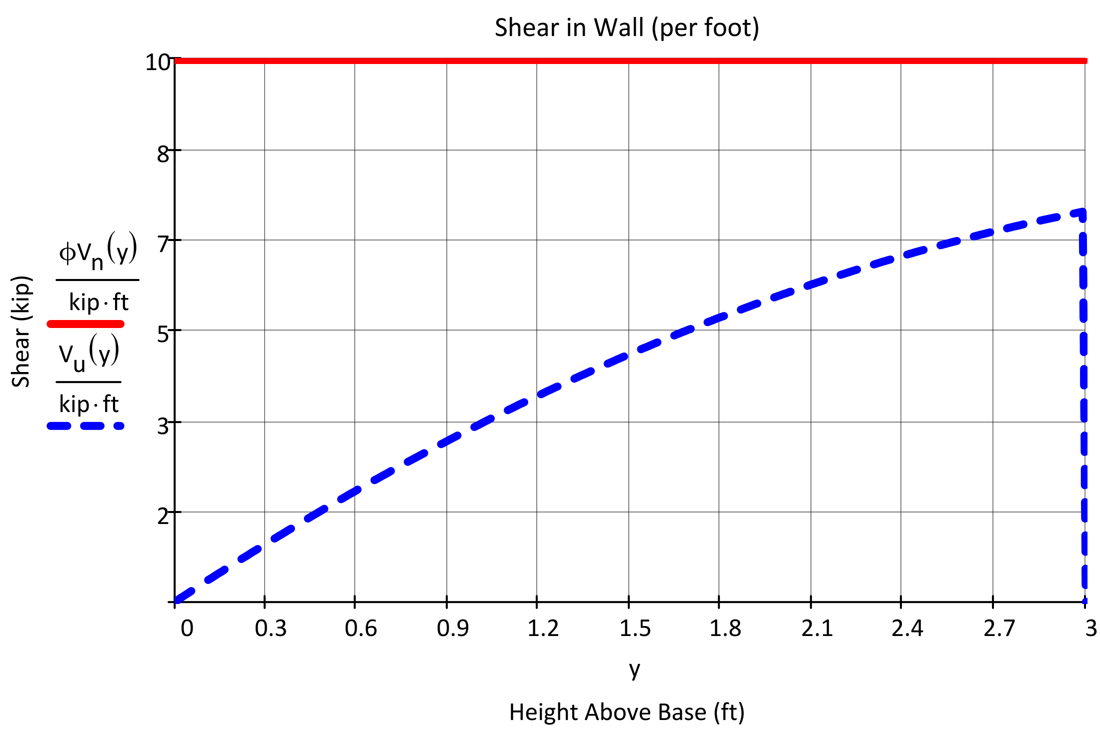
$$V_u(0ft) = 0.00 \cdot kip$$

Demand Capacity Ratio:

$$DCR_V(y) := \frac{V_u(y)}{\phi V_n(y)}$$

$$DCR_V\left(H_{key}\right) = 0.72$$

< 1.0 therefore section is okay for shear strength



Key Strength Capacity Cont'd

Key Capacity (1-foot Design Strip) - MOMENT CAPACITY:

Tension Face Nominal Strength:
(ACI 318-14)

$$M_n(y) := \rho_{SV}(y) \cdot f_y \cdot b_w \cdot d_{SV}(y)^2 \cdot \left(1 - 0.59 \cdot \rho_{SV}(y) \cdot \frac{f_y}{f'_c}\right)$$

Design Moment Capacity:

$$\phi M_n(y) := \phi_t \cdot M_n(y)$$

$$\phi M_n(0) = 14.38\,ft \cdot kip$$

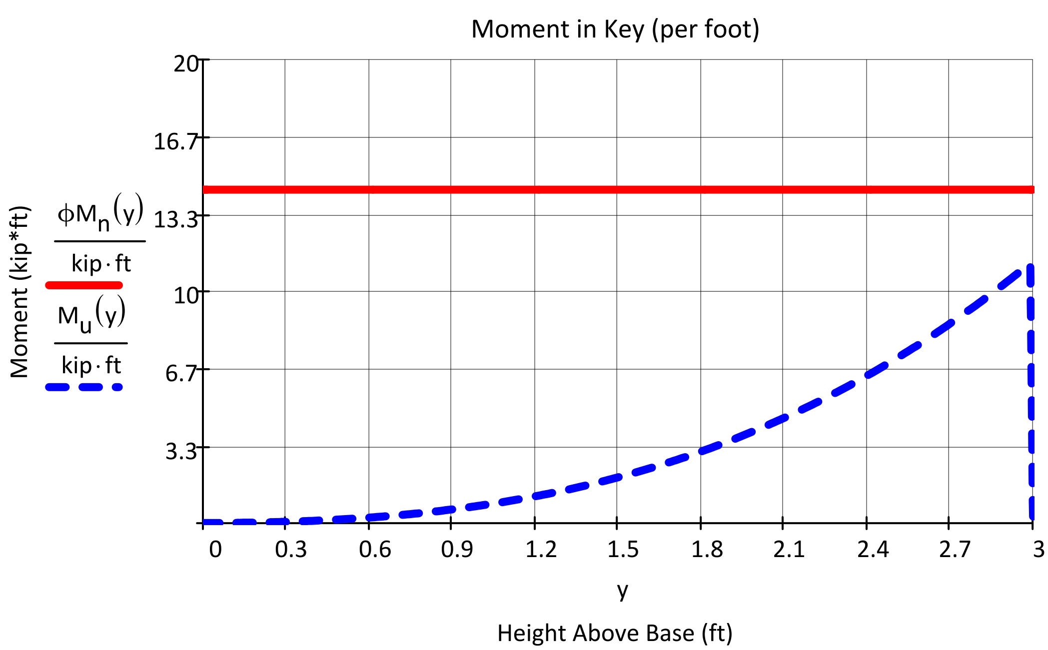
$$M_u\left(H_{key}\right) = 11.29 \cdot kip \cdot ft$$

Demand Capacity Ratio:

$$DCR_M(y) := \frac{M_u(y)}{\phi M_n(y)}$$

$$DCR_M\left(H_{key}\right) = 0.79$$

< 1.0 therefore section is okay for flexural strength



Key Rebar Design

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Maximum Spacing:
(ACI 318-14, §7.7.6.2.1)

$$s := \min\left[18in, 5\cdot 0.5\left(t_{key.B} + t_{key.T}\right), 12in\right] = 12.00\cdot in$$

Minimum Area of Steel Ratio:
(ACI 318-14, Table 7.6.1.1)

$$\rho_{ACI} := \begin{cases} 0.0020 & \text{if } f_y < 60ksi \\ \max\left[\left(0.0018\cdot 60ksi \div f_y\right), 0.0014\right] & \text{otherwise} \end{cases} = 0.0018$$

Minimum Area of Steel Ratio:
(USACE §2-8)

$$\rho_{USACE} := \begin{cases} 0.003 & \text{if } L_{CJ} < 30ft \\ 0.004 & \text{if } 30ft \le L_{CJ} \le 40ft \\ 0.005 & \text{otherwise} \end{cases} = 0.0040$$

Minimum Area of Steel Ratio:

$$\rho_{max} := \max\left(\rho_{ACI}, \rho_{USACE}\right) = 0.0040$$

Minimum Steel Area:
(USACE §2-8)

$$A_{s.min} := \rho_{max}\cdot b_w\cdot 0.5\cdot\left(t_{key.B} + t_{key.T}\right) = 0.58\cdot in^2$$

Shrinkage and Temperature Reinforcement (S&T): ACI 318-14 & USACE §2-8

Vertical Reinforcement:

Bar Size and Spacing of Reinforcement: $Size_{SV} = 4.00$ $sSV = 6.00\cdot in$

Diameter & Cross-Sectional Area of Bars: $dbSV = 0.50\cdot in$ $AbSV = 0.20\cdot in^2$

Area of Reinforcement: $A_{sSV} = 0.40\cdot in^2$

$$\boxed{\frac{A_{s.min}}{2A_{sSV}} = 0.72}$$ < 1.0 therefore okay

Horizontal Reinforcement:

Bar Size and Spacing of Reinforcement:
(Embankment & Channel) $Size_{SH} = 4.00$ $sSH = 6.00\cdot in$

Diameter & Cross-Sectional Area of Bars: $dbSH = 0.50\cdot in$ $AbSH = 0.20\cdot in^2$

Area of Reinforcement: $A_{sSH} = 0.40\cdot in^2$

$$\boxed{\frac{A_{s.min}}{2\cdot A_{sSH}} = 0.72}$$ < 1.0 therefore okay

Key Rebar Design Cont'd

Minimum Flexural Steel: (ACI 318-14, §9.6.1.2)

Vertical Reinforcement:

$$A_{sV.min} := \min\left(\frac{3\sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d_{SV}(H_{key}), \frac{200 \cdot psi \cdot b_w \cdot d_{SV}(H_{key})}{f_y}\right) = 0.33 \cdot in^2$$

$$\boxed{\frac{A_{sV.min}}{A_{sSV}} = 0.83}$$

< 1.0 therefore okay

Ductility Check: (ACI 318-14, §7.3.3.1 & §9.3.3.1)

Reinforcement Ratio Provided:
(Vertical)

$$\rho_{pV} := \frac{A_{sSV}}{d_{SV}(H_{key}) \cdot b_w} = 0.0040$$

Reinforcement Ratio Provided:
(Horizontal)

$$\rho_{pH} := \frac{A_{sSH}}{d_{SH}(H_{key}) \cdot b_w} = 0.0038$$

Depth to Neutral Axis:
(Vertical)

$$cV := \frac{\rho_{pV} \cdot f_y \cdot d_{SV}(H_{key})}{0.85 \cdot \beta_1 \cdot f'_c} = 0.63 \cdot in$$

Depth to Neutral Axis:
(Horizontal)

$$cH := \frac{\rho_{pH} \cdot f_y \cdot d_{SH}(H_{key})}{0.85 \cdot \beta_1 \cdot f'_c} = 0.63 \cdot in$$

Minimum Strain in Tension Steel at Nominal Strength: $\varepsilon_{t.min} := 0.004$

Calculated Strain in Tension
Steel at Nominal Strength:
(Vertical)

$$\varepsilon_{tV} := 0.003 \cdot \left(\frac{d_{SV}(H_{key})}{cV} - 1\right) = 0.04$$

(Horizontal)

$$\varepsilon_{tH} := 0.003 \cdot \left(\frac{d_{SH}(0ft)}{cH} - 1\right) = 0.04$$

$$\boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tV}} = 0.11} \quad \boxed{\frac{\varepsilon_{t.min}}{\varepsilon_{tH}} = 0.10}$$

< 1.0 therefore okay

Spillway Sliding Stability Analysis - Upper Black Creek Reservoir

References:

- Design Drawings, Upper Black Creek Replacement Spillway, GEI Consultants, 2020
- Army Corps of Engineers EM 1110-2-2100, "Stability Analysis of Concrete Structures," Dec 2005

Structural Analysis Summary

The following structural analysis checks the sliding stability of the spillway at Upper Black Creek Reservoir Dam. The sliding stability is checked based on the general wedge analysis as prescribed in EM 1110-2-2100. The inclined portion of the spillway downstream from the contraction joint was assumed to be subject to longitudinal sliding. This check was performed for the extreme seismic case, not based on detailed site-specific data.

Check spillway structure for sliding stability

Unit weight of concrete: $\gamma_{conc} := 150pcf$

Angle of inclination: $\theta := 26.57deg$

10,000yr Horizontal Seismic Coefficient: $k_h := 0.4972$ [USGS Unified Hazard Tool]

Fill Properties (Soil type: SM)

Angle of internal friction: $\phi_f := 30 \cdot deg$

Unit weight of backfill: $\gamma_{fill} := 130pcf$

Angle of inclined backfill: $\alpha_f := 0deg$

At-rest earth pressure coefficient: $k_0 := 1 - \sin(\phi_f) = 0.50$

Active earth pressure coefficient: $k_a := \tan(45deg - 0.5 \cdot \phi_f)^2 = 0.33$

Passive earth pressure coefficient: $k_p := \tan(45deg + 0.5 \cdot \phi_f)^2 = 3.00$

Base/Soil friction coefficient: $\delta := 0.70$ *NAVFAC, Design Manual 7.2*

Bottom Wall Thickness: $t_{wall.B} := 1ft + 6in$

Top Wall Thickness: $t_{wall.T} := 1ft + 0in$

Footing width: $B := 8.0ft$

Toe projection: $Toe := 3.5ft$

Heel projection: $Heel := B - Toe - t_{wall.B} = 3.00\,ft$

Footing thickness: $t_{base} := 2.0ft$

Slab thickness (inclined portion): $t_{slab.1} := 1.0ft \div \cos(\theta) = 1.12\,ft$

Slab thickness (stilling basin): $t_{slab.2} := 2.0ft$

Key depth below slab (chute slab): $d_{key.slab} := 3.0ft$

Key depth below slab (stilling basin): $d_{key.still} := 3.0ft$

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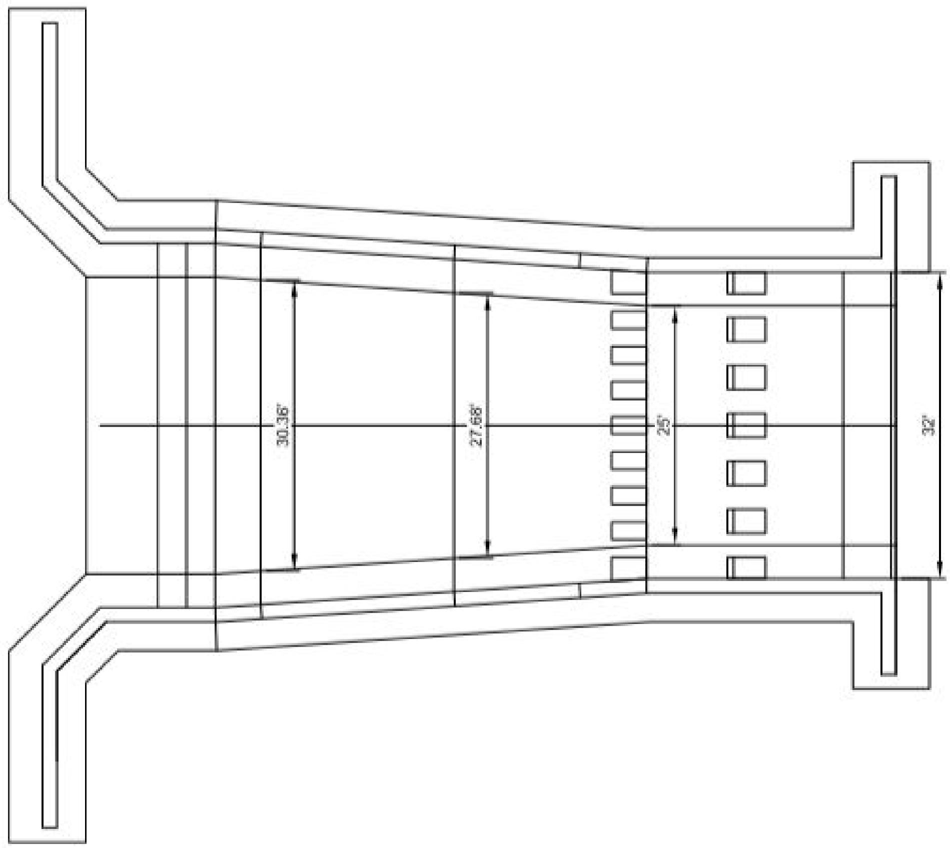
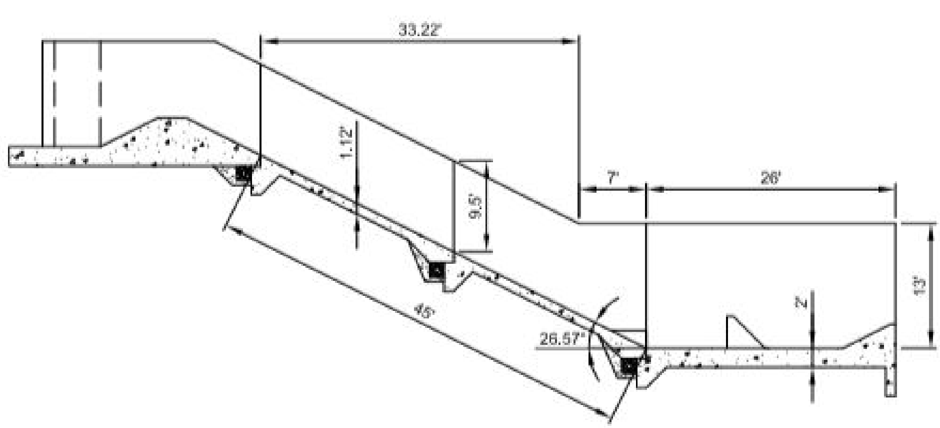
Spillway Design Plan and Profile





Calculate vertical loads

Concrete volume:

Training Wall Volume:

$$V_{tw.1} := 2\cdot\left(33.22ft\cdot 9.5ft\cdot\frac{t_{wall.B}+t_{wall.T}}{2}\right) = 788.98\cdot ft^3$$

$$V_{tw.2} := 2\cdot\left(7ft\cdot\frac{9.5ft+13ft}{2}\cdot\frac{t_{wall.B}+t_{wall.T}}{2}\right) = 196.87\cdot ft^3$$

$$V_{tw.3} := 2\cdot\left(13ft\cdot 26ft\cdot\frac{t_{wall.B}+t_{wall.T}}{2}\right) = 845.00\cdot ft^3$$

Wall Footing Volume:

$$V_{wf.1} := 2\cdot\left(40ft\cdot B\cdot t_{base}\right) = 1280.00\cdot ft^3$$

$$V_{wf.2} := 2\cdot\left(26ft\cdot B\cdot t_{base}\right) = 832.00\cdot ft^3$$

Slab Volume:

$$V_{sl.1} := 40ft\cdot t_{slab.1}\cdot\frac{30.36ft+25ft}{2} = 1237.94\cdot ft^3$$

$$V_{sl.2} := 25ft\cdot 26ft\cdot t_{slab.2} = 1300.00\cdot ft^3$$

End Sill:

$$V_{es.1} := 7.65ft^2\cdot 32ft = 244.80\cdot ft^3$$

Baffle Piers:

$$V_{bp.1} := 2\left(7.78ft^2\cdot 2.25ft\right) + 5\left(7.78ft^2\cdot 2.5ft\right) = 132.26\cdot ft^3$$

Chute Blocks:

$$V_{cb.1} := 2\left(3.36ft^2\cdot 2.25ft\right) + 7\left(3.36ft^2\cdot 1.83ft\right) = 58.16\cdot ft^3$$

Calculate vertical loads - continued

Total Volume (inclined portion): $V_{conc.inc} := V_{tw.1} + V_{tw.2} + V_{wf.1} + V_{sl.1} + V_{cb.1}$

$V_{conc.inc} = 3561.95 \cdot ft^3$

Structure weight (inclined portion): $W_{conc.inc} := V_{conc.inc} \cdot \gamma_{conc} = 534.29 \cdot kip$

Total Volume (stilling basin): $V_{conc.still} := V_{tw.3} + V_{wf.2} + V_{sl.2} + V_{bp.1} + V_{es.1}$

$V_{conc.still} = 3354.06 \cdot ft^3$

Structure weight (stilling basin): $W_{conc.still} := V_{conc.still} \cdot \gamma_{conc} = 503.11 \cdot kip$

Volume of soil over footing:

Wall footing: $V_{s.wf.1} := 2 \cdot (40ft \cdot Heel \cdot 8.5ft) = 2040.00 \cdot ft^3$

$V_{s.wf.2} := 2 \cdot (26ft \cdot Heel \cdot 8.0ft) = 1248.00 \cdot ft^3$

Total Volume (inclined portion): $V_{soil.inc} := V_{s.wf.1} = 2040.00 \cdot ft^3$

Soil weight (inclined portion): $W_{soil.inc} := V_{soil.inc} \cdot \gamma_{fill} = 265.20 \cdot kip$

Total Volume (stilling basin): $V_{soil.still} := V_{s.wf.2} = 1248.00 \cdot ft^3$

Soil weight (stilling basin): $W_{soil.still} := V_{soil.still} \cdot \gamma_{fill} = 162.24 \cdot kip$

Calculate driving forces

Summation of vertical loads: $V_{sum.inc} := W_{conc.inc} + W_{soil.inc} = 799.49 \cdot kip$

$V_{sum.still} := W_{conc.still} + W_{soil.still} = 665.35 \cdot kip$

Seismic load: $H_{d.inc} := k_h \cdot V_{sum.inc} = 397.51 \cdot kip$

$H_{d.still} := k_h \cdot V_{sum.still} = 330.81 \cdot kip$

Calculate resisting forces

Passive earth pressure:

Resistance to bottom of downstream shear key under chute slab:

$$F_{F.T1} := 3 \cdot \left[0.5 \cdot k_p \cdot \gamma_{fill} \cdot \left(t_{slab.1} + d_{key.slab}\right)^2\right] \cdot 32ft = 317.47 \cdot kip$$

Resistance to bottom of shear key at stilling basin:

$$F_{F.T2} := 0.5 \cdot k_p \cdot \gamma_{fill} \cdot \left(t_{slab.2} + d_{key.still}\right)^2 \cdot 32ft = 156.00 \cdot kip$$

Calculate factor of safety against sliding

Factor of safety:

$$FS_{sl} = \frac{\left[\left(V_{sum.inc} \cdot \cos(\theta) - H_{d.inc} \cdot \sin(\theta)\right) + V_{sum.still}\right] \cdot \delta}{H_{d.inc} \cdot \cos(\theta) + V_{sum.inc} \cdot \sin(\theta) - \left(F_{F.T1}\right) \cdot \cos(\theta) + H_{d.still} - F_{F.T2}} = 1.4$$

> 1.3 for seismic load, therefore okay

Sponsor / Owner:

[Please describe ownership of the reservoir. Type of corporate entity. Source of revenue. Ability to take on debt.]

Blue Lake Reservoir Company is a Non-Profit corporation established under the Colorado Non-Profit Corporation Act and Article 42 of title 7. This Corporation shall have perpetual existence. At this time it does not have any source of revenue and its object is to receive and hold title to the reservoir and its Water rights and be separate from Summit Trust. Summit Trust provide funds to this company for all Dam and Reservoir needs.
(Provided by: David Knowlton, Blue Lake Reservoir Company)

Water Rights:

[Please describe water rights associate with the reservoir.]

All Water and Water Rights associated with, appurtenant to or historically used in connection with reservoir, including but not limited to the decree entered by Summit County District Court on March 10, 1952, in case No. 1806, which adjudicated the Reservoir for 139.81 acre feet for propagation and culture of fish, resort, boating, domestic, and Power purposes with a priority date of August 10, 1940, Reservoir Priority No. 79, the decree entered by the Federal District Court for the District of Colorado in Case 1806 which made 139.81 acre feet of the Reservoir final and Absolute, and then in 2009 the decree entered by the District Court Water Division 5 Case No. 06CW99 which changed 288.53 AF of Water Rights from the Lower Black Creek Reservoir to the Reservoir to match the actual Storage Capacity of the Reservoir of which is now the total of 428 acre feet that is final and absolute.
(Provided by: David Knowlton, Blue Lake Reservoir Company)

Permits:

[Please list all permits and approvals necessary prior to construction.]

At this point once NWP is received from the Corps you should be set for environmental permits – but I believe SHPO review is currently holding up the Corps' issuance of the NWP. I'll send an email to the Corps today to see if they've heard anything.

The only County permit that may be required is for stormwater control during construction, typically something the contractor acquires before construction starts.
(Provided by: Sarah Skigen-Caird, GEI Consultants)