Hogchute (AKA Carson) Reservoir Dam Rehabilitation

Mesa County, Colorado
DAM ID 420127
CONSTRUCTION NUMBER C-0454A
Water Division 4
Water District 42

Prepared for:

City of Grand Junction

February 2021

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Engineer's Certification

I hereby certify that I am a Professional Engineer licensed in the State of Colorado and that this report was prepared by me or under my direct supervision.

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Contents

Page No.

Project Summary 1
Hydrologic Hazard Evaluation 2
Hydrology Study 2
Geotechnical Reports 2
Embankment Design 3
- Embankment Stability 4
- Seepage Modeling 7
- Two-Stage Filter Design 9
- Filter Diaphragm 9
- Embankment Freeboard 10

Spillway Design 11
- Existing Spillway Channel 11
- Design Objectives 12
- Hydraulic Model Approach 15
  - Survey and Topography 15
  - Existing Conditions Model 15
  - Proposed Conditions Model 19
- Proposed Hydraulic Model Results 21
- Discharge Rating Curve 23
- IDF Flood Routing Using Hydraulic Model 24
- Spillway Weir Structural Design 25

Outlet Design 26
- Design Objectives 26
- Design Description 26
- Outlet Capacity 27
- Air Venting 31
- Intake Buttress Structural Design 32
- Trash Rack Structural Design 33
- Gate and Operator 34
- Energy Dissipation 34
- Impact Basin Structural Design 35

CIPP Liner 37
Replacement Steel Pipe 37
Pipe Encasement 37
Instrumentation and Monitoring Plan 38
Dewatering 39
Proposed Approach 39
Hydrology 39
Phase 1 Dewatering Simulations 40
Phase 2 Dewatering Simulations 41

List of Appendices

Appendix A Hydrologic Hazard Evaluation
Appendix B Hydrology Study
Appendix C Geotechnical Reports
Appendix D Drawings and Specifications
Appendix E Slope Stability and Seepage Modeling
Appendix F Filter and Drain Design
Appendix G Freeboard Analysis
Appendix H Site Photos
Appendix I Spillway Hydraulic Calculations
Appendix J Structural Calculations
Appendix K Outlet Hydraulics
Appendix L Early Warning System Feasibility Study
Appendix M Dewatering Calculations

List of Figures

Page No.

Figure 1. Spillway weir and channel 12
Figure 2. Incised spillway channel and dense vegetation 12
Figure 3. Roaring River after failure 13
Figure 4. Pine Creek rapid on Arkansas River 14
Figure 5. Depth variant Manning's n values for the channel and dense willows 17

Figure 6. Hydrographs for upstream boundary conditions 17
Figure 7. Rating curves for downstream boundary conditions 18
Figure 8. Existing conditions peak velocity for the 100- and 10,000-yr flow events 19
Figure 9. Buried rock wall locations 20
Figure 10. Proposed low flow channel 21
Figure 11. Existing and proposed 10-year flow paths 22
Figure 12. Proposed condition peak velocity for the 100- and 10,000-yr flow events 23
Figure 13. Existing and proposed spillway weir rating curves 24
Figure 14. Outlet works rating curve comparisons 29
Figure 15. Drawdown flows and pool elevations: Gate fully open, concurrent steady inflow of 10 cfs 31

List of Tables

Page No.

Table 1. Huddleston-Berry Soil Parameters 3
Table 2. Embankment Stability Load Conditions 4
Table 3. Embankment Model Parameters 5
Table 4. Embankment Stability Results 6
Table 5. Seepage Model Parameters 7
Table 6. Piezometer Readings 8
Table 7. Wind-wave setup and runup inputs and results 10
Table 8. Historical wind speed data from nearby sites 11
Table 9. Material Coverages 16
Table 10. Model control settings 18
Table 11. Existing conditions results 18
Table 12. Rock sizing for buried rock walls 20
Table 13. Proposed condition results 22
Table 14. IDF flood routing results computed using proposed-condition 2D hydraulic model 24
Table 15. Spillway stability results 26
Table 16. Final outlet works rating curve 30
Table 17. Buttress structural stability results 33
Table 18. Energy dissipation Type VI Impact Basin stability results 36
Table 19. Comparison of computed 2-yr storm depths against those extracted from NOAA Atlas 14 40
Table 20. Hogchute inflow hydrograph characteristics 40
Table 21. Results of Phase 1 dewatering simulations 41
Table 22. Estimated monthly inflows for Hogchute Dam 43
Table 23. Results of Phase 2 dewatering simulations 43

Project Summary

Hogchute Dam is an approximately 56-ft earth embankment structure located in Mesa County, CO. The dam is owned and operated by the City of Grand Junction. With an upstream contributing area of approximately 11 mi^2 and a normal-pool storage of about 520 ac-ft, the dam is primarily operated for water supply for the City. The dam was constructed in 1947 and needs various embankment, outlet works, and spillway repairs and improvements. The City retained Ayres to complete the design of these repairs and improvements. In general, the project consists of the following:

- Reservoir drawdown, streamflow diversion, and dewatering.
- Demolition of the existing low-level outlet works intake structure.
- Construction of a new low-level outlet works intake structure.
- Downstream embankment excavation.
- Construction of a 2-stage filter and toe drain system on the downstream side of the embankment.
- Rehabilitation of the existing 30-inch outlet works pipe with segments of new steel pipe and a cured-in-place pipe liner.
- Downstream embankment backfill and restoration.
- Demolition of the existing low-level outlet works outfall structure.
- Construction of a new low-level outlet works energy dissipation structure.
- Construction of a new concrete overflow spillway weir.
- Construction of buried rock walls and berms along the overflow spillway channel to provide protection against erosion.
- Installation of instruments and electronics to support a real-time flood early warning system.
- Site restoration and reservoir refill.

The City plans for construction to take place in Summer and Fall 2021.

Hydrologic Hazard Evaluation

A hydrologic hazard evaluation was completed in March 2020 by Ayres Associates and recommended the Hogchute Dam's spillway be classified high hydrologic hazard. With a high-hazard rating, the spillway must pass the 0.01% annual exceedance probability (10,000-year) flood. This determination was made using the Reclamation Consequence Estimating Methodology. A copy of the final report is included in Appendix A.

Hydrology Study

Ayres completed a hydrology study in June 2020 which was submitted to SEO for review (Appendix B). This includes a Probable Maximum Flood (PMF) study completed in October 2019 and an appended memorandum containing results from frequency-based design storm simulations. The design inflow event is the flood resulting from the 10,000-year precipitation event. The inflow design flood hydrograph was developed using a HEC-HMS model representing the 11-square-mile watershed as 13 subbasins, including storage and potential failure at three upstream dams. Subbasin loss and unit hydrograph parameters were assigned following guidance in the SEO's *Hydrologic Basin Response Parameter Estimation Guidelines.* A 2-hour, 0.01% annual exceedance probability storm time series was developed using the Colorado-New Mexico Regional Extreme Precipitation Study publication and software tool. The resulting design flood inflow hydrograph has a peak discharge of 5,930 cubic feet per second (cfs) and a total volume of 1,048 acre feet.

Geotechnical Reports

In January 2019 RJH Consultants Inc. complete a geotechnical investigation of the Hogchute dam. This investigation included seven soil borings, six of which were converted to piezometers, laboratory testing of select samples and logging of soil borings. A copy of RJH's report is included in Appendix C.

From the 2019 RJH report the general geology consists of Quaternary-age colluvium overlying tertiary-age Green River Formation Bedrock. Published maps do not show faults in the site vicinity, however, the southern edge of the Grand Mesa is defined by an escarpment above the flat-lying valley below. The site is located near the top of the escarpment. The site geology consists of colluvial deposits with basalt outcrops forming cliffs to the west, north and east. A general site geology map is included in the RJH report.

Four of the soil borings were installed in the embankment and three borings were installed near the toe of the dam. The embankment fill is described as clayey sand with gravel (SC) clayey gravel with sand (GC) clayey sand (SC) and poorly graded gravel with clay and sand (GP-GC). Three insitu permeability tests were conducted and the calculated vertical hydraulic conductivity was 1.2 x 10-4 cm/s and horizontal hydraulic conductivity values ranged from 8.0 x 10-6 to 2.7 x 10-6 cm/s. Complete results are found in the RJH report. The findings of the report were used to construct SEEP/W and SLOPE/W models to be discussed in latter sections of this report.

Huddleston-Berry of Grand Junction CO provided Ayres with seismic parameters, liquification susceptibility and design soil parameters in an August 2020 letter report. The results from the Huddleston-Berry report are summarized in Table 1.

Table 1. Huddleston-Berry Soil Parameters

0.2-second Maximum Considered Earthquake (MCE)	0.277g
Liquefaction Susceptibility	Not susceptible
Φ	32 degrees
C	0
γ_{moist}	125 pcf
γ_{sat}	130 pcf
$\gamma_{bouyant}$	68 pcf
$\delta_{sliding}$	21 degrees
Net bearing capacity for static load	2000 psf
Net bearing capacity for seismic load	2666 psf

Huddleston-Berry also observed installation of three test pits in the area of the auxiliary spillway the Huddleston-Berry November 2019 report is in Appendix C. Soils in the three test pits are described as 1.5 to 2.0 feet of rich organic matter over a dense to very dense clayey sandy with gravel to clayey gravel with sand soils. Laboratory tests confirmed the soil description and the soils from two samples were slightly to moderately plastic. The anticipated post constructing settlement will be less than 0.5 inches and total settlements will be less than 1.0 inch.

Appendix C also contains geotechnical information included with the 1947 design drawings that were based on borings completed prior to the dam's construction.

Embankment Design

The existing embankment based on recent surveys and the 1947 plans is a 56-foot high structure with a crest width of 18 feet. The embankment has a low permeability earthen core and 3:1 upstream and 2:1 downstream slopes overlaid with a cobble, gravels and boulders rock shell. The 1947 plans show an 8-inch toe drain that outlets to the existing outlet works. Seepage and wet areas at the toe indicate the existing toe drain is not functioning and does not protect the embankment core from piping of embankment material.

The existing embankment will be modified by removing and stockpiling the existing rock and cobble shell, removal of the existing toe drain system and removal of 75 feet of the downstream section of the 30 inch diameter outlet pipe. The downstream section of 30-inch pipe will be replaced with a new length of 30 inch pipe, a filter diaphragm will be placed at the upstream end of the replaced pipe section and a two stage seepage filter will be constructed along the toe of the embankment. The stockpiled rock and cobble fill will be reinstalled on the downstream slope. Drawings of the proposed repairs are included in Appendix D and design details are summarized in the following sections.

Embankment Stability

To evaluate factors of safety for the modified embankment, we constructed a SLOPE/W model based on available geotechnical data from previous investigations (summarized previously) and embankment geometry from the 1947 plans and subsequent topographic surveys. The dam embankment was analyzed at Stations 20+60 and 22+50 (the approximate limits of construction modifications) and at Station 21+50 (adjacent to the 30-inch outlet conduit). Load conditions incorporated into the model are summarized in Table 2. Sudden drawdown elevation was determined using the drawdown water elevation at the pipe intake.

Table 2. Embankment Stability Load Conditions

Load Case	Headwater Elevation (feet)	Tailwater Elevation (feet)
Steady seepage with maximum storage pool – Upstream and Downstream Slopes before and after construction	9895.0	Model-computed downstream toe elevation for stations 20+60 and 22+50. 9840.5 for Station 21+50.
Steady seepage with surcharge pool – Downstream Slope	9900.4	Model-computed downstream toe elevation for stations 20+60 and 22+50. 9848.5 for Station 21+50.
Sudden drawdown from maximum storage pool – Upstream Slope	9895.0 to 9855.0	---
Earthquake with maximum storage pool – Upstream and Downstream Slopes	9895.0	Model-computed downstream toe elevation for stations 20+60 and 22+50. 9840.5 for Station 21+50.

For subsurface data, Ayres used geotechnical information from the RJH and Huddleston-Berry Geotechnical Data reports, as well as soils data included with the 1947 plans. Soil properties were determined using field observation and testing, laboratory testing, and published correlations and estimations for soils based on field and laboratory information. Stratigraphy was developed by a professional geologist. The soil engineering properties are summarized in Table 3.

Table 3. Embankment Model Parameters

Material	Maximum Dry Unit Weight (pcf)	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Moist Phi (degrees)	Saturated Phi (degrees)	Cohesion (psf)
1. Embankment Fill: (GC to SC), Medium Dense to Dense Clayey Sand with Gravel to Clayey Gravel with Sand	130	116	125	34	32	0
2. Colluvium: (CL to SC), Sandy Lean Clay with Gravel/Sand to Clayey Sand with Gravel	NA[3]	106[1]	130	36	34	0
3. Riprap: (Gravel to Boulders) Cobble and Rock Fill	NA[4]	125	135	39	38	0
4. Type I Filter Material	NA[4]	125	134	36	34	0
5. Type II Filter Material	NA[4]	NA	140[2]	39	38	0

1. Laboratory Dry unit weight, no sample collected above phreatic surface.
2. Filter material assumed to be saturated continuously.
3. Material is not expected to be graded or modified in proposed conditions.
4. Material is not to be compacted during placement of material in proposed conditions.

The embankment was analyzed using the 2019 GeoStudio SLOPE/W computer program. Circular failure surfaces were investigated in accordance with the Morgenstern-Price method which satisfies all conditions of force and moment equilibrium. Analysis of the failure slip surfaces show that most of the slices occur above the phreatic surface. Therefore, we conservatively evaluated the embankment under consolidated drained conditions with no effective cohesion. We used averages of suggested parameter values obtained from reference publications to avoid outlier influence. To maintain a conservative approach, we reduced friction angles by 1 to 2 degrees during slip surface analyses that were intercepting the phreatic surface. Embankment fill to be removed and replaced during proposed conditions was installed at the maximum dry weight for placement to accommodate for minimum required compaction efforts. Final proposed conditions were analyzed with a surcharge load of 500 pcf for maintenance traffic on the crest of the dam. Earthquake loading used a horizontal seismic coefficient of 0.1062 based on 2/3 of the site PGA of 0.1593. The results of the stability analyses are presented in Table 4 for Stations 20+60, 21+50, and 22+50.

Table 4. Embankment Stability Results

Load Case	Upstream	Downstream	USACE Required Safety Factor	Bureau Required Safety Factor
Station 20+60				
Steady-state seepage normal pool	1.9	1.5	1.5	1.5
Maximum surcharge pool	---	1.4	1.4	1.2
Rapid drawdown from maximum storage pool	1.3	---	1.1	1.2
Seismic at normal pool	1.2	1.1	1.0	1.0
Station 21+50				
Steady-state seepage normal pool	1.6	1.5	1.5	1.5
Maximum surcharge pool	---	1.3	1.4	1.2
Rapid drawdown from maximum storage pool	1.2	---	1.1	1.2
Seismic at normal pool	1.1	1.2	1.0	1.0
Station 22+50				
Steady-state seepage normal pool	1.7	1.6	1.5	1.5
Maximum surcharge pool	---	1.6	1.4	1.2
Rapid drawdown from maximum storage pool	1.2	---	1.1	1.2
Seismic at normal pool	1.2	1.3	1.0	1.0

The results of the loading conditions are evaluated in accordance with Rule 7.4.2.5 of State of Colorado Dam Safety and Dam Construction Rules and Regulations. Rule 7.4.2.5 references either Design Standards No. 13, Chapter 4, Bureau of Reclamation 2011 or EM 1110-2-1902, U.S. Army Corps of Engineers, 2003. As can be seen in Table 4, factor of safety requirements are met except for the maximum surcharge pool loading case on the downstream side of the embankment at station 21+50, where the computed factor of safety of 1.3 does not meet the USACE requirement of 1.4, though it does meet the Bureau requirement of 1.2. However, the USACE methodology allows for factors of safety less than 1.4 at maximum pool for existing embankments based on actual performance over a period of time. Over 70 years of good historical performance of the existing embankment and the relatively short duration the pool would be at its maximum height during the IDF suggest that the evaluated factors of safety are adequate for the proposed condition.

For the downstream slope stability analyses, shallow surface failures that do not reach the phreatic surface of the embankment are not considered dam safety threats. The shallow failures may pose a maintenance challenge during wet years, but only failure circles that intercepted the crest mid-line, phreatic surface, or that extended deeper than five feet were considered as realistic threats to the safety of this wide embankment. In other words, any numerically-indicated shallow slope failures (while

theoretically possible) would be mitigated by the wide crest width, the width of embankment at the elevation of the phreatic surface, and conservatism in the soil properties. While shallow failure circles with factors of safety under 1.5 may be numerically possible, only those slope failures with credible correlation with dam safety threats were included in the report's respective appendices and tables.

Output files from the SLOPE/W model and complete documentation of slope stability calculations are included in Appendix E.

Seepage Modeling

Seepage through the embankment and under/around appurtenances was analyzed to support design of seepage controls to prevent internal erosion, piping, and external sloughing and to provide for adequate stability of the dam. Results of the seepage analyses formed the basis for design of the filters and toe drain, which are further elaborated in the Two-Stage Filter section below.

In general, the seepage properties were developed using published values based on soil descriptions and classifications, in-situ testing results, and correlations from gradation and estimated anisotropic ratios. The seepage properties incorporated into the SEEP/W model are summarized in Table 5.

Table 5. Seepage Model Parameters

Material	Permeability, Kx (cm/sec)	Saturated Water Content (%)	Residual Water Content (%)	Anisotropy Ratio (Ky/Kx)	Liquid Limit
1. Embankment Fill: (GC to SC), Medium Dense to Dense Clayey Sand with Gravel to Clayey Gravel with Sand	2.7E-6	29	25.4	0.3	28
2. Colluvium: (CL to SC), Sandy Lean Clay with Gravel/Sand to Clayey Sand with Gravel)	3.1E-3	25.2	23.3	0.52	27
3. Riprap: (Gravel to Boulders) Cobble and Rock Fill	2.5E-2	16	11.4	1.0	NP
4. Type I Filter Material	3.05	20	5	0.3	NA
5. Type II Filter Material	36.58	14	5	0.3	NA

The piezometer wells installed during the 2018 geotechnical investigation, listed in Table 6, provided a phreatic elevation of seepage throughout the embankment. Comparisons of observed piezometer well readings to the simulated SEEP/W phreatic surface indicate an unmodeled seepage drawdown affecting the phreatic surface in the observed field conditions. This finding was corroborated based on the results of the rising- and falling-head tests for permeability and anisotropy ratio values on the embankment fill material. Therefore, proposed conditions were addressed as if the seepage drawdown is corrected from construction efforts. The horizontal seepage values were generally uniform and were kept for SEEP/W modeling. The anisotropy ratios values are referenced from the conservative FERC creep theory, where the horizontal permeability (Kx) is three times as great as the vertical permeability (Ky), resulting in an anisotropy ratio value of 0.3 (Ky/Kx). This inconsistency was not identified in the underlying native

colluvium deposits. This correction results in the proposed phreatic elevations to be above existing piezometer readings for their respective headwater elevations. All load cases were analyzed using this correction.

As mentioned earlier, proposed improvements are expected to address the drawdown seepage indicated in the piezometers' phreatic readings. This correction is expected to address the system as a gradual steady-state seepage in the SEEP/W modeling. Seepage analysis shows a reduction in seepage velocity and acceleration throughout the system to reduce the potential of future piping and propagating any existing impairments within the embankment. Thus, the proposed efforts shall reduce or control uplift pressures as indicated in the slope stability factor of safety values. Seepage will be controlled, filtered, and monitored at the designated seepage exit points to reflect the performance of the improvements and to identify any erosion over time. Piezometers shall reflect the corrected phreatic surface modeled to indicate the remediated embankment seepage gradient.

SEEP/W output and supporting calculations are included in Appendix E.

Table 6. Piezometer Readings

Piezometer Wells	**7/28/2018 (NHW EL 9895)**	**9/06/2018 (NHW EL 9895)**	**9/20/2018 (PW EL 9885)**	**9/22/2018 (PW EL 9885)**
B-101 (9902.2') Lat: 38.995296 Long: -108.109759 Total Depth: 77.5'	9871.7	9870.2	9863.1	9863.1
B-102A (9902.1') Lat: 38.995677 Long: -108.109800 Total Depth: 48.0'	9868.5	9868.5	6864.2	9863.9
B-103 (9901.8') Lat: 38.996095 Long: -108.109881 Total Depth: 30.0'	9879.3	9879.1	9879.3 (DRY)	9879.3 (DRY)
B-104 (9846.1') Lat: 38.995285 Long: -108.110292 Total Depth: 33.0'	NA	NA	9838.6	9840.5
B-105A (9865.7') Lat: 38.995632 Long: -108.110160 Total Depth: 73.6'	NA	NA	9856.0	9855.9

Two-Stage Filter Design

A two-stage filter was designed for the toe of the dam following the 2011 Bureau of Reclamation Design Standard No. 13, Chapter 5. The Type I filter material was designed using a sample from boring B-101, collected at a depth of 51 to 65 feet below top of embankment and a sample from B-103, collected from a depth of 10 to 17.5 feet below top of embankment. Using the Bureau methodology, a C-33 fine aggregate sand is acceptable for the Type I filter material. Following the Bureau methodology, the Type II filter material was designed based on the selection of C-33 fine aggregate as the Type I material. Using this approach, a Type II material with a gradation meeting ASTM D448 No. 89 is compatible with the Type I filter. Computations documenting the design procedure for the filter material are provided in Appendix F.

To convey seepage from the toe of the dam to the outlet works, a 6-inch perforated HDPE pipe will be bedded inside the Type II filter material. Toe drains will be located on both the right and left downstream embankment as shown on the Drawings. Cleanouts will be located at the upstream end and approximate midpoint of each drain. Where the perforated pipes discharge out of the embankment at the toe, solid HDPE pipe will be used. Each outlet pipe will discharge into the outfall impact basin where seepage can be collected and measured.

Using the SEEP/W model results and historical measured seepage flow rates from the existing toe drain system, we estimated a maximum seepage flow rate of approximately 12 gpm (0.03 cfs) from each side of the embankment. We then sized a 6-inch diameter drainpipe in the two stage filter, which can pass approximately 232 gpm (0.516 cfs) at full flow in the flattest portion of the pipe. Since only 0.03-cfs discharge is needed, the 6-inch drainpipe provides a factor of safety of at least 17.

The D50 particle size of the ASTM D448 No. 89 Type II filter material into which the toe drainpipe is bedded is approximately 4 mm. Therefore, following FEMA guidance, perforated portions of the toe drainpipe will require holes no larger than 4 mm (approximately 5/32”). We determined that eight 5/32” perforations drilled at equal intervals around the pipe circumference will be required every 2 inches wherever perforation is required. Perforations of this size and frequency will result in a factor of safety of 14.4.

Supporting calculations for toe drainpipe sizing and perforations are included in Appendix F.

Filter Diaphragm

The existing outlet pipe has anti seep collars placed along the pipe. Anti-seep collars are no longer the preferred practice to prevent migration of embankment materials along pipes through embankment dams. To reduce the risk of piping along the outlet pipe a sand filter diaphragm will be placed around the new 30 inch steel pipe from the junction of the new and old pipe, extending downstream approximately 25 feet. The filter diaphragm is also designed following the Bureau’s Design Standard No. 13, Chapter 5. The Type I and Type II filter material in the diaphragm will be the same as used for the toe drain. Filter diaphragm dimensions were determined based on the Bureau’s design standard and are shown on the attached Drawings.

Embankment Freeboard

We assessed freeboard during the maximum pool of the IDF for Hogchute using the wind-wave setup and runup computation procedure described in Reclamation Design Standards No. 13, Chapter 6 (September 2012). To develop a design wind speed for this computation, we used historical wind data from Grand Junction Walker Field Airport and the wind speed adjustment approach described in Reclamation Technical Release No. 69 (February 1983). We did not consider wind direction in our review of historical wind data. Rather, as a conservative approach, we analyzed historical wind data as if all extreme winds occurred at a critical direction to maximize wind wave setup and runup on the embankment. Design wind speed and wave setup and runup computations are provided in Appendix G. Key inputs and results are provided in Table 7. As can be seen, the maximum pool elevation resulting from wave action during the IDF is 9900.76 ft, which is 0.24 ft below the top of the embankment. At the normal pool elevation of 9895 ft, computed wave setup and runup under the same conditions total 1.38 ft. Under both IDF and normal-pool conditions, sufficient freeboard is available to accommodate wind setup and runup without overtopping the embankment.

Table 7. Wind-wave setup and runup inputs and results

Input/Result	Value
Maximum wind velocity (fastest mile of record), mph	80
Design wind speed, mph	69
Fetch length, mi	0.339
Average lakebed elevation, ft plan datum	9883.1
Top of embankment elevation, ft plan datum	9902.0
IDF maximum pool elevation, ft plan datum	9900.4
Minimum wind duration needed to reach maximum wave height, hr	0.22
Wave setup, ft	0.05
Wave runup, ft	1.31
Total wave setup + runup, ft	1.36
Maximum pool elevation resulting from wave action, ft plan datum	9901.76

To confirm the maximum wind velocity used as the basis of our analysis, we also reviewed wind data from Aspen Pitkin County Airport and Leadville Lake County Airport. A comparison of elevations and historical wind speed data from these two sites, in addition to Grand Junction Walker Field Airport, are provided in Table 8. The historical maximum 5-second winds reported in the table are all less than the 80 mph fastest-mile (1-min) wind speed used for our setup and runup calculations despite Grand Junction Airport being at a lower elevation than Aspen and Leadville. This finding confirms that the 80-mph fastest-mile wind speed is conservative for this application.

Table 8. Historical wind speed data from nearby sites

Site	Elevation (ft)	Max 5-Second Wind		Period of Record
		Speed (mph)	Direction (deg)	
Hogchute Dam	9900	--	--	--
Grand Junction Walker Field	4858	72.9	290	1996-2020
Leadville Lake County Airport	9934	69.1	36	1998-2020
Aspen Pitkin County Airport	7820	62	44.1	1998-2019

Spillway Design

Existing Spillway Channel

Ayres staff performed a site visit of the Carson Lake Spillway on June 23, 2020 to inform the hydraulic model and guide spillway design. During the site visit, Ayres assessed and documented important hydraulic controls and features, roughness characteristics, and channel stability indicators. Site photos can be seen in Appendix H.

The spillway channel begins at the weir located immediately north of the main dam embankment. The spillway channel flows west, eventually turning south, until its confluence with Kannah Creek (Figure 1). The spillway channel was likely an ephemeral vegetated valley prior to the construction of the dam. An incised channel has developed within a heavily vegetated riparian corridor because of the introduction of regular flow from Carson Lake (Figure 2). Like Kannah Creek, flow from the spillway channel eroded smaller soil particles away, exposing cobble and boulders that provide stable armoring for the typical range of flows.

The spillway channel is approximately 1,000 feet long. The upper half of the spillway has a slope of approximately 4%, and the lower half is steeper with a slope of over 6%. Bank erosion and incision were apparent at some locations; however, these processes appear to be mostly inactive under normal flow regimes.

A Wolman Pebble Count was also performed to determine typical bed material size at two locations in the spillway channel. The median grain size (D_{50}) was determined to be approximately 7 inches and including boulders in the 3 to 4 foot range.

Figure 1. Spillway weir and channel

Figure 2. Incised spillway channel and dense vegetation

Design Objectives

The design objective for the spillway channel is to protect the weir from failure during high flow events. The primary threat to the weir is failure due to downstream erosion in the form of headcutting, channel incision, or the development of a large scour hole immediately downstream of the weir.

Headcut potential of the spillway channel was evaluated using the SITES computer model developed by the U.S. Department of Agriculture. The model simulates flow through a spillway channel using a three-phase erosion process including vegetation removal, concentration erosion to develop knickpoints, and deepening and upstream migration of headcuts. It was determined that there is a high potential for headcut progression in the spillway channel during the 10,000-year event.

The existing channel material and riparian vegetation provide some level of erosion protection, but high flows are likely to activate lateral and vertical instabilities. Analyzing the hydraulics, associated erosive forces, and erosion resistance of dense riparian vegetation in steep mountain stream is complex. Practical examples exist that provide insight into the potential risk from extreme flows in steep mountain stream systems as well as potential stability mitigation options for high gradient, high flow channels.

The limitations of riparian vegetation and erosion hazards that exist in high gradient systems is highlighted by the impacts from the 1982 Lawn Lake dam failure. Lawn Lake is an earthen dam that sits at nearly 11,000 feet on the Roaring River in Rocky Mountain National park. The flood caused by the dam break reached a peak discharge of approximately 18,000 cfs. The Roaring River is a high-gradient stream with an average slope of 10% and was noted to be surrounded by dense vegetation prior to the flood. High shear stress and flow velocities caused severe channel erosion and transported significant amounts of debris and bed material downstream. The channel downstream of the dam was widened by tens of feet and scoured between 5 and 50 feet locally. Trees were ripped from the surrounding area and transported downstream. Figure 3 shows Roaring River after the failure occurred. While the peak flow of the Lawn Lake Dam failure is significantly higher than the Carson Lake IDF, it highlights the limitations of riparian vegetation to stabilize high mountain systems in extreme flow events.

Figure 3. Roaring River after failure

While high flows on steep stream systems can be destructive, Colorado also has several large river systems with short steep reaches that remain stable during high flow events. The Pine Creek rapid on the Arkansas River is one example (Figure 4). This rapid has an overall slope of 3% to 4% with localized sections significantly steeper. The bed material is made up of cobbles and boulders. This section of river typically experiences annual flows exceeding 1,000 cfs and has experienced flows over 4,000 cfs multiple times in the last 30 years. While some material has shifted, during high flow events, most of the channel and grade controlling features have remained stable. Large rock material can stabilize steep channels during high flows similar to the design flows for the Carson Lake Spillway.

Figure 4. Pine Creek rapid on Arkansas River

In addition to the design objective of stabilizing the channel and preventing the failure of the spillway weir, the City of Grand Junction prefers a design that preserves the natural setting of the area. The City wants to avoid using concrete and fully armoring the spillway channel with large riprap.

Design Description

The proposed design primarily consists of buried rock trenches with associated raised berms oriented perpendicular to the primary flow path. The existing channel is proposed to be filled in some locations, and the berms include low flow notches to encourage multiple flow paths during lower flows. The purpose of these features is to minimize the possibility of an incised channel that concentrates energy during high flow events.

The buried rock is designed to impede headcut progression. The berms are proposed to be vegetated to help provide stability in regular low flow events. During normal high flow events (10 to 100-year) the rock is designed to remain stable, though erosion and disturbance of vegetation and soil is expected. During higher flow events, including the IDF, it is expected that high shear stresses will begin to erode soil downstream of each berm. During these events, the rock is designed to launch into the anticipated scour hole. As scour occurs downstream of the berm, rock will launch into the depression and armor the

channel to mitigate potential headcut development additional scour. The resulting channel will consist of flatter sections with steep sections armored with large rock, similar to step-pool features in steep river systems observed in Colorado, including downstream along Kannah Creek.

Several rock berms are proposed along the spillway channel from the weir to its confluence with Kannah Creek. Flow from the spillway channel makes a 90° turn at the confluence at Kannah Creek which reduces velocity and associated shear stress. The confluence would likely be a location of deposition of finer material in large flow events and does not require scour countermeasures. A similar sharp bend exists on the Roaring River downstream of the Lawn Lake Dam and, based on aerial imagery, did not experience erosion and was an area of deposition.

It is recommended the spillway channel be inspected following high flow events larger than the 50-year event to assess the stability of vegetation and to determine if any scour develops downstream of the berms. If a large enough event occurs to launch the rock, the launched rock should be inspected to determine if the new rock configuration is sufficient to armor the channel during future events. If the launched rock is too sparse, additional rock may need to be placed to reinforce the rock location. It is not anticipated that the berms need to be reconstructed following large flow events.

Hydraulic Model Approach

A two-dimensional (2D) hydraulic model was developed to assess the existing and proposed condition hydraulics. Aquaveo's SMS version 13.0 was used to develop an SRH-2D version 3.2, developed by the U.S. Bureau of Reclamation. All hydraulic models assume a rigid bed with no sediment transport included in the analysis.

Survey and Topography

The hydraulic model and spillway channel protection design were based off a combination of ground survey and aerial lidar. The City of Grand Junction provided survey data collected by the City in 2019. The lidar was collected in 2016 for Mesa County by Wolpert Inc. under a contract with the U.S. Geological Survey. The lidar bare earth ground surface was assumed to be representative of the channel because depths are generally shallow, and the rock in the channel is typically visible during low flow events. Ayres merged the lidar and survey data using Global Mapper to create a continuous, existing condition terrain surface. A 1-foot grid of this surface was used for the hydraulic model.

The reservoir bathymetry was estimated using the As-Built Vicinity Map and General Plan (Sheet 705-30) dated May 24, 1947. The General Plan layout was georeferenced in Global Mapper and the reservoir contours were traced and converted into a surface.

All topographic data have been transformed into Colorado State Plane Central (U.S. survey feet), North American Vertical Datum of 1988 (NAVD88) for the purpose of analysis and design.

Existing Conditions Model

The existing conditions model extends approximately 80 feet upstream of the spillway crest to approximately 300 feet downstream of the confluence of the spillway and Kannah Creek. The mesh comprises 29,400 elements. Element size ranges from 5 feet in the spillway to 25 feet along the mesh boundary.

A materials coverage was developed to represent the land cover types within the model domain. Material coverages and associated Manning's roughness coefficients (n) were determined using a combination of aerial imagery and observations during the June 2020 site visit. Various sources were used to relate coverage to roughness values. Table 9 shows material types and associated Manning's n values.

Research has shown that roughness in high gradient streams can be dominated by turbulent forces. A depth variant relationship was developed using a combination of Jarrett (1985) and Aberle and Smart (2003) for the existing channel to account for additional resistance due to turbulence. Jarrett's equation was used to establish an upper limit n value during lower flows. A lower limit for n for high flows were calculated assuming near critical conditions (Fr = 0.9) during a preliminary run for the existing conditions 10,000-year event. Then, Aberle and Smart's approach, using relative submergence, was used to create a continuous relationship between depth and roughness and shifted to meet the upper and lower limits.

The roughness of riparian vegetation made up of willows and other shrubs is dependent on flow depth and velocity. During higher flows, willows will lay down and become significantly less rough.

Figure 5 shows the depth variant curves used for the channel and dense willows roughness values.

Table 9. Material Coverages

Material Type	Manning's n	Description
Upland Sparse Vegetation	0.05	Overbank light density scattered scrub, usually sagebrush, over very light cover to dirt
Dense Willows	0.08-0.18	Dense willows, depth variable based on willows lying flat during deeper flows
Riprap	0.05	Artificially placed boulders along slopes of the dam
Gravel Road	0.02	Gravel service road on top of the dam and spillway
Channel	0.034-0.134	Main channel, boulders, steep, depth variable based on Jarrett and Aberle and Smart
Upper Valley Vegetation	0.10	Denser tree and scrub in steep rocky upper reach, pines, oaks, pinyon-juniper scrub

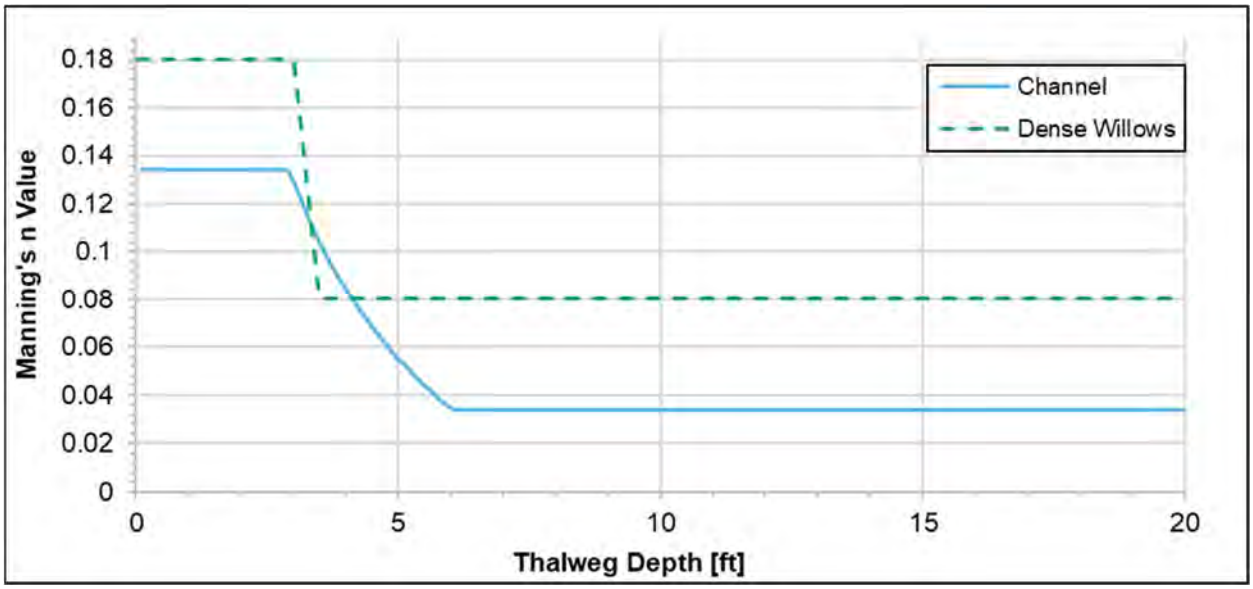

Figure 5. Depth variant Manning's n values for the channel and dense willows

Boundary conditions (BC) coverages included an upstream (US) inflow at the spillway entrance and an outflow water surface elevation at the downstream (DS) end of the model for each storm event. The hydrograph developed by Ayres during the PMF study for the 2-, 10-, 100-, 10,000-year events was used for the upstream boundary condition, depicted in Figure 6. It was assumed that the inflow from the diversion channel and outlet were negligible. Downstream boundary conditions were developed for each simulation using a normal depth rating curve based on a slope (S = 0.048 ft/s) and composite Manning's n at the downstream cross section, depicted in Figure 7. The composite Manning's n values were estimated using an iterative process in which the depth and extents were evaluated.

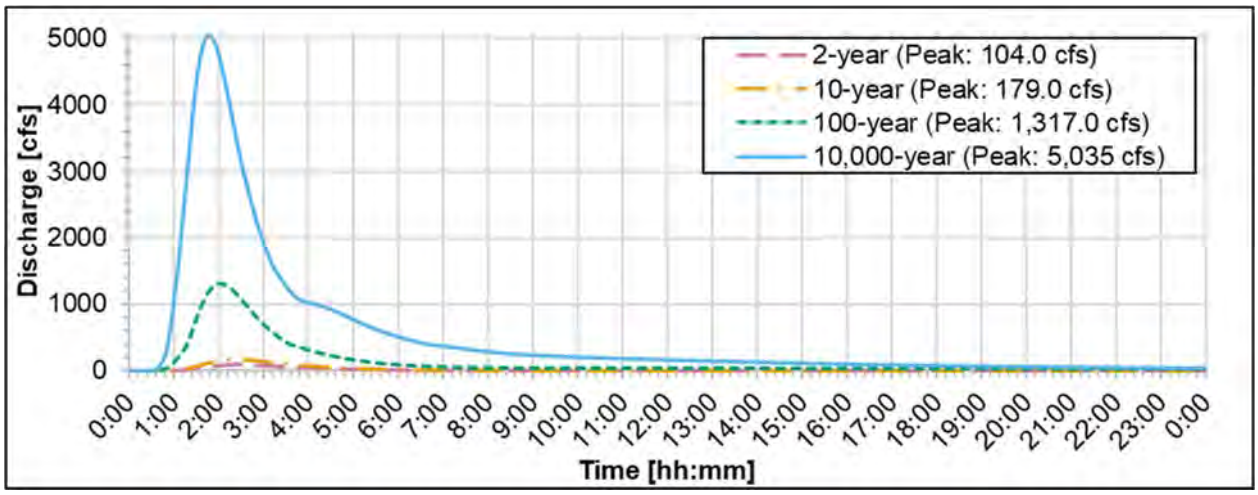

Figure 6. Hydrographs for upstream boundary conditions

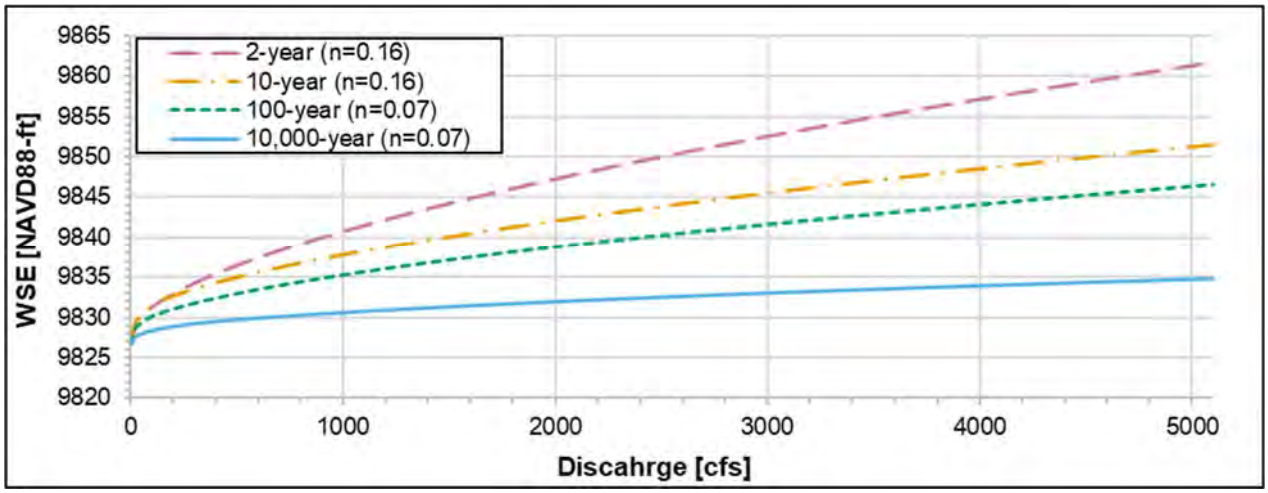

Figure 7. Rating curves for downstream boundary conditions

Appropriate model control settings were determined through an iterative process. Monitoring lines and residual plots were used to quantify and evaluate model stability. Table 10 provides the model control settings for each recurrence interval simulation. A duration of 4.0 hours was considered appropriate for each simulation because it allowed sufficient time for the model to reach the peak flow conditions.

Table 10. Model control settings

Recurrence Interval [year]	Peak Flow [cfs]	Time Step [sec]	Duration [hrs]
2	104.0	0.50	4.0
10	179.0	0.50	4.0
100	1317.0	0.25	4.0
10,000	5035.2	0.25	4.0

Results from the existing conditions model were used to guide the proposed condition design. Table 11 provides peak existing condition results for the 2-, 10-, 100- and 10,000-year events. Peak shear and velocity results indicate that there is a high potential for both lateral and vertical instabilities during high flow events. Figure 8 shows the existing 100- and 10,000-year peak velocity.

Table 11. Existing conditions results

Recurrence Interval [year]	Time [hr]	Peak Velocity [ft/s]	Peak Shear [lb/ft2]
2	2:30	6.0	15.3
10	2:30	7.0	18.6
100	2:06	11.6	36.8*
10,000	1:48	15.2	62.6*

*Areas with high shear will experience erosion which will remove the vegetation thereby decrease the calculated shear stress.

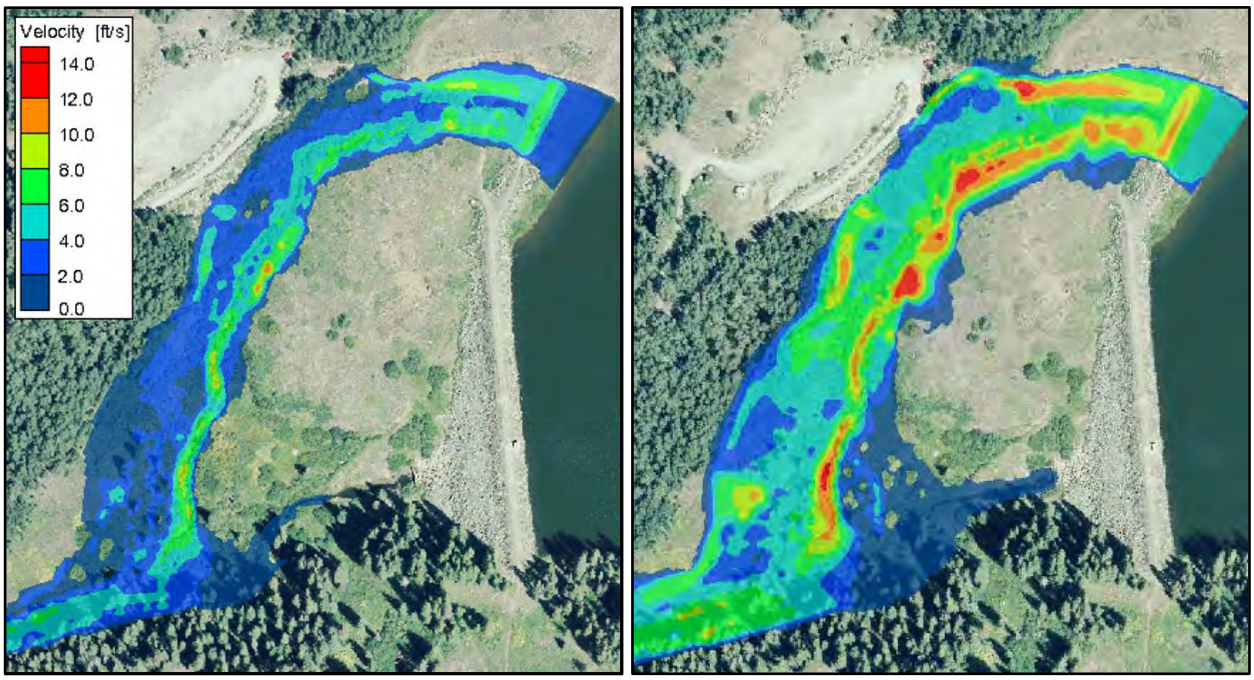

Figure 8. Existing conditions peak velocity for the 100- and 10,000-yr flow events

Proposed Conditions Model

The exiting conditions model results were used to guide decisions for the proposed countermeasure design. The existing conditions hydraulic model indicates a high potential for channel and bank erosion at various points in the channel based on the 10,000-year velocity and shear stress values. In addition, the headcut analysis suggests that headcut progression is likely during the 10,000-year event. Buried rock was determined to be an appropriate measure for preventing scour and headcut progression in the spillway in a way that preserves the natural setting of the site. The proposed design is intended to prevent damage to the spillway during the 100-year event. However, it is anticipated that some maintenance may be necessary following larger than the 100-year events if rocks shift and headcuts begin to develop.

A median rock size (D_{50}) of 24 inches was determined to be adequate based on existing conditions 100-year model results. Peak existing conditions results were assumed to provide rock sizing estimates that were conservative, yet realistic. The proposed condition model shows higher shear stress values at the downstream edges of the proposed berms. The purpose of the berms is to relocate areas of higher energy to areas with sufficient rock to reduce or arrest erosion as it encounters the rock. The rock is intended to shift during high flow events. While the ultimate configuration of the launched rock is difficult to predict, the existing natural channel terrain provides an adequate representation. The Isbash equation, U.S. Bureau of Reclamation method for sizing riprap below a stilling basin, and U.S. Army Corps of Engineers method for steep slopes were used to guide rock sizing decisions. Results are presented in Table 12. Detailed calculations are provided in Appendix I.

Table 12. Rock sizing for buried rock walls

Method	Percent Finer Reference	Rock Size [in]
Isbash	D_{50}	23.3
USBR	D_{50}	22.7
USACE	D_{30}	8.4

The buried rock is proposed to be 4 feet wide by 6 feet deep and is oriented perpendicular to the 100-year flow direction. Each rock trench extends to at least the 100-year flood extents or to where the 10,000-year velocities exceed 4 ft/s, whichever is greater.

It was assumed that the rock will launch to a 1.5 H: 1 V slope which is in the range of what occurs on natural river systems with large flows and material. Since headcuts are expected to occur during the 10,000-year event, the launching elevation was projected at a 0% slope down the existing dominant flow path. This is a conservative approach because the design anticipates headcutting to form and erode the channel at a flat slope until the headcut encounters the rock. Figure 9 shows the locations of the rock trenches. Note that the upstream Rock Trench 1 is at the weir and Rock Trench 2 is at the access road. These locations are closer than needed based on the potential slope but is necessary to protect the weir and access road provides some redundant protection for the weir.

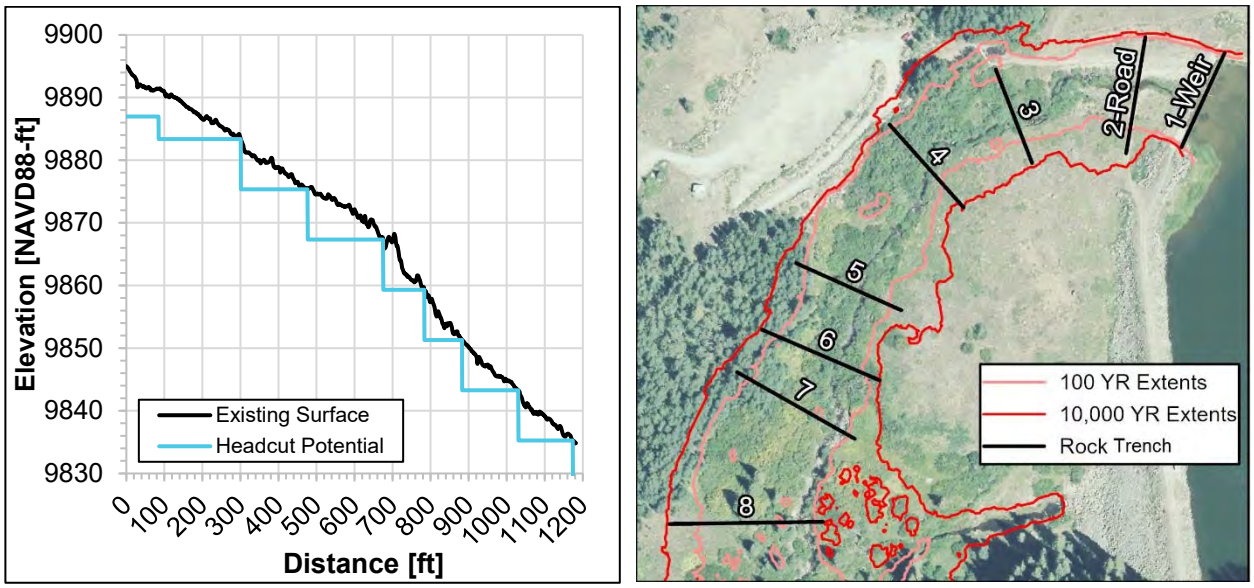

Figure 9. Buried rock wall locations

Each buried rock trench and berm has a low flow channel 10 feet wide and 2 feet deep designed to convey the 2-year or less frequent flow. The low flow notch is designed to encourage more diverse, lower gradient flow path and divert flow away from the existing incised channel in some locations. The existing and proposed flow path is shown in Figure 10. The proposed low flow channel follows a secondary channel that runs adjacent to the current dominant flow path. Grading may be necessary in some locations where the proposed path deviates from the existing flow path. Overtime a new channel will form as the spillway area experiences yearly flows.

Figure 10. Proposed low flow channel

The proposed conditions model updates the existing model to include the proposed berms and low flow notches. The mesh was expanded relative to the existing conditions model because the berms cause shallow flow to spread out around the berm edges in the higher flow events. The proposed mesh is comprised of 36,300 triangular elements. Element size ranges from 4 feet across the top of the rock walls and spillway crest to 25 feet along the mesh boundary.

The existing conditions materials coverage was updated to represent the proposed conditions. The proposed materials coverage assumes that native vegetation will develop in locations where the existing channel will be abandoned and in areas influenced by the berms. Native seed will be planted on the berms to provide additional resistance to erosion and create a natural aesthetic. Additionally, the depth varied channel roughness was applied at the proposed channel location.

Boundary conditions and model control settings for the proposed conditions model are identical to the existing conditions model.

Proposed Hydraulic Model Results

The proposed conditions model results suggest the berms add complexity to the low flow channel paths, reduce erosive forces in unprotected areas during larger events, and divert flow away from the existing incised channel during lower flow events. It was assumed that the rock would remain stable during the 100-year event. However, it can be expected that some of the buried rock will launch into downstream scour holes during the 10,000-year flow. In this case, the launching rock will arrest upstream headcut progression.

The 2 and 10-year flow results show how the proposed berms encourage more diverse flow paths and divert flow away from incised areas of the existing channel. Figure 11 shows the existing and proposed low flow paths at the peak of the 10-year event. The berms encourage flow through the secondary channel, spreading out flow throughout the reach, especially at the downstream end. The combined effect of diverting flow from the existing flow path, increasing the channel length, and spreading out the

flow reduces the erosive forces in unprotected portions of the channel and will encourage more riparian vegetation to form along the varied flow paths.

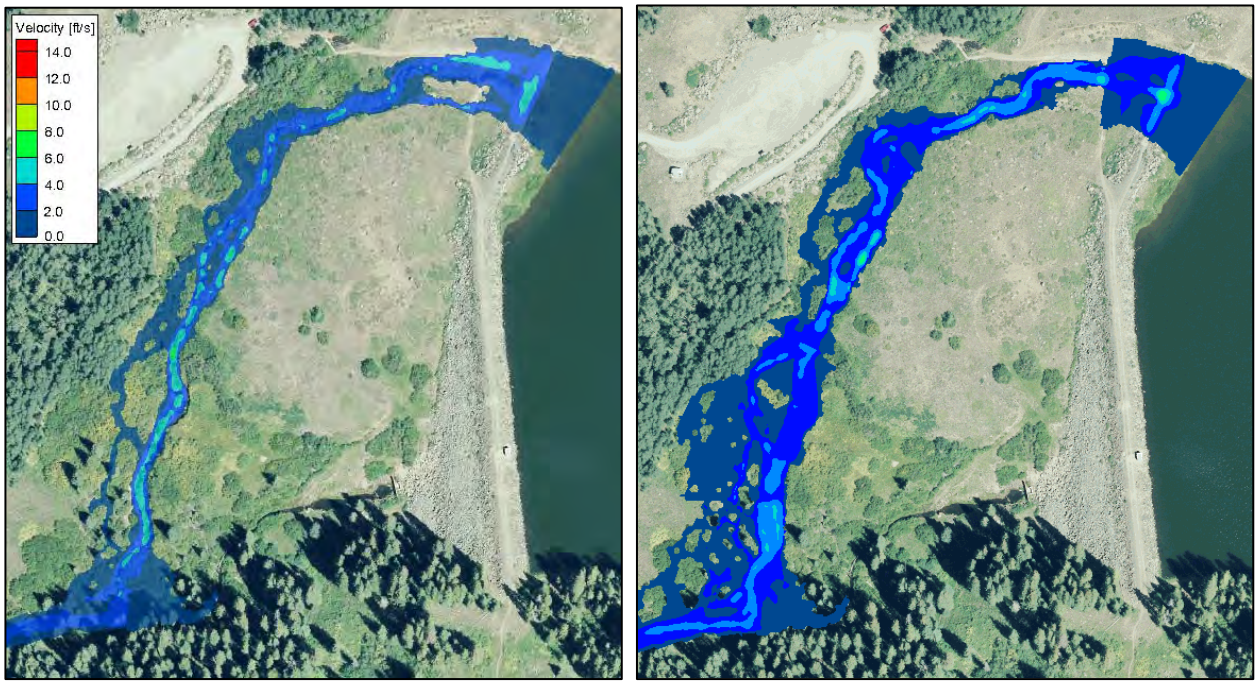

Figure 11. Existing and proposed 10-year flow paths

It is expected that the downstream side of the berms will experience greater shear stress and velocity during large events than compared to the existing conditions. As noted earlier, this is planned to focus the energy where the buried protection exists. Buried rock will launch into downstream scour holes and impede headcut progression. Table 13 provides peak proposed condition results for the 2-, 10-, 100- and 10,000-year events. Unprotected areas in the proposed model experience less shear and lower velocities than the existing conditions. Figure 12 shows the proposed 100- and 10,000-year peak velocity.

Table 13. Proposed condition results

Recurrence Interval [year]	Time [hr]	Peak Velocity [ft/s]	Peak Shear [lb/ft^2]
2	2:30	5.2	21.5
10	2:30	6.7	25.8
100	2:06	11.8	40.9
10,000	1:48	15.0	67.2

Recurrence Interval [year]	Existing Peak Shear [lb/ft^2]	Proposed Peak Shear* [lb/ft^2]	Difference [lb/ft^2]
100	36.8	30.4	6.4
10,000	62.6	48.0	14.6

* Peak shear at locations other than the downstream side of the berms

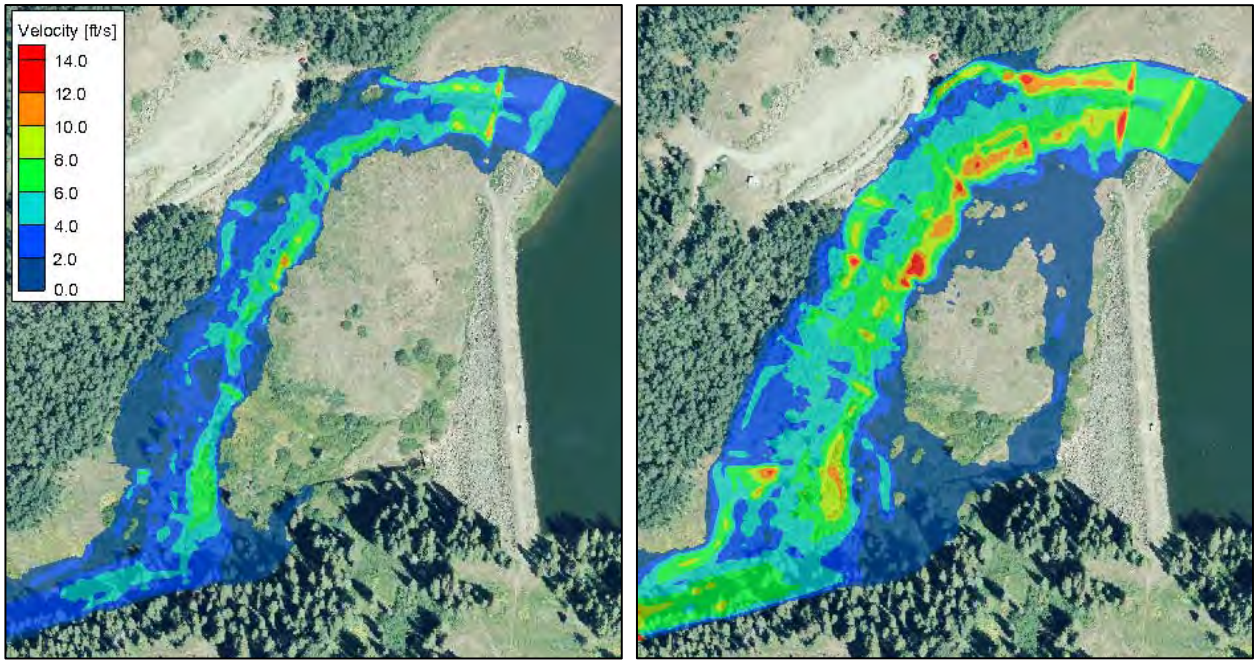

Figure 12. Proposed condition peak velocity for the 100- and 10,000-yr flow events

Discharge Rating Curve

A stage-discharge relationship was developed for the proposed spillway weir. For the proposed conditions rating curve, a monitoring point and line upstream of the weir in the 2D model provided discharge and lake elevations at each time step. Lake elevations were extracted far enough upstream of the weir to be at a location where a level pool was computed. As the reservoir filled, the simulation provided stage and discharge information. The simulation was run under steady-flow conditions through the reservoir until the dam top elevation was reached. Figure 13 indicates that the weir rating curve, when computed using the 2D model, is slightly less efficient than the rating curve provided with the 1947 drawings. This result is because the 2D model accounts for full backwater effects from the spillway channel downstream of the submerged weir, whereas the rating curve shown in the 1947 drawings likely was computed using a simple weir equation with no consideration for backwater effects and submergence.

For reference, we also developed a spillway rating curve using the 2D model for the existing condition, which is also shown in Figure 13. As can be seen, the existing and proposed condition rating curves, when computed using the 2D model, are nearly identical. In both cases, the 2D model-computed curves have slightly less capacity than indicated by the curve provided with the 1947 drawings.

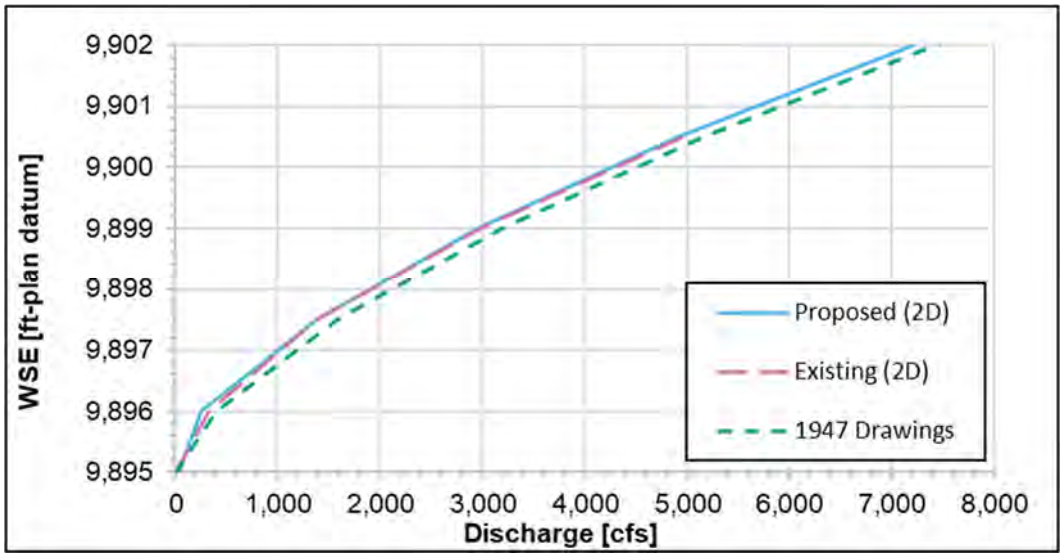

Figure 13. Existing and proposed spillway weir rating curves

IDF Flood Routing Using Hydraulic Model

Utilizing the proposed-condition 2D hydraulic model developed for spillway design, we routed the IDF hydrograph through the reservoir and spillway to determine how the currently understood IDF max pool elevation (9900.4 ft) and peak spillway discharge (5,035 cfs) compared to the results obtained from using the more refined 2D model. For this effort, we added the reservoir to the proposed-condition 2D model using bathymetric data estimated from the original 1947 design plans. We then routed the IDF inflow hydrograph through the reservoir and dam starting with a normal pool elevation of 9895 ft. Results of the routing are summarized in Table 14. As can be seen, the 2D model results in both an IDF maximum pool elevation and peak spillway discharge less than that reported in the 2020 hydrology study (provided in Appendix B). Despite the 2D model's rating curve indicating that the spillway capacity may be less than previously understood (as described in the previous section), the resulting smaller IDF metrics are largely a consequence of 2D routing through the reservoir, as opposed to the level-pool routing used for the 2020 hydrology study. These results suggest that the 9900.4-ft IDF pool elevation used for design is reasonable and conservative.

Table 14. IDF flood routing results computed using proposed-condition 2D hydraulic model

	Hydrology Study	2D Model
Peak inflow (cfs)	5,930	5,930
Peak outflow (cfs)	5,035	4,479
Maximum pool elevation (ft)	9900.4	9900.2

Spillway Weir Structural Design

The spillway consists of concrete I-shaped structure that is almost completely buried in fill.

The following references were used in the design:

1. *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2. *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*.
3. EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, USACOE, 30 Nov. 2016.
4. EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACOE, 1 December 2005.
5. EM 1110-2-2200, *Gravity Dam Design*, USACOE, 30 June 1995.
6. EM 1110-2-2502, *Retaining and Flood Walls*, USACOE, 29 September 1989.
7. Letter: Geotechnical Design Parameters, Carson Lake, Mesa County, Colorado, from Michael A. Berry, PE & 15 September 2020 email.
8. Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.

The unit weights of concrete and water were taken as 150 pcf and 62.5 pcf, respectively. The moist and saturated soil unit weights were taken as 125 pcf and 130 pcf, respectively. The soil backfill and foundation sliding friction angles were taken as 32 degrees and 21 degrees, respectively. Soil cohesion was not considered. The net bearing capacities for static and seismic loads were taken as 2,000 psf and 2,666 psf, respectively.

The seismic evaluation used the Mononobe-Okabe Method. US Army Corps of Engineers (COE) earthquake ground motions are the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE).

The OBE is defined as 1/3 of the site Peak Ground Acceleration (PGA) value of 0.236g that yields 0.08g. The MDE is defined as the site Maximum Considered Earthquake (MCE) of 0.277g. Vertical earthquake acceleration is zero.

Full fill was used on the driving side and full fill less the top three feet was used on the resisting side. Uplift was taken as linear from full headwater to full tailwater.

Sliding stability at the foundation interface was computed using the Shear Friction Factor (SFF) concept.

The design was based on a concrete compressive strength of 4,000 psi and a reinforcement yield strength of 60 ksi (ASTM A615, Grade 60). The concrete reinforcement was based on temperature requirements and structure forces. Mathcad worksheets were used for the design.

The following load cases were considered:

1. Unusual: Construction (dewatered)
2. Usual: Normal Pool (EL. 9895.00 feet NAVD88)
3. Unusual: Infrequent Flood = Q100 (EL. 9897.16 feet NAVD88)

4. Extreme: Construction with OBE
5. Unusual: Coincident Pool = Normal Pool with OBE
6. Extreme: Coincident Pool = Normal Pool with MDE
7. Extreme: Maximum Design Flood = Q10000 (EL. 9900.40 feet NAVD88)

Load cases 1 and 4 were not completed as an independent cofferdam and/or drawdown is required during construction. The results are summarized in Table 15. A complete set of structural computations for the spillway weir is included in Appendix J.

Table 15. Spillway stability results

Load Case	Base In Compression			Sliding	
	Effective Base (%)	Required Effective Base (%)	Maximum Net Base Pressure (psf)	[1]Shear Friction Factor (SFF)	Required SFF
1. Construction	--	--	--	--	--
2. Normal Pool	100.0	100	888	Stable	1.7
3. Infrequent Flood = Q100	100.0	75	1,173	Stable	1.3
4. Construction with OBE	--	--	--	--	--
5. Normal Pool with OBE	100.0	75	1,043	Stable	1.3
6. Normal Pool with MDE	71.9	In Base	1,874	1.34	1.1
7. Maximum Design Flood = Q10000	67.6	In Base	1,994	6.61	1.1

1 Stable noted if resisting forces are greater than driving forces.

Outlet Design

Design Objectives

The existing outlet works at Hogchute Dam consist of two valve-controlled 20-inch steel pipe inlets feeding to a single 30-inch steel pipe through the embankment. The outlet works are currently operational. However, given the age of the outlet works components and a televised inspection indicating degradation of the 30-inch steel outlet pipe, the City has opted to rehabilitate the outlet as part of this effort.

Design Description

The City proposes the following improvements to the outlet works at Hogchute Dam:

- Removal and replacement of the existing intake structure on the upstream side of the 30-inch steel pipe. The proposed intake structure is a concrete buttress fitted with a single hydraulically operated

steel slide gate to control flow into the 30-inch steel pipe. The slide gate will be operated from the top of the embankment by connecting a portable hydraulic pump to the lines leading down to the gate.
- Construction of a steel trash rack to protect 30-inch pipe intake and slide gate.
- Replacement of approximately 100 ft of the existing 30-inch steel pipe: 75 ft on the downstream side and 25 ft on the upstream side. Replacement of pipe on the downstream side is to facilitate construction of a proposed filter diaphragm. Replacement of pipe on the upstream side is to facilitate connection to the proposed outlet works intake structure.
- Encasement of the 30-inch steel pipe in concrete. Per the original design plans, the existing 30-inch steel pipe is completely encased in concrete. Therefore, all replacement pipe will also be encased so that the entire conduit from inlet to outlet is protected.
- Placement of a filter diaphragm along the 30-inch steel pipe to prevent seepage along the conduit. The diaphragm extends a minimum of 8 ft above, 8 ft to the sides, and 4 ft below the pipe for a thickness of 8 ft in the direction of flow (upstream to downstream).
- Lining of the entire 30-in steel pipe run—both new portions and existing—with a cured-in-place pipe (CIPP) lining product.
- Construction of an USBR Type VI impact basin for energy dissipation at the 30-inch pipe outfall.

Outlet Capacity

According to original design plans for Hogchute Dam, the existing outlet works have a release capacity of approximately 130 cfs at normal pool when both 20-inch gate valves are operated fully open. Our design objective was therefore to maintain at least this discharge capacity. Of importance to note is that the City has indicated they historically have only used a small fraction of their release capacity. Generally, no more than 20 cfs are released from the dam's outlet works.

We used two approaches to develop an elevation vs. discharge rating curve for the proposed outlet works:

- A HY-8 model of the 30-in steel pipe through the embankment.
- Application of Bernoulli's theorem for full pipe flow conditions.

The HY-8 model of the 30-inch pipe models the outlet as though it were a culvert. A benefit to using this approach for rating curve development is that the software accounts for the possibility of different governing flow regimes for varying levels of reservoir head.

To model the 30-inch pipe in HY-8, we reduced the pipe diameter from 30 inches to 28 inches to account for the installation of the CIPP liner. While the required thickness of the CIPP liner will be determined by the contractor per the Specifications, for our hydraulic calculations we estimated a liner thickness of 1 inch based on discussions with a vendor.

We set the invert of the pipe inlet to be 9,851 ft and the invert of the pipe outlet to be 9,839.8 ft, consistent with the Drawings. The proposed inlet has a 45-degree beveled edge, and we set the Manning's *n* value to 0.010, again based on CIPP vendor-provided information. For tailwater conditions, we used a normal depth assumption based on an idealized channel cross section estimated from survey data collected downstream of the dam.

HY-8 has no direct way to account for minor losses in its computations. To account for the head loss produced by 68-degree bend at the entrance to the pipe, we calculated an equivalent pipe length by which to increase the actual pipe length to produce the same amount of head loss. With an estimated minor loss coefficient of 0.27 for a 68-degree bend, we computed an equivalent pipe length of approximately 50 ft. Thus, in our HY-8 model, we increased the pipe length from 288 ft to 338 ft. Supporting calculations for the 50 ft of equivalent pipe length are provided in Appendix K.

Adding 50 ft of equivalent pipe length artificially decreases the slope of the outlet conduit from about 3.9% to 3.3% in the HY-8 model. This decrease in slope has a negligible affect on pipe hydraulics for normal-head, full-pipe flow conditions where the pool elevation is greater than approximately 9,880 ft (15 feet below the dam's normal operating pool). At elevations below this, the pipe as represented in HY-8 with 50 ft of additional length results in lower discharges than would result from computing flows with the actual pipe length.

As a check on the rating curve computed using HY-8, we also applied Bernoulli's theorem for full pipe flow conditions to estimate an elevation-discharge relationship for the outlet works. Again, we reduced the pipe diameter from 30-inches to 28-inches to account for the estimated 1-inch thick CIPP liner. Flow computations assume a supported outflow jet at the pipe outfall. Computation of the elevation-discharge relationship using this method is also documented in Appendix K.

In Figure 14, we compare the rating curves computed using the two methods. As can be seen, computed discharges converge at about 9,880 ft—15 ft below Hogchute Dam's normal operating pool of 9,895 ft. We adopted the HY-8 rating curve to serve as the final elevation-discharge relationship for Hogchute Dam because it shows less capacity at lower heads and is therefore more conservative. The final curve, with some minor smoothing implemented, is also plotted in Figure 14 and tabulated in Table 16. At the normal pool elevation of 9,895 ft, we estimate that the proposed outlet works will release 139 cfs with the upstream slide gate fully open. This slightly exceeds the estimated 130-cfs discharge capacity of the existing outlet works.

The DSB requires that outlets have capacity to draw down the reservoir at least 5 ft in 5 days, beginning at the high water line. Using the final proposed outlet rating curve and the HEC-HMS watershed model, we estimated the time required to draw down the reservoir from 9,895 ft (the normal pool elevation) to 9,890 ft with a concurrent steady inflow of 10 cfs. We found the fully open outlet works are capable of this drawdown in less than 18 hours, which easily meets DSB requirements. Furthermore, under the same baseflow condition, the fully open outlet works are capable of completely emptying the reservoir in about 2.5 days. Therefore, the proposed outlet has sufficient capacity for emergency drawdown procedures, per DSB guidelines. Computed drawdown discharge and pool elevation time series are plotted in Figure 15**.**

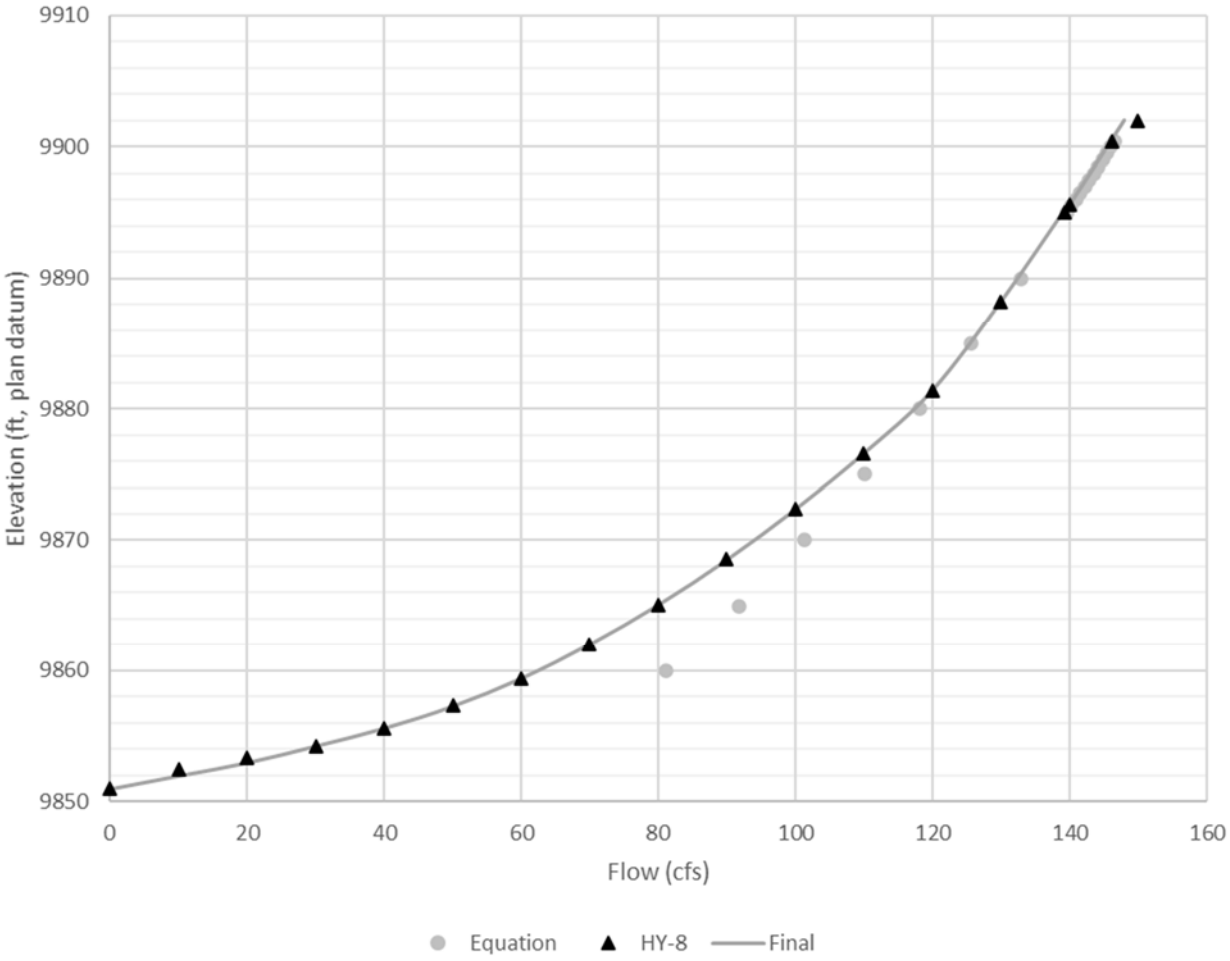

Figure 14. Outlet works rating curve comparisons

Table 16. Final outlet works rating curve

Water Surface Elevation (ft, plan datum)	Flow (cfs)[1]
9851	0
9852	10
9853	20
9854.27	30
9855.63	40
9857.31	50
9859.44	60
9862.07	70
9865.1	80
9868.54	90
9872.38	100
9876.62	110
9881.41	120
9888.23	130
9895	139
9895.6	140
9900.4	146
9902	148

1. Gate to outlet assumed to be fully open. 30-inch steel pipe reduced in size by CIPP lining estimated to be 1 inch thick.

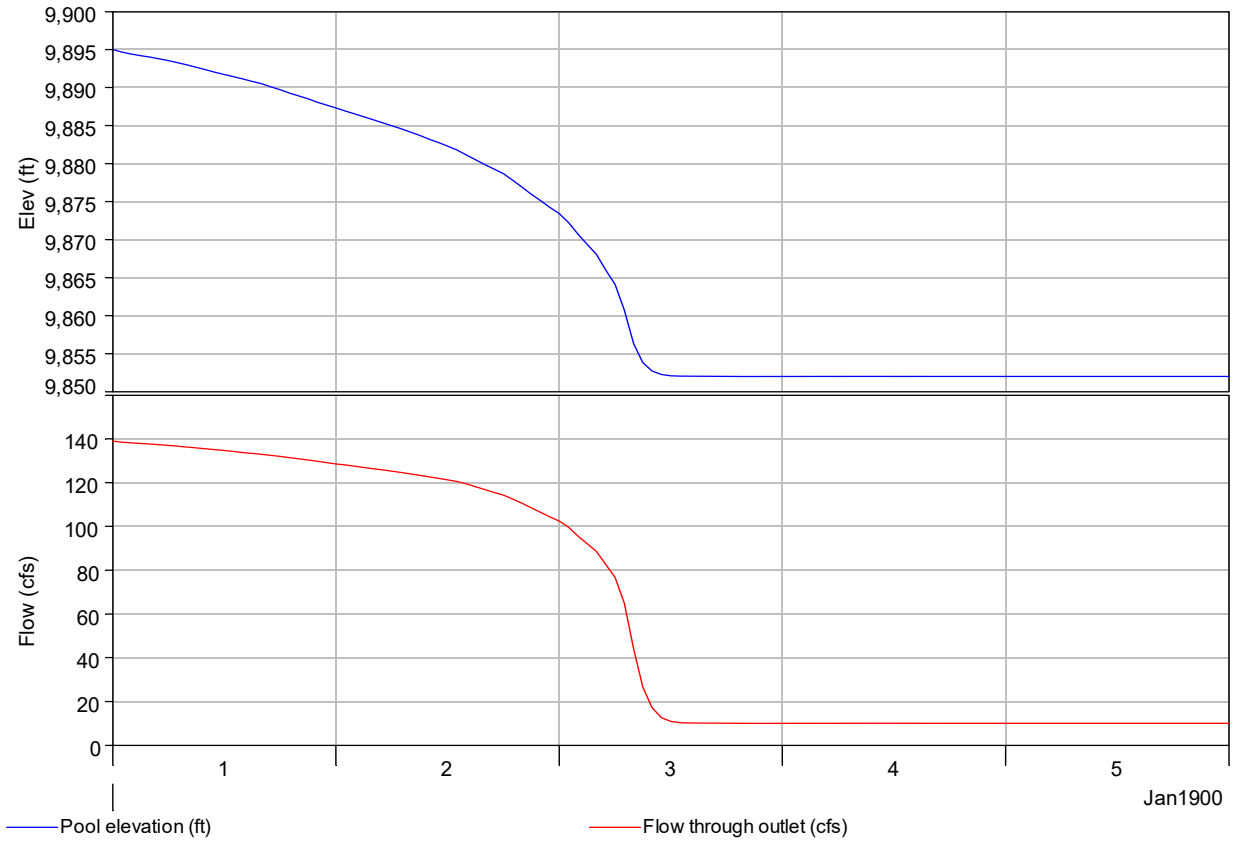

Figure 15. Drawdown flows and pool elevations: Gate fully open, concurrent steady inflow of 10 cfs

Air Venting

According to original design plans, the existing outlet works are equipped with a 4-inch air relief vent. For the proposed outlet works, our calculations indicate a larger 8-inch air relief vent would be more appropriate. An 8-inch air relief vent, running from the top of the dam to the inner bend of the 30-inch intake elbow, will supply air demand with an allowable air velocity of 150 ft/s.

The air vent will be constructed of 8-inch PVC and will be encased in concrete from the outlet works up the face of the dam to the dam top. At the top of dam, the vent will transition to an exposed ductile iron pipe extending vertically above the ground surface. The ductile iron pipe will terminate with two 90-degree bends, and a stainless steel screen will be installed at the mouth to keep debris from being sucked into the pipe.

Air vent calculations are provided in Appendix K.

Intake Buttress Structural Design

The intake structure consists of a mass concrete structure that will encase the new pipe and be the foundation for the slide gate and trash rack. The intake structure is located at the toe on the upstream side of the embankment.

The following references were used in the design:

1. *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2. EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, USACOE, 30 Nov. 2016.
3. EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACOE, 1 December 2005.
4. EM 1110-2-2200, *Gravity Dam Design*, USACOE, 30 June 1995.
5. EM 1110-2-2502, *Retaining and Flood Walls*, USACOE, 29 September 1989.
6. EM 1110-2-6053, *Earthquake Design and Evaluation of Concrete Hydraulic Structures*, USACOE, 1 May 2007.
7. Letter: Geotechnical Design Parameters, Carson Lake, Mesa County, Colorado, from Michael A. Berry, PE & 15 September 2020 email.
8. Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.

The unit weights of concrete and water were taken as 150 pcf and 62.5 pcf, respectively. The moist and saturated soil unit weights were taken as 125 pcf and 130 pcf, respectively. Uniform surcharge was taken as 200 psf. The soil backfill and foundation sliding friction angles were taken as 32 degrees and 21 degrees, respectively. Soil cohesion was not considered. The net bearing capacities for static and seismic loads were taken as 2,000 psf and 2,666 psf, respectively.

The use of at-rest earth pressures required the Seismic Coefficient Method instead of the Mononobe-Okabe Method that uses active and passive earth pressures for earthquake evaluation. US Army Corps of Engineers (COE) earthquake ground motions are the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE).

The OBE is defined as 1/3 of the site Peak Ground Acceleration (PGA) value of 0.236g that yields 0.08g. The MDE is defined as 2/3 of the site Maximum Considered Earthquake (MCE) of 0.277g that yields 0.185g. Vertical earthquake acceleration is zero.

The structure is located on the upstream toe of the structure so full uniform uplift was utilized.

The concrete reinforcement was based on COE temperature guidelines. Mathcad worksheets were used for the design.

Sliding stability at the foundation interface was computed using the Shear Friction Factor (SFF) concept.

The following load cases were considered:

1. Unusual: Construction (dewatered)
2. Extreme: Construction with OBE

3. Usual: Normal Pool (EL. 9895.00 feet NAVD88)
4. Unusual: Infrequent Flood = Q100 (EL. 9897.16 feet NAVD88)
5. Unusual: Coincident Pool = Normal Pool with OBE
6. Extreme: Coincident Pool = Normal Pool with MDE
7. Extreme: Maximum Design Flood = Q10000 (EL. 9900.40 feet NAVD88)

Load cases 5 and 6 were not completed as the structure will not be either pushed into the embankment or pushed into the water column. The results are provided in Table 17**.**

Table 17. Buttress structural stability results

			Sliding		Flotation	
Load Case	**Effective Base In Compression (%)**	**Maximum Net Base Pressure (psf)**	**Shear Friction Factor (SFF)**	**Required SFF**	**Safety Factor**	**Required Safety Factor**
1. Construction	100.0	1,193	1.46	1.3	--	--
2. Construction with OBE	100.0	1,195	1.40	1.1	--	--
3. Normal Pool	100.0	732	2.19	1.7	2.31	1.3
4. Infrequent Flood = Q100	100.0	732	2.19	1.3	2.31	1.2
5. Normal Pool with OBE	--	--	--	--	--	--
6. Normal Pool with MDE	--	--	--	--	--	--
7. Maximum Design Flood = Q10000	100.0	732	2.19	1.1	2.31	1.1

Design computations are included in Appendix J.

Trash Rack Structural Design

A steel trash rack and frame are located on the upstream mass concrete intake structure that includes the slide gate and pipe.

The following references were used in the design:

1. *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2. *Project Review Guide*, Colorado Dam Safety, 21 January 2020.
3. *Steel Construction Manual*, AISC, Fourteenth Edition, 2011.
4. *Specification for Structural Steel Buildings*, AISC, 7 July 2016.
5. *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-10.
6. *Design Standard No. 6: Hydraulic and Mechanical Equipment, Chapter 12 – Trashracks and Trashrack Cleaning Devices*, U.S. Department of the Interior, Bureau of Reclamation, Denver, December 2006.

7. *Steel Design Guide 1: Base Plate and Anchor Rod Design*, 1996, AISC, 2nd, 2006.
8. *Design Guide No. 24: Hollow Structural Section Connections*, AISC, March 2015.

The rack bars and frame were designed for a head differential of 20 feet. The unit weights of steel and water were taken as 490 pcf and 62.4 pcf, respectively. The yield and tensile strengths for ASTM A572 Grade 50 steel plate were taken as 50 ksi and 65 ksi, respectively. The yield and tensile strengths for square ASTM A500 Grade C tubes were taken as 50 ksi and 62 ksi, respectively.

The steel design was based on the Allowable Stress Design (ASD) method. Mathcad worksheets, Staad.Pro computer software, and Hilti's Profis Anchor computer software were used in the design. The frame was analyzed in Staad.Pro and the steel interaction equations for all members were less than the 1.0 required.

Design computations are included in Appendix J.

Gate and Operator

A stainless steel slide gate will be mounted in the concrete intake structure. The gate as specified will comply with AWWA C561, will have self-adjusting seals, compensate debris strikes and the bottom edge shall resist a static load of 1,000 lb per linear foot that is applied 3 in. above gate bottom lip. Gate shall be operable at variable gate open positions in increments of 0.10 ft to regulate flow. Gate shall be able to remain in any closed, partially open, or fully open position for long periods of time without excessive vibrations.

The gate will be operated by a portable hydraulic operator and have a backup hand operated hydraulic operator. Access to the hydraulics will be via a manhole located on the crest of the dam. ¾" stainless steel hydraulic conduits will be contained in an 8-inch diameter pipe, encased in a concrete collar.

Energy Dissipation

The original design plans for Hogchute Dam indicate that a riprap basin was constructed for energy dissipation downstream of the outlet pipe. Our calculations indicate that, with an outlet jet velocity of 34 ft/s and a Froude number of nearly 4, a riprap basin is not sufficient to protect against scour downstream of the outlet pipe when the controlling slide gate is operated fully open.

Using the HY-8 model developed for computing an outlet rating curve, we determined that a USBR Type VI impact basin is an appropriate energy dissipation structure to prevent scour downstream of the pipe outfall. We sized the impact basin based on a 146 cfs discharge from the pipe, which corresponds to that expected at that IDF pool with the upstream slide gate fully open. HY-8 energy dissipation results are provided in Appendix K. These results include the general required dimensions of the structure.

Impact Basin Structural Design

The following references were used in the design:

1. *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2. *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*.
3. EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, USACOE, 30 Nov. 2016.
4. EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACOE, 1 December 2005.
5. EM 1110-2-2200, *Gravity Dam Design*, USACOE, 30 June 1995.
6. EM 1110-2-2502, *Retaining and Flood Walls*, USACOE, 29 September 1989.
7. EM 1110-2-6053, *Earthquake Design and Evaluation of Concrete Hydraulic Structures*, USACOE, 1 May 2007.
8. Letter: Geotechnical Design Parameters, Carson Lake, Mesa County, Colorado, from Michael A. Berry, PE & 15 September 2020 email.
9. Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.
10. HY-8 Energy Dissipation Report output, Ayres, December 2020.

The unit weights of concrete and water were taken as 150 pcf and 62.5 pcf, respectively. The moist and saturated soil unit weights were taken as 125 pcf and 130 pcf, respectively. Uniform surcharge was taken as 200 psf. The soil backfill and foundation sliding friction angles were taken as 32 degrees and 21 degrees, respectively. Soil cohesion was not considered. The net bearing capacities for static and seismic loads were taken as 2,000 psf and 2,666 psf, respectively.

The horizontal force of the water jet during the flood condition, due to the open slide gate, was included on the basin baffle.

The use of at-rest earth pressures required the Seismic Coefficient Method instead of the Mononobe-Okabe Method that uses active and passive earth pressures for earthquake evaluation. US Army Corps of Engineers (COE) earthquake ground motions are the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE).

The OBE is defined as 1/3 of the site Peak Ground Acceleration (PGA) value of 0.236g that yields 0.08g. The MDE is defined as 2/3 of the site Maximum Considered Earthquake (MCE) of 0.277g that yields 0.185g. Vertical earthquake acceleration is zero.

The structure is located on the downstream toe of the structure so full uniform uplift was utilized.

Sliding stability at the foundation interface was computed using the Shear Friction Factor (SFF) concept. The design was based on a concrete compressive strength of 4,000 psi and a reinforcement yield strength of 60 ksi (ASTM A615, Grade 60). The concrete reinforcement was based on temperature requirements and structure forces. The maximum moments and shears were taken from a simplified finite model of the basin using Staad.Pro computer software. Mathcad worksheets were used for the rest of the design.

The following load cases were considered:

1. Unusual: Construction (dewatered)
2. Extreme: Construction with OBE
3. Usual: Normal Pool (EL. 9838.91 feet NAVD88)
4. Unusual: Normal Pool with Surcharge
5. Unusual: Coincident Pool = Normal Pool with OBE
6. Extreme: Coincident Pool = Normal Pool with MDE
7. Extreme: Maximum Design Flood = IDF (EL. 9842.29 feet NAVD88)

The results are included in Table 18. Design computations are included in Appendix J.

Table 18. Energy dissipation Type VI Impact Basin stability results

			Sliding		Flotation	
Load Case	**Effective Base In Compression (%)**	**Maximum Net Base Pressure (psf)**	**Shear Friction Factor (SFF)**	**Required SFF**	**Safety Factor**	**Required Safety Factor**
1. Construction	100.0	700	6.23	1.3	--	--
2. Construction with OBE	100.0	670	5.68	1.1	--	--
3. Normal Pool	100.0	520	3.86	1.7	4.04	1.3
4. Normal Pool with Surcharge	100.0	540	1.95	1.3	4.04	1.3
5. Normal Pool with OBE	100.0	510	1.88	1.3	4.04	1.3
6. Normal Pool with MDE	100.0	580	1.12	1.1	4.04	1.3
7. Maximum Design Flood = IDF	100.0	490	2.96	1.1	2.22	1.1

CIPP Liner

The 30 inch outlet pipe consist of an existing pipe section in the central part of the embankment what that will be remain in place. The existing pipe where the pipe bifurcates into two pipes will be replaced with a new section of the 30 inch pipe and the downstream section will be replace with a 74 linear feet of new steel pipe. The entire pipe from the outlet to the upstream elbow in the intake structure will be lined with a cast-in-place pipe (CIPP) lining.

The specifications for the project will require the installation contractor to have experience lining a minimum of 20,000 linear feet of pipe. Supplier/installer will be responsible for providing liner that must be capable of supporting no less than 58 ft of overburden material with an estimated unit weight of 125 lb per cu ft (pcf). The pipe lining shall also be capable of supporting 200 lb per sq (psf) of surcharge load located above the overburden.

Replacement Steel Pipe

The existing steel pipe, which is wrapped in concrete, will be field cut to remove the upstream and downstream sections as shown on the plans. The concrete collar will be removed a minimum of 12 inches to expose the existing pipe ends. The new pipe will be welded to the existing pipe. The pipe specified is AWWA C200 steel pipe with a minimum wall thickness of 0.375 inches. The pipe interior will be lined with primer and coal-tar enamel (AWWA C203), cement mortar (AWWA C205), or near-white blast cleaned (SSPC SP-10) and painted with one 7-mil coat of coal-tar epoxy. The exterior of the pipe will be coated with two coats of coal tar epoxy with a minimum of 7 mil thickness.

Pipe Encasement

The new sections of 30-inch pipe will be encased in a concrete collar. The existing collar will be sawcut to provide clean joint, dowls will be required to connect the new collar to existing collar. The new collar will have number 5 hoop bars 6 inches on center and number 5 lateral bars, six inches on center. The design of the new collar matches the existing collar design.

Instrumentation and Monitoring Plan

In support of a proposed flood early warning system, the following instruments will be installed:

- Bubbler-type reservoir pool level sensor.
- Pressure transducer-type reservoir pool level sensor (to serve as a backup to the bubbler).
- Tipping bucket rain gauge installed at the top of dam.
- Pressure transducer-type stream level sensor installed in Kannah Creek below the dam and spillway channel confluence.

The instruments listed above will measure data at sub-5-minute resolution and transmit the data in real time via satellite telemetry for monitoring by City staff and other key stakeholders. (Because of Hogchute Dam's remote location, data transmittal via cellular networks, land-line telephone, or radio telemetry is likely not feasible.) For a complete description of the proposed instrumentation and early warning system, see Appendix L, which includes the Draft Early Warning System Feasibility Study prepared for Hogchute Dam and submitted to the DSB for review in December 2020.

An existing system of piezometers is currently installed at the top and near the downstream toe of Hogchute Dam. These piezometers will not be disturbed during construction and will remain operational following. Locations of existing piezometers with notes not to disturb are shown on the Drawings.

The DSB requires that the dam be equipped with a staff gage to facilitate manual water level measurements in the reservoir. Our Drawings show a proposed staff gage installed on the face of the concrete curb that is used to house the 8-inch PVC air vent and hydraulic line carrier pipes. The gage will consist of brass stamped survey washers set every vertical foot from the top of dam down to the invert elevation of the outlet works intake. Between the foot-marker washers, survey nails will be set at 0.1-ft increments.

Hogchute Dam is not currently equipped with station markers. Our Drawings show proposed station markers installed at 100-ft increments at the top of the embankment. Station markers will be installed along the downstream side of the dam crest in a straight line with the same top elevation to permit observation of embankment movement during inspections.

Hogchute Dam currently has benchmarks and surface movement monuments installed both on and off the embankment. These monuments will not be disturbed during construction and will remain after project completion.

Dewatering

Proposed Approach

We propose dewatering to take place in two phases corresponding with different construction activities:

- Phase 1: The reservoir will be drained using its existing outlet works to achieve a construction pool of elevation 9851 ft. A cofferdam with a top elevation of 9860 ft will then be constructed on the upstream side of the embankment to isolate and dewater the outlet works for replacement. To allow ongoing drainage of the reservoir, a 30-inch pipe will pass through the cofferdam and tie into the dam's existing 30-inch outlet pipe. Flow into the 30-inch relief pipe will be controlled using either a gate or a valve. During Phase 1 of dewatering, inflows to the reservoir will be cut off as much as possible using the north bypass channel, which the City estimates has a capacity of approximately 50 cfs. The City will operate gates at the bypass channel diversion structure upstream of the lake to pass as much stream flow as possible around the lake during this phase. Construction activities during this phase of dewatering will include replacement of the existing outlet works intake structure and appurtenances, rehabilitation of the dam's existing 30-inch outlet pipe, embankment work, and construction of the downstream Type VI impact basin.
- Phase 2: The completed outlet works will operate with the new slide gate operating in a fully open position to pass stream flow from Kannah Creek. The City will operate the diversion structure upstream of the reservoir so that flow into the bypass channel is minimized and flow from Kannah Creek passes through the reservoir and dam downstream. This will dewater the overflow spillway so that construction on the concrete weir and downstream spillway channel can take place.

Hydrology

For dewatering hydrology, we simulated p = 0.5 (2-yr), 0.1 (10-yr), and 0.01 (100-yr) annual exceedance probability (AEP) storms using the HEC-HMS model developed for the June 2020 hydrology study. We considered both 2-hr and 48-hr storm durations for our dewatering calculations recognizing that larger volumes associated with longer-duration events may govern and require a larger cofferdam height to protect the work site. Following is a description of how we developed design storm hyetographs for each event simulated:

- For the 100-yr and 10-yr, 2-hr and 48-hr storms, we extracted synthetic storms for Hogchute's contributing watershed using the MetPortal CO-NM Precipitation Frequency GUI v1.2.4. We applied these hyetographs to the HEC-HMS model with no allowance for future climate change since dewatering activities are to occur soon.
- For the 2-yr, 2-hr and 48-hr storms, we applied precipitation ratios to scale down the 10-yr storm depths. (MetPortal storms were not available for 2-yr annual exceedance intervals.) We developed precipitation scaling ratios through trial and error to compute peak inflows consistent with the 2-yr peak determined using USGS regression equations, about 170 cfs. For the 2-yr, 2-hr storm, we determined that the 10-yr, 2-hr hyetograph must be multiplied by a ratio of 0.925; for the 2-yr, 48-hr storm, we determined that the 10-yr, 48-hr hyetograph must be multiplied by a ratio of 0.73. We applied both ratios in HEC-HMS from within the simulation run manager. As a check of our computed 2-yr, 2-hr and 2-yr, 48-hr precipitation depths, we compared them against those same storm depths extracted from NOAA Atlas 14 for Hogchute Dam. We show these comparisons in Table 19. As can be seen, our modeled precipitation depths for both the 2-hr and 48-hr durations

are about 25 to 30 percent higher than those from NOAA Atlas 14, which indicates our modeled precipitation depths used for dewatering calculations may be conservative.

Table 19. Comparison of computed 2-yr storm depths against those extracted from NOAA Atlas 14

Source	2-yr, 2-hr storm depth (in)	2-yr, 48-hr storm depth (in)
HEC-HMS model	0.91	2.61
NOAA Atlas 14	0.70	2.10

Computed Hogchute inflow hydrograph characteristics for all events modeled in support of dewatering design are provided in Table 20. Additional details regarding HEC-HMS model setup are provided in Appendix B. We kept unchanged the initial conditions and model parameters described in Appendix B for all additional model runs required to support dewatering design.

Table 20. Hogchute inflow hydrograph characteristics

Event	Peak inflow (cfs)	Total inflow volume (ac-ft)
2-yr, 2-hr	171	32
2-yr, 48-hr	172	61
10-yr, 2-hr	295	49
10-yr, 48-hr	442	134
100-yr, 2-hr	1,802	314
100-yr, 48-hr	1,007	382

Phase 1 Dewatering Simulations

To model Phase 1 dewatering, we modified the HEC-HMS basin model as follows:

- Configured a diversion upstream of the Carson Lake reservoir element to represent a 50-cfs bypass around the lake. In the model, this diverts 100 percent of the lake's inflow for discharges up to 50 cfs. Only when lake inflows exceed 50 cfs does flow into the reservoir commence.
- Configured a starting reservoir water level surface elevation of 9851 ft, the proposed construction pool elevation.
- Configured a 30-inch outlet pipe at invert elevation 9851 ft to serve as the lake's temporary relieve outlet during construction. For our simulations, we modeled the relief outlet as fully open. Outflows from the 30-inch relief conduit were approximated in HEC-HMS using the rating curve for the proposed outlet works, the development of which is described earlier in this report.

To inform cofferdam sizing, we simulated the six events listed in Table 20 with the Phase 1 dewatering configuration described above. The resulting maximum pool elevations, pool level rises, and outflows from the 30-inch relief conduit are presented in Table 21. With an approximate lake bottom and cofferdam base elevation of 9851 ft, the results indicate that a cofferdam must be at least 7.1 ft to provide 2-yr protection, at least 18.3 ft to provide 10-yr protection, and at least 32.2 ft to provide 100-yr protection. The City has elected to specify that the contractor construct a 10 foot high cofferdam (top elevation 9861 ft), which would likely provide somewhere between 2-yr and 10-yr flood protection for the worksite. The results in Table 21 also show that even 100-yr flood flows will not cause the reservoir pool to rise to a level which would activate the dam's overflow spillway located at elevation 9895 ft.

These dewatering calculations and computed water surface rises require that the bypass channel operate at 50-cfs capacity and that no inflows to the reservoir occur until the bypass reaches this capacity. Furthermore, the reservoir must be kept at its specified construction pool elevation of 9851 ft between rain events, and the 30-inch relief outlet must be kept fully open during periods of increased flow. If the fully open 30-inch relief outlet is not able to keep the construction pool at 9851 ft during periods between storm events, supplemental pumping may be required to achieve this condition.

Table 21. Results of Phase 1 dewatering simulations

Event	Maximum reservoir elevation (ft)	Maximum rise above construction pool (ft)	Maximum reservoir outflow (cfs)
2-yr, 2-hr	9858.0	7	53
2-yr, 48-hr	9858.1	7.1	54
10-yr, 2-hr	9864.5	13.5	78
10-yr, 48-hr	9869.3	18.3	92
100-yr, 2-hr	9883.2	32.2	123
100-yr, 48-hr	9881.8	30.5	121

Phase 2 Dewatering Simulations

During Phase 2 dewatering, the proposed outlet works, downstream embankment, and Type VI impact basin will be complete and operational. Therefore, the upstream cofferdam can be removed and flows from Kannah Creek can be allowed to flow into the reservoir and through the completed outlet. During this phase, the dewatering objective will be to keep the overflow spillway dry so that construction can commence on the concrete weir and downstream channel.

To model Phase 2 dewatering, we modified the HEC-HMS basin model as follows:

- Configured all Kannah Creek flows to discharge into the reservoir, as the basin model was originally configured, without the bypass channel in operation.

- Used the rating curve for the proposed outlet works to model flows through the fully open slide gate.
- Set initial reservoir pool elevations to levels such that the water surface immediately prior to the onset of the incoming flood is approximately 9871 ft, the maximum steady-state pool level expected during normal flow periods with the reservoir operating under its Phase 2 configuration.

For our Phase 2 dewatering simulations, we developed two sets of model simulations—one to represent normal flows and another to represent flood flows. For normal flows, we estimated expected monthly average inflows to the reservoir using both StreamStats and a nearby basin comparison. The nearby basin comparison is based on long-term monthly flow data from the Surface Creek near Cedaredge (USGS 09143000) streamgage, which is located about 14 miles east of Hogchute Dam in a hydrologically similar watershed. Following are properties of the Surface Creek streamgage pertinent to our flow estimation effort:

- Period of record 1938-2020.
- Contributing drainage area 27.4 mi^2 (Hogchute 11 mi^2).
- Average elevation for contributing watershed 9,875 ft (Hogchute 10,607 ft).
- Mean annual precipitation for contributing watershed 38.89 inches (Hogchute 38.17 inches).
- Information from the USGS indicated no significant imports or exports into or out of the watershed above the streamgage.

To estimate monthly flows at Hogchute Dam, we obtained historical average monthly flows for the Surface Creek streamgage and adjusted them according to contributing area (11/27.4) and mean annual precipitation (38.17/38.89). Estimated monthly flow averages obtained using this approach, as well as estimated monthly flows obtained from StreamStates, are shown in Table 22. As can be seen, the two approaches resulted in monthly flow estimates that generally compare well, with the most significant differences being in June and July. Recognizing the uncertainty associated with estimating monthly average flows at Hogchute using either method, we chose the larger value for each month to compute corresponding reservoir pool elevations using the HEC-HMS model configured for Phase 2 dewatering. These corresponding pool elevations are also provided in Table 22. As can be seen, under average-flow conditions during Phase 2 dewatering, corresponding reservoir pool elevations range from about 9,851 ft to 9,871 ft. These lake elevations remain well below the overflow spillway weir crest elevation of 9,895 ft.

We also simulated flood flows for the Phase 2 dewatering configuration to provide some assurance that a moderately sized flood event would not approach the spillway weir crest elevation during construction. For these simulations, we reran all storm events listed in Table 21 using the Phase 2 dewatering configuration in HEC-HMS. As noted previously, we used a starting water surface elevation of 9871 ft for these simulations, which represents the maximum average monthly elevation from Table 22. This conservative approach represents the storms occurring when the antecedent pool elevation is at a plausibly high, steady-state level. We show the results from these simulations in Table 23, which indicate that none of the storm events produce a maximum lake elevation that approaches the spillway weir crest elevation of 9,895 ft.

Table 22. Estimated monthly inflows for Hogchute Dam

Month	Estimated inflow (cfs)		Corresponding reservoir pool elevation (ft)
	From Surface Creek gage comparison	From StreamStats	
Jan	2	3	9851.3
Feb	2	3	9851.3
Mar	3	3	9851.3
Apr	14	7	9852.4
May	48	47	9857.0
Jun	52	96	9870.8
Jul	28	41	9855.8
Aug	20	16	9853.0
Sep	13	9	9852.3
Oct	7	7	9851.7
Nov	3	5	9851.5
Dec	2	4	9851.4

Table 23. Results of Phase 2 dewatering simulations

Event	Maximum reservoir elevation (ft)	Maximum reservoir outflow (cfs)
2-yr, 2-hr	9871.4	98
2-yr, 48-hr	9871.7	98
10-yr, 2-hr	9873.5	103
10-yr, 48-hr	9876.0	109
100-yr, 2-hr	9885.5	126
100-yr, 48-hr	9884.6	125

Supporting calculations for dewatering are provided in Appendix M.

Appendix A
Hydrologic Hazard Evaluation

Carson Lake Dam

Mesa County, CO

Dam ID 420127

Hazard Classification Study

Using the Reclamation Consequence Estimating Methodology

Prepared for:

City of Grand Junction, CO

March, 2020

Carson Lake Dam, Mesa County, CO

Dam ID 420127

Hazard Classification Study Using the Reclamation Consequence Estimating Methodology

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Registered Professional Engineer
State of Colorado No. 50522
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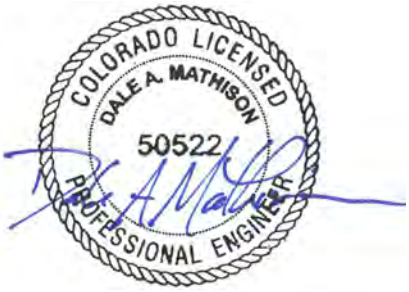

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Contents

Page No.

Executive Summary 1
Background 1
Study Approach 2
Downstream Development 3
Land Use 3
Identification of Residential Structures and Occupancy 3
Dam Breach Development and Outflow Hydrographs 4
Breach Parameters 4
Carson Lake Breach Outflow Hydrographs 5
Two-Dimensional Hydraulic Modeling 6
Model Development and Assumptions 6
Topography 6
Model Geometry Inputs 8
2D Model Boundary Conditions 10
Concurrent Downstream Flows 10
Model Results 11
Estimates of Expected Life Loss 13
Population at Risk 13
Estimates of Flood Severity 13
Estimates of Fatality Rates 13
Expected Loss of Life 14
Recommendations 16
Recommended Hydrologic Hazard Classification 16
Considerations for Developing a Warning System 16

List of Appendices

Appendix A Breach Parameter Estimation
Appendix B Flood Maps (Depth and DV)
Appendix C Fatality Rate Charts from RCEM Interim Guidelines
Appendix D Estimates of Fatality Rate and Loss of Life at Downstream Homes

List of Figures

Page No.

Figure 1: Breach Outflow Hydrographs for Carson Lake Dam 5

Figure 2: US 50 crossing of Kannah Creek, terrain surface generated from a 5x5 ft grid 7

Figure 3: Kannah Creek Road Crossing Kannah Creek, terrain surface generated from a 5x5 ft grid 7

Figure 4: Mesh density and cell size around the US Highway 50 crossing of Kannah Creek 8

Figure 5: Mesh density and cell size, upstream Kannah Creek Road Crossing 9

Figure 6: Flow Hydrographs at Breach and at US Highway 50 for Overtopping Flood Failure 12

Figure 7: Flow Hydrographs at Breach and at US Highway 50 for PMF Failure 12

Figure 8: Flood Stage Hydrographs at Most Upstream House (#14) 17

Figure 9: Carson Lake PMF Inflow, Outflow, and Stage Hydrographs (no failure) 18

List of Tables

Page No.

Table 1: Manning's Roughness (n) Values 10

Table 2: Calculated Peak Flows at Upstream End of Model Reach and at Highway 50 11

Table 3: Expected Loss of Life for Failures of Carson Lake Dam 15

Table 4: Expected Life Loss at Most Severely Impacted Homes for Just-Overtopping Flood Failure 15

Table 5: Expected Life Loss at Most Severely Impacted Homes for Flood Failure during the PMF 16

Executive Summary

This report describes a dam failure analysis and application of the Reclamation Consequence Estimating Methodology (RCEM) for the Carson Lake (Hogchute) Dam. The investigations were undertaken by Ayres Associates on behalf of the City of Grand Junction to help determine the appropriate hydrologic hazard classification for the dam under Colorado regulations. The hydrologic and hydraulic analyses were conducted with the HEC-HMS and HEC-RAS models, Versions 4.3 and 5.0.8 respectively. HEC-HMS was used to develop inflow hydrographs and breach hydrographs for failure during the just-overtopping flood and on the rising limb of the Probable Maximum Flood (PMF). HEC-RAS's two-dimensional dynamic routing routines were used to characterize the progressive depth, velocity, and timing of dam breach flooding along a 20-mile reach of Kannah Creek.

Potentially affected downstream structures were identified based on parcel data and photographs maintained on the Mesa County e-Assessor website. The Population at Risk (PAR) was estimated from a count of residential structures and a single-family residence occupancy rate of 2.4 persons, listed in the 2018 US Census data for Mesa County. The estimated PAR is 79 persons, occupying 33 single-family residences.

Fatality rate estimates were derived from HEC-RAS model results and Figures 3 and 4 of the Bureau of Reclamation's 2015 *RCEM – Reclamation Consequence Estimating Methodology: Interim Guidelines for Estimating Life Loss for Dam Safety Risk Analysis* (RCEM Interim Guidelines). Two warning cases were considered, "little or no warning" and "adequate warning," corresponding to Figures 3 and 4 of the RCEM Interim Guidelines respectively. As an initial estimate of fatalities, maximum DV (depth x velocity) values were extracted at each house location and compared to the upper "suggested limit" curve for homes on permanent foundations, and to the upper "overall limit" curve for mobile homes. Each house's expected fatality rate was then calculated and the total expected fatalities summed over all houses. For the just-overtopping failure case, expected fatalities totaled 6.50 persons with little or no warning, and 0.26 persons with adequate warning. For the PMF failure case, total expected fatalities were 8.88 persons with little or no warning, and 0.33 persons with adequate warning.

These results support a "high hydrologic hazard" classification, if an effective early warning system is implemented. The life loss estimates are low enough for the adequate-warning case, and high enough for the no-warning case, that refinements to the assumptions used to select fatality rates are unlikely to change this conclusion. The adequate-warning fatalities are derived from upper-envelope (conservative) fatality rates and are still much less than one. In contrast, the little-or-no-warning fatalities are so high that an order of magnitude reduction in all the selected fatality rates at a given DV value would barely move the total below one. Therefore, we conclude that with an effective and compliant early hydrologic warning system, triggered by hydrometeorological events that occur before dam overtopping or failure, a "high hydrologic hazard" designation is appropriate.

Background

The City of Grand Junction (City) owns and operates Carson Lake Dam (also called Hogchute Dam) in Mesa County, CO. The dam consists of a 56-foot-high earthen embankment which impounds Kannah Creek and creates a normal storage volume of 637 acre feet in Carson Lake. The dam was originally designed and constructed in 1947. A 2018 Comprehensive Dam Safety Evaluation (CDSE) by the

Colorado Division of Water Resources - Dam Safety Branch (DWR) identified potential failure modes to serve as the basis for future dam rehabilitation design. In January 2019, RJH Consultants prepared a Dam Safety Evaluation Report for the City which included a preliminary estimate of the Probable Maximum Flood (PMF). The PMF was considered to be the appropriate spillway design flood based on the dam's high hazard and presumed extreme hydrologic hazard classification. Later in 2019, Ayres Associates was tasked with 1) finalizing the hydrologic analysis to determine the appropriate spillway design flood, and 2) designing improvements to the dam and outlet works, based on the updated hydrology.

Ayres Associates submitted a draft of the new hydrology report to the City and the DWR for review in September, 2019. The 2019 Colorado-New Mexico Regional Extreme Precipitation Study (REPS) data and software tool were used to develop a two-hour, local storm Probable Maximum Precipitation sequence. The HEC-HMS model was used to estimate a PMF peak inflow of 22,200 cubic feet per second (cfs), which would overtop the dam. The peak computed outflow assuming the current dam configuration and no overtopping failure was 22,100 cfs. The current spillway capacity at the top-of-embankment elevation is 7,400 cfs.

In subsequent consultation with the City and the DWR, the question was raised as to whether the extreme hydrologic hazard designation applies to the Carson Lake Dam. The State Engineer's Office (SEO) 2019 *Rules and Regulations for Dam Safety and Dam Construction* define an extreme hydrologic hazard dam as one whose overtopping failure would create a life loss potential of one or more. To assess whether a shift to a high hydrologic hazard classification would significantly alter the required effort and cost to develop compliant spillway capacity, Ayres Associates applied the Kannah Creek HEC-HMS model to a 0.01 percent exceedance probability (10,000-year) storm. This resulted in a peak computed outflow of 5,035 cfs, less than the existing spillway capacity. This would be the design flow if a high hydrologic hazard classification was assigned instead of an extreme hydrologic hazard classification.

(Note: for comparison, the USGS StreamStats online application was used to estimate flood frequency quantiles at the dam site. The 100-year flood estimated from state regression equations is 368 cfs, and the estimated 500-year peak flow is 427 cfs.)

Initial inspection of downstream development and previously calculated inundation areas suggested that life loss potential was present, but probably very low. The analyses described below were undertaken to provide a systematic and quantitative estimate of life loss potential, using guidance from the RCEM Interim Guidelines.

Study Approach

Ayres' approach to estimating potential life loss from the overtopping failure of Carson Lake Dam included the following tasks:

- Review existing inundation studies to estimate the approximate area of expected flooding due to dam failure.
- Identify and characterize downstream development, including type and average occupancy of each residential structure in the expected inundation area.
- Define the flows, breach conditions, and downstream river and floodplain reach to be analyzed.

- Develop a two-dimensional (2D) HEC-RAS model of the defined study reach at a resolution allowing accurate hydraulic calculations and characterization of flooding parameters (depth, velocity, arrival time, and rate of rise) at any single structure's location
- Estimate breach parameters for the dam's embankment based on SEO guidance, simulate a breach in the dam's embankment during two flooding conditions, and develop an outflow hydrograph for each flooding scenario.
- Using the 2D HEC-RAS model, route the breach hydrographs through the downstream channel and floodplain and generate depth-velocity sequences at each model gridpoint.
- For each identified residential structure, calculate the maximum depth-velocity product at any given point in time by averaging the depth-velocity product at all gridpoints bordering the structure.
- From the envelope curves presented in Figures 3 and 4 of the RCEM Interim Guidelines, develop a consistent procedure for deriving the expected fatality rate at any given structure as a function of maximum depth-velocity product.
- Sum the estimated fatality rates over all residential structures in the inundation zone.
- Review the procedure and assumptions considering other factors such as access to evacuation routes, time of day, possible non-resident use, and type of population; and if appropriate revisit the components of the total fatality estimate.

Downstream Development

Land Use

The first seven miles of Kannah Creek downstream of Carson Lake Dam lie within the Grand Mesa National Forest and are undeveloped and mostly inaccessible by road. After exiting the national forest, the creek flows under a bridge at the east end of Kannah Creek Road, which continues to parallel the creek's south side for the next 6.5 miles before crossing to the north side of the creek and eventually intersecting US Highway 50, approximately 18 stream miles below the dam.

The Mesa County zoning map shows that the land bordering the creek upstream of US 50 is either zoned AFT or AF-35. AFT refers to an Agricultural, Forestry, Transitional District, which is "is primarily intended to accommodate agricultural operations and very low-density single-family residential development." The maximum residential density in AFT districts is one dwelling unit per 5 acres. AF-35 districts have a minimum residential lot size of 35 acres. Downstream (west) of US 50, aerial imagery shows two subdivisions near the creek but at significantly higher elevations. Further downstream the creek drops into a steep and undeveloped gorge as it approaches the Gunnison River.

There are no documented campgrounds or other public recreation facilities near the creek, either in or outside of the National Forest.

Identification of Residential Structures and Occupancy

Preliminary HEC-RAS model runs provided enough flooding information to delineate the approximate downstream area of potential effect due to dam failures. Potentially affected residential structures were then identified by reviewing properties on the Mesa County Assessor's e-map GIS viewer, which includes aerial imagery, house photographs, and descriptions of structure type, construction, and assessed value. A total of 33 residential structures were identified in the potential inundation area. Most are single-family

residences. One small structure is designated a “monastery” but an August, 2019 sale transferred it from a monastic order to private ownership. One is listed as a “guest house” on the same property as a larger residence, and seven of the structures are listed as mobile homes. None are listed as business properties, although it is unknown whether some structures host home-based businesses.

The United States Census Bureau’s 2018 population data for Mesa County lists an average household size of 2.4 persons. For the purpose of calculating the PAR per house, we used 2.4 occupants per house.

Dam Breach Development and Outflow Hydrographs

The dam breach outflow hydrographs were generated with the HEC-HMS model for two flooding scenarios: one in which the Probable Maximum Flood (PMF) occurs and the dam’s embankment fails upon overtopping, on the rising limb of the inflow hydrograph; and one in which a flood just large enough to overtop the dam occurs and the embankment fails at the peak reservoir stage (which occurs on the falling limb of the inflow hydrograph because of the lag between peak inflow and peak outflow.)

Both inflow hydrographs were generated with the HEC-HMS model developed in 2019 to determine the PMF. These analyses are documented in a separate report, *Carson Lake, Dam ID 420127, Improvements to Dam, Spillway, and Outlet Works: Hydrology Update.* The *Hydrology Update* HMS model divides the 11-square-mile watershed into 13 subbasins, including storage and potential failure at three upstream dams and ending with storage routing through Carson Lake. Subbasin loss and unit hydrograph parameters were assigned following guidance in the SEO’s *Hydrologic Basin Response Parameter Estimation Guidelines.* For the 2019 hydrology study, two-hour and six-hour local storm PMP events were developed using the Colorado-New Mexico Regional Extreme Precipitation Study publication and software tool. For this RCEM analysis, the HEC-HMS model was modified to include a failure of the Carson Lake Dam embankment under approximately 0.2 foot of overtopping.

Breach Parameters

Breach parameters were estimated using the Froehlich method as recommended for large dams (more than 50 feet high) under the SEO’s *Guidelines for Dam Breach Analysis.* The following parameters were estimated based on a dam height of 51 feet, top-of-dam storage volume of 750 acre-feet, and overtopping coefficient K_o of 1.3.

Average breach width:	104 feet
Breach bottom elevation:	9851 feet (project datum, approximately MSL)
Breach side slope:	1:1
Time of formation:	0.35 hour

A check of the lateral erosion rate as a function of dam height showed the estimated breach parameters to be within the accepted range per the *Guidelines for Dam Breach Analysis.*

The breach calculations are included as Appendix A. In each flooding scenario, the breach was assumed to begin forming when the embankment crest was overtopped by 0.25 foot.

Carson Lake Breach Outflow Hydrographs

The breach hydrographs, which served as the inflow boundary condition to the two-dimensional HEC-RAS model described in the following section, were developed in the HEC-HMS model. Hydrographs for failure during a just-overtopping flow and failure on the rising limb of the PMF are shown in Figure 1, below.

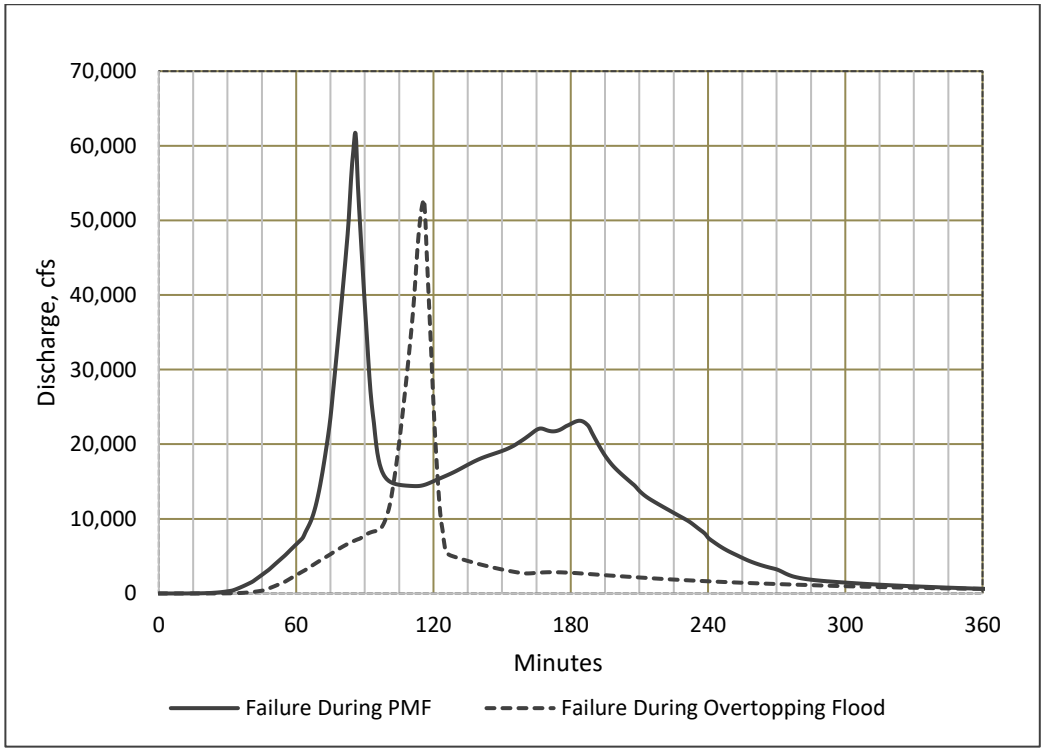


Figure 1: Breach Outflow Hydrographs for Carson Lake Dam

The peak computed outflow for failure during a flood which peaks just above the embankment crest was 52,700 cfs. The peak computed outflow for failure at 0.25 foot of overtopping during the PMF inflow was 61,600 cfs. Although both breach scenarios had the same dimensions and the same triggering pool level, the peak outflow for the PMF case significantly exceeded that for the just-overtopping flood because the PMF breach accompanied an inflow hydrograph that was still rising at the time of failure, whereas the just-overtopping inflow hydrograph had already begun to recede when the assumed failure occurred.

Two-Dimensional Hydraulic Modeling

Downstream flow conditions were dynamically modeled using the two-dimensional (2D) hydraulic routines of the U.S. Army Corps of Engineers HEC-RAS model, Version 5.0.8. A 2D model accounts for local variations in depth and velocity that are difficult to distinguish using the results of a one-dimensional model.

Model Development and Assumptions

To estimate the approximate model domain needed, Ayres first reviewed a 1-D HEC-RAS model provided by the City of Grand Junction which was previously used to develop Emergency Action Plan inundation maps. The 2D model developed for this study covered Kannah Creek from a point just downstream of Carson Lake Dam to a point 0.25 mile upstream of the confluence with the Gunnison River, a distance of approximately 20 miles. The modeling intention was to adequately capture the overall extent of the flood pulse with enough detail to accurately assess effects on structures.

Topography

The City of Grand Junction's GIS department supplied Lidar data tiles covering the Kannah Creek area from above Carson Lake Dam to the junction with the Gunnison River. The LiDAR data for Mesa County were collected by Wolpert Inc. under contract to the U.S. Geological Survey in 2015 and 2016. The Lidar horizontal datum was NAD83 (HARN) UTM Zone 12N US feet. The vertical datum was the North American Vertical Datum of 1988 (NAVD88). The collected data and modeled data maintained the datums of the supplied Lidar.

Point density for the ground only points was approximately 2.8 points per square ft. A surface was generated for the entire Kannah Creek mainstem drainage using the Global Mapper version 20.1 software. A 5-foot by 5-foot grid of this surface was exported for use in the hydraulic model. The size of the modeled area, 13.7 square miles, necessitated reduction of the point density found in the Lidar data.

The grid surface density of 5 feet by 5 feet was considered more than adequate for the dam break analysis. Roadway and riverine features as well floodplain features were clearly captured and displayed by the generated grid surface. Figures 2 and 3 show typical surface topography captured in the terrain model.

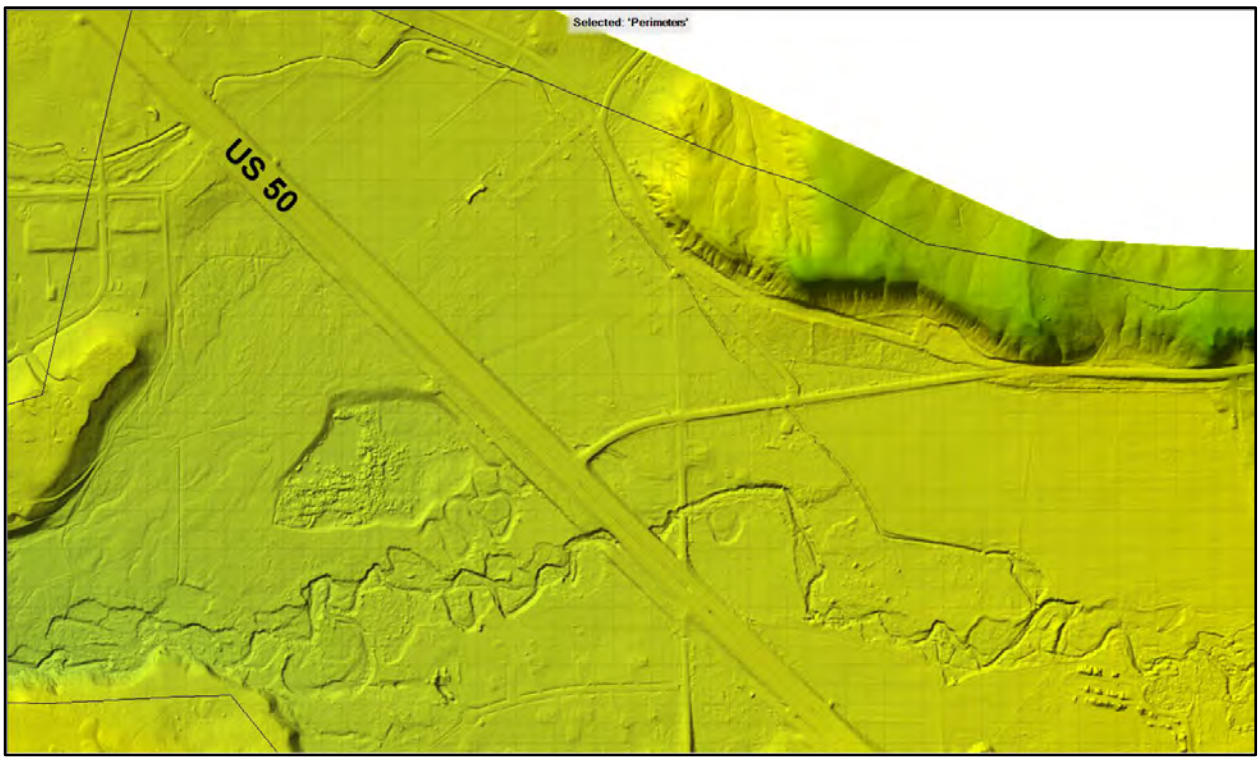


Figure 2: US 50 crossing of Kannah Creek, terrain surface generated from a 5x5 ft grid

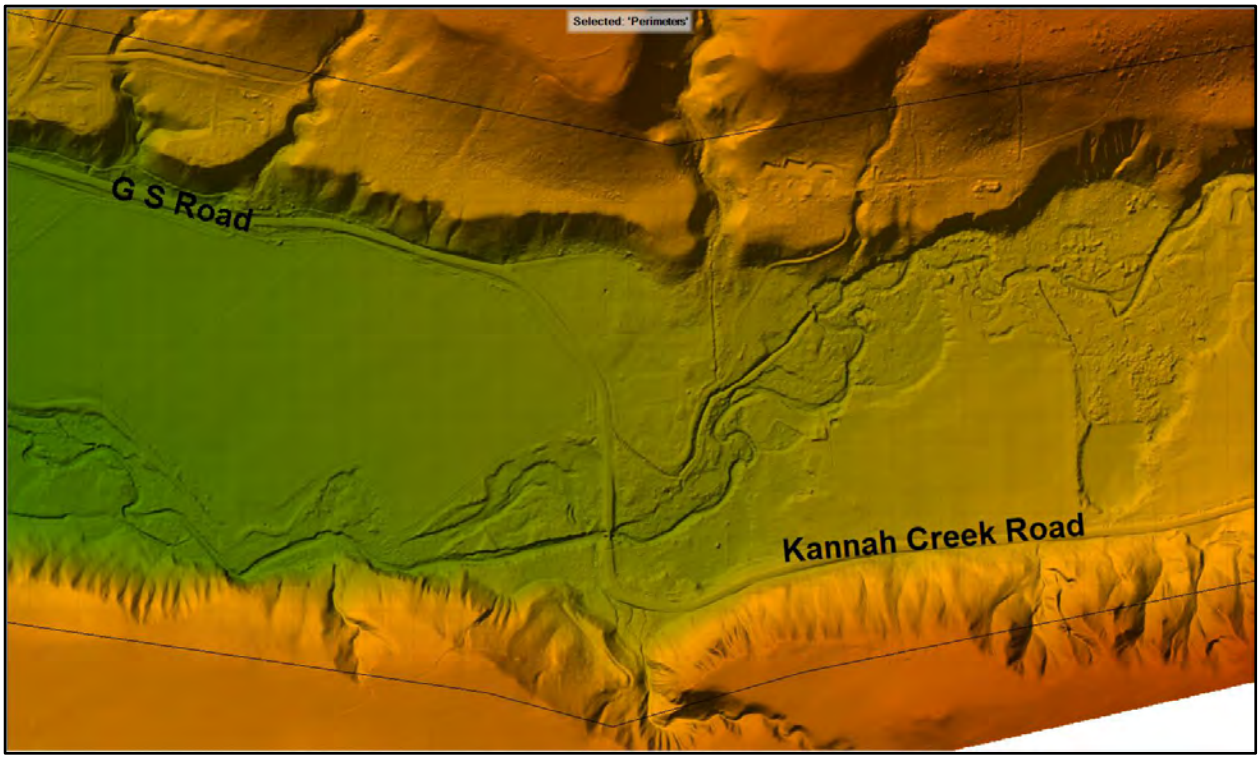


Figure 3: Kannah Creek Road Crossing Kannah Creek, terrain surface generated from a 5x5 ft grid

Model Geometry Inputs

HEC-RAS 2D allows the user to develop a mesh consisting of structured and unstructured sections of mesh. For this effort the structured exterior elements were 100 feet by 100 feet. Throughout the model domain breaklines with reduced spacing were added to the mesh, capturing the hydraulic controls of the terrain. For example, the main channel and the US Highway 50 roadway have 50-foot spacing and unstructured mesh. Overall the mesh domain consists of 27,935 cells. Figures 4 and 5 show typical sections of the mesh and the element layout.

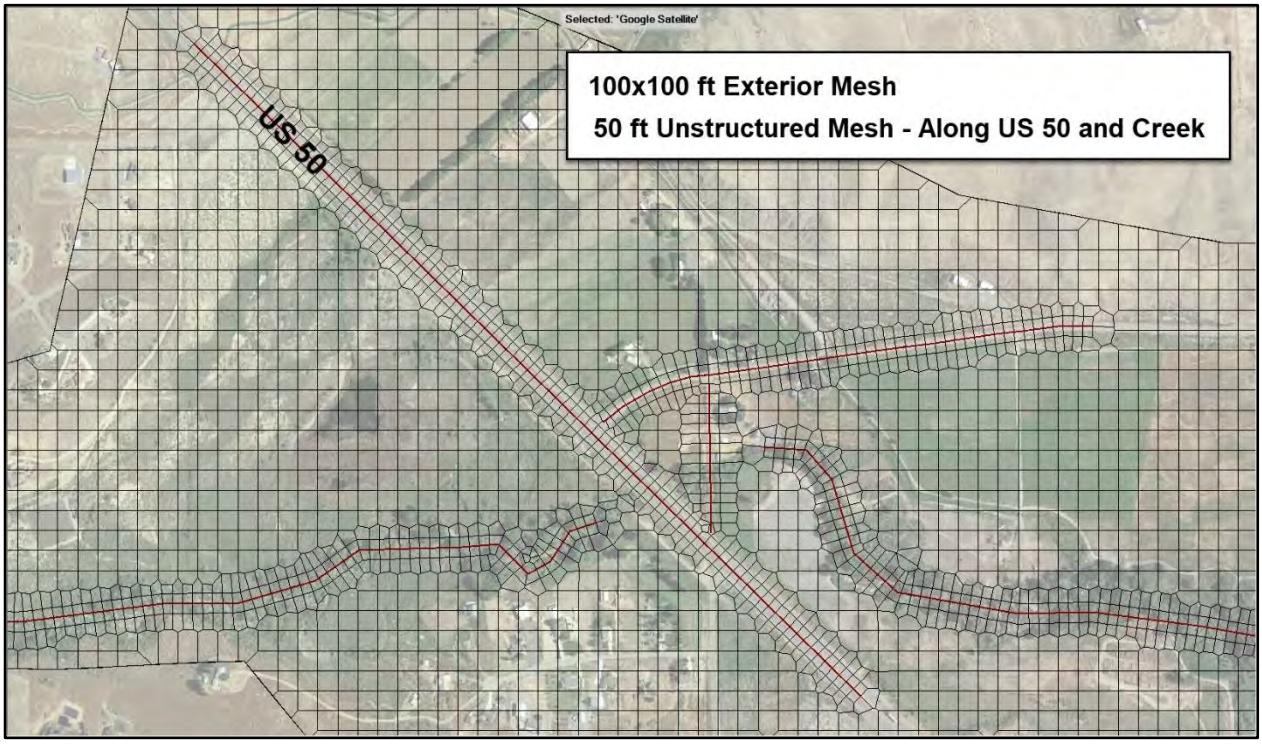


Figure 4: Mesh density and cell size around the US Highway 50 crossing of Kannah Creek

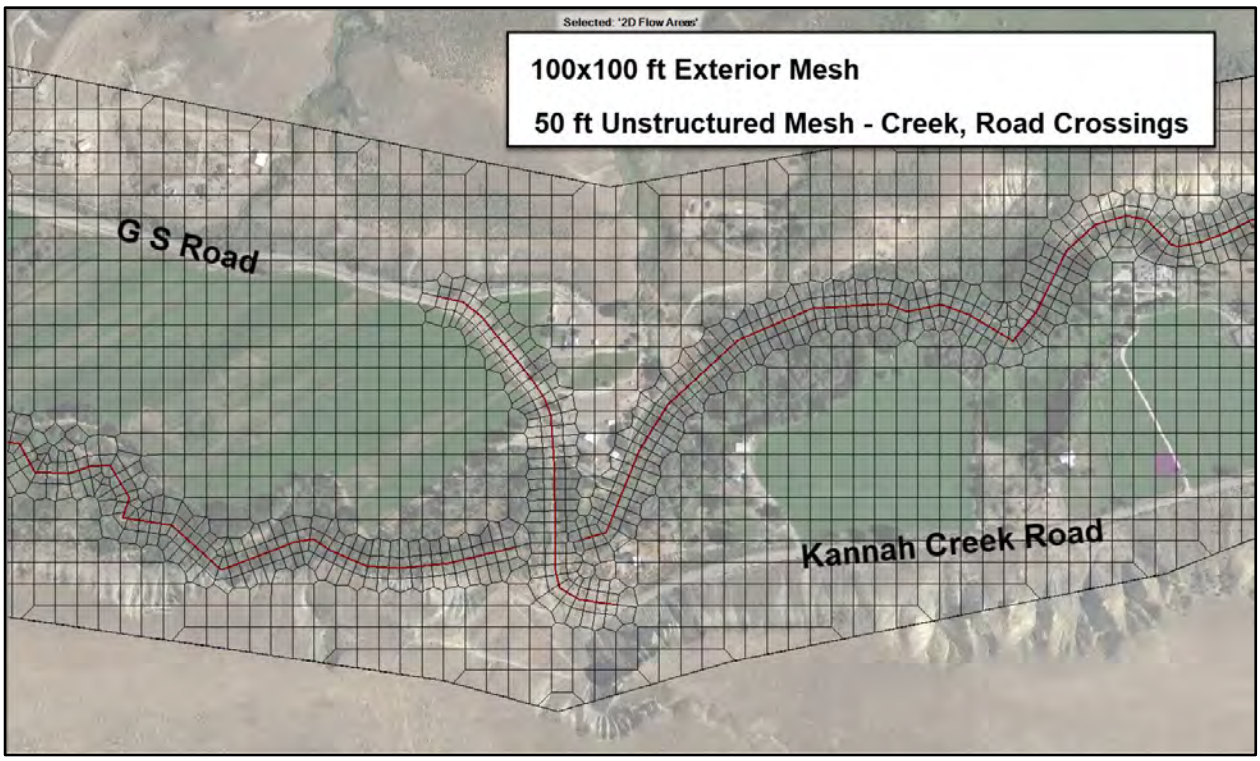


Figure 5: Mesh density and cell size, upstream Kannah Creek Road Crossing

Bridges in the model include the Kannah Creek Road crossing about 7 miles downstream of the dam, a second crossing of Kannah Creek Road approximately six miles further downstream, and the U.S. Highway 50 crossing approximately 16 miles downstream of the dam. Given the magnitude of the flows being modeled, debris generation, transportation, and capture were assumed to be very high. Consequently, all bridge openings in the model extents, including the downstream US Highway 50 bridge, were assumed blocked.

A materials coverage was developed from aerial imagery and used to assign Manning's roughness values. Materials coverage and the associated Manning roughness values were developed using aerial imagery. Simplification of the roughness coverage over the 13.7 square miles of mesh included grouping the channel and immediate riparian vegetation into one region and ignoring changes in roughness smaller than the mesh elements. The Manning n roughness values used for this project are presented in Table 1.

Material Type	Manning n Value	Description
Roadway	0.025	Driving surface of roadway and immediate verge
Cropland	0.04	Overbank agricultural lands, fields, pastures and other short grass lands.
Upland Sparse Vegetation	0.045	Overbank light density scattered scrub, usually sagebrush, over very light cover to dirt.
Upland Medium Vegetation	0.05	Medium scattered pinyon-juniper scrub over light cover to dirt.
Medium Vegetation	0.06	Medium density scattered scrub over light cover.
Canyon Edge	0.07	Scrub, sagebrush, and rocky edges of downstream canyon/creek
Floodplain Vegetation	0.07	Valley bottom vegetation
Structures with Vegetation	0.08	Buildings plus outbuildings and planted trees
Channel and Riparian	0.09	Channel and thickest riparian vegetation; tree stands with understory of shrub and taller grass.
Upper Valley	0.09	Denser treed and scrub section in steep rocky upper reach, pines, oaks, pinyon-juniper scrub

Table 1: Manning's Roughness (n) Values

2D Model Boundary Conditions

The upstream model boundary was an inflow hydrograph derived from the HEC-HMS dam breach modeling described above. Three different upstream boundary hydrographs were modeled: the just-overtopping, no-failure case (to help describe the "baseline" condition just before dam failure), the just-overtopping breach hydrograph, and the PMF hydrograph with a breach occurring on the rising limb just after first overtopping the dam. The upstream boundary hydrographs are shown above in Figure 1.

The downstream boundary conditions for each flow were based on a 1D rating curve for a valley-wide cross section and valley slope at the downstream end of the 2D model coverage. The cross section station and elevation data and downstream slope was generated from the 2D 5x5 ft terrain surface. The rating curve was developed within Federal Highways Administration's Hydraulic Toolbox Version 4.4. Possible local errors from using the 1D rating curve (from applying the single water surface elevation per flow over many boundary cells) were not seen to propagate upstream in the 2D model. The location of the 2D downstream boundary is also approximately 0.5 stream mile downstream from the closest structure of concern.

Concurrent Downstream Flows

The Kannah Creek drainage area increases along the modeled reach from 11 square miles at the dam to 101 square miles at US Highway 50. However, tributary inflows to Kannah Creek downstream of Carson Lake were considered insignificant relative to the magnitude of the post-failure floods and were not accounted for in the model. The initial pre-failure dam outflow in the assumed scenarios is over 8,000 cfs. Depending on the storm centering and orientation, the downstream drainages might show a significant

response to the same storm event. However, the watershed is elongated (roughly 25 miles long by an average of four to five miles wide) and so a local storm centered above Carson Lake would produce only residual rainfall over the lower watershed, and the lower tributary peaks would be offset in time relative to the outflow hydrograph from the dam.

A flood frequency analysis was run in the StreamStats application for the 101-square-mile watershed at US Highway 50. The estimated 100-year flood at this location is 1,530 cfs, less than 20 percent of the pre-breach Carson Lake outflow and less than 10 percent and 8 percent of the post-breach peak flows for the just-overtopping and PMF failure cases respectively. Considering the effect of neglecting these flows in the life loss estimation, we concluded that the possible changes in the depth-velocity product (DV) would be negligible in comparison to the recommended range of possible fatality rates at a single DV value. Furthermore, severe advance rain and flash floods from tributaries would likely result in more pre-evacuation and heightened awareness, possibly reducing fatality rates to offset any positive change in the DV value.

Model Results

Each breach outflow hydrograph was routed through the model domain, producing timeseries of depth and velocity at each model gridpoint. Maps showing the maximum calculated depth and DV values for the just-overtopping failure case and the PMF failure case are included as Appendix B. Wave front arrival times (relative to the time of breach initiation) and house locations are also shown on the Appendix B maps. For comparison, Appendix B also shows the inundation area for the overtopping flood without failure.

The computed downstream attenuation in the peak flows is substantial. Table 2 compares peak flows just below the dam and at the upstream side of US Highway 50, approximately 16 miles downstream of the dam. The flow hydrograph at Highway 50 was computed as the flux across the valley section just upstream of the crossing. For the case of overtopping failure on the rising limb of the PMF hydrograph, the breach outflow is smaller than the later-arriving PMF peak flow. Figures 6 and 7 depict the Table 2 information in hydrograph form.

Failure Scenario	**Peak Breach Outflow at Dam (cfs)**	**Peak Flow at US Highway 50 (cfs)**	**Comments**
Just-Overtopping	52,700	14,200	Natural and breach flow peaks are coincident
Overtopping During PMF	61,600	18,000	Larger (22,000 cfs) PMF peak arrives later

Table 2: Calculated Peak Flows at Upstream End of Model Reach and at Highway 50

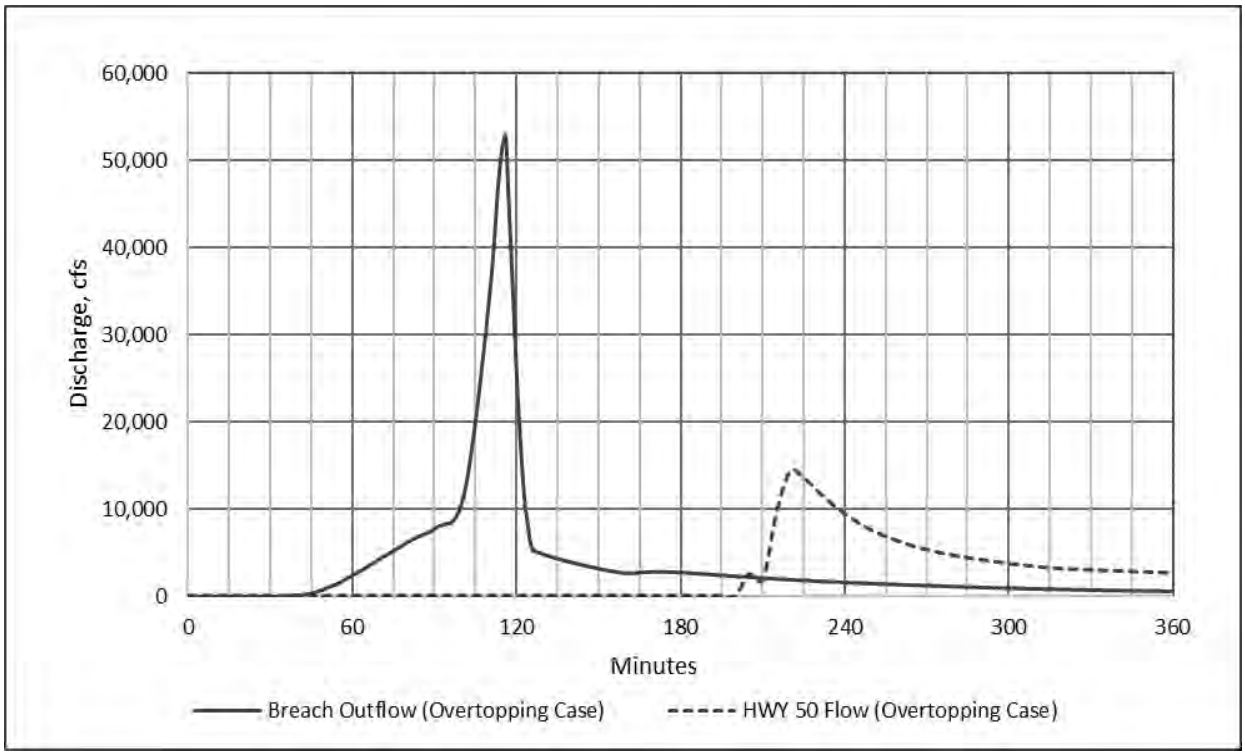


Figure 6: Flow Hydrographs at Breach and at US Highway 50 for Overtopping Flood Failure

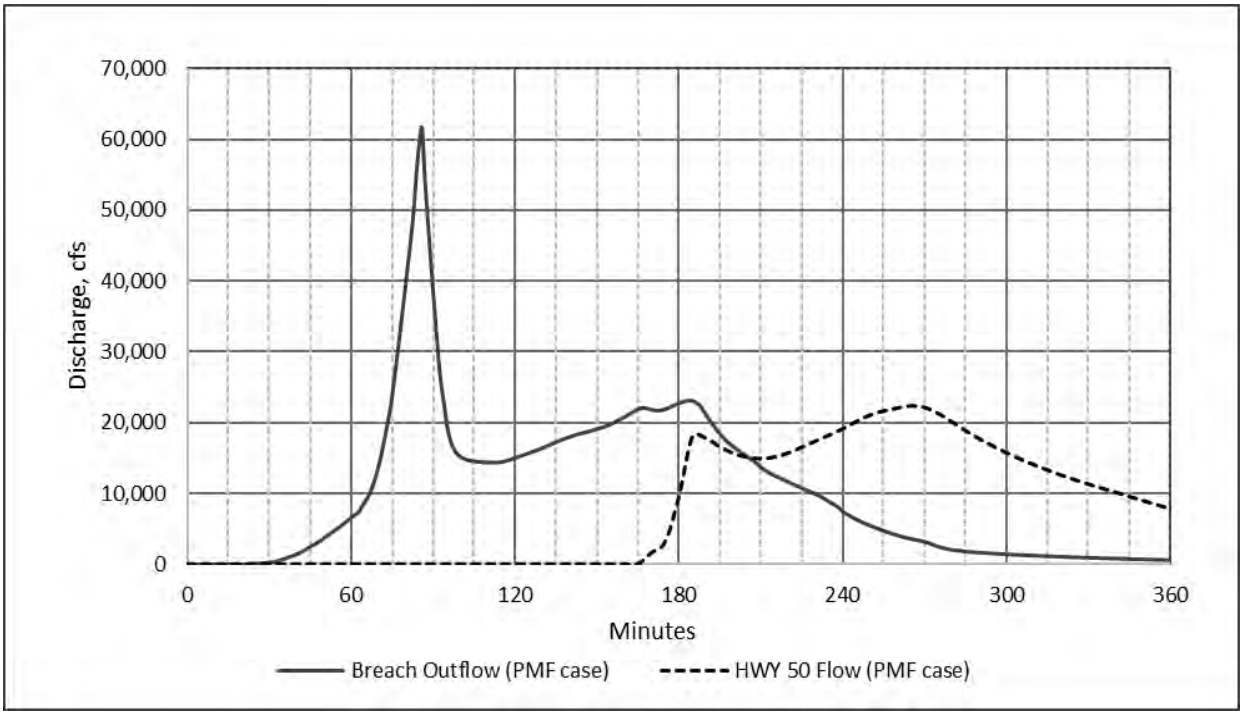


Figure 7: Flow Hydrographs at Breach and at US Highway 50 for PMF Failure

Estimates of Expected Life Loss

The methodology for estimating life loss as a result of a failure of Carson Lake Dam is based on the Bureau of Reclamation's 2015 publication *RCEM – Reclamation Consequence Estimating Methodology: Interim Guidelines for Estimating Life Loss for Dam Safety Risk Analysis* (RCEM Interim Guidelines). An update to the procedure outlined in Reclamation's Dam Safety Report No. DSO-00-06, this methodology became the consequence estimating procedure for all dam safety studies by the Bureau starting in 2015. The procedure outlined in this report uses worldwide case histories to generate empirically-based graphical representations of fatality rate as a function of flood severity and warning time. The report also provides guidelines for consideration of potential variables and uncertainties for evaluating life loss estimates. The guidance is both quantitative (envelope curves relating fatality rate to flood hydraulics) and qualitative (additional site-specific and population-specific considerations).

Population at Risk

The estimation of the population at risk (PAR) was based on the review of residential structures detailed in an earlier section of this report. Assuming that each "single family" residential structure housed an average of 2.4 persons (based on the 2018 US Census data for Mesa County), the total estimated PAR is 79 persons in 33 households. This assumes that all residents are at home at the time of the event (or that the number of visitors is roughly equal to the number of persons away from home). Since thunderstorms often occur in late afternoon or early evening, an early evening flood when most residents are at home is not an unrealistic scenario.

Estimates of Flood Severity

The metric identified by the RCEM Interim Guidelines for flood severity is the product of depth and velocity (DV). Per the recommendations of the Guidelines, simulations of downstream flooding resulting from failures of Carson Lake Dam were performed using a two-dimensional (2D) hydraulic model as described above. Results of the model were used to generate a grid dataset of maximum DV values. This dataset was imported to GIS to determine the maximum DV at each house identified in the flood inundation area. In most cases, a structure's footprint contacted multiple grid points, in which case the average maximum DV in all adjacent grid points was used.

Estimates of Fatality Rates

Figures 3 and 4 of the RCEM provide graphical representations of the empirical relationship between DV and fatality rate for "Little or No Warning" and "Adequate Warning" cases respectively. Each figure includes many empirical data points and several curves to represent the case-by-case variability of fatality rates. Figures 3 and 4 are reproduced as Appendix C.

In addition to the DV product, fatality rates depend on additional variables such as:

- Public understanding of flood danger
- Effectiveness of evacuation efforts
- Ability of structures to withstand flood forces (well-built houses versus temporary shelters or trailer homes)

- Rate of rise for flood flows
- Dam failure mode
- Potential of flood to carry debris
- Potential for damage to infrastructure that would disrupt warning systems and/or evacuation routes

As an initial approach to estimating the total expected life loss, the upper envelope curves denoted "suggested limit" on Figures 3 and 4 of the RCEM Interim Guidelines were assumed to apply to all structures except mobile homes. This is a somewhat conservative assumption. The following considerations support conservative fatality rate estimates:

- Many of the houses are single-story, older homes.
- Many are reached by long gravel drives which could also be flooded.
- Houses are set far apart and screened by brush, trees, and outbuildings, making communication and mutual assistance between neighbors less likely.

Supporting a less conservative fatality rate were the following considerations:

- At no location does the predicted rate of water level rise exceed 10 feet in five minutes (a criterion listed in the RCEM guidance).
- The public roads accessing the homes would not be flooded (although flooding or destruction of bridge crossings would isolate one stretch of Kannah Creek Road).
- The overtopping failures modeled would occur in conjunction with an unprecedentedly large rainstorm and natural flood, of which all residents would be aware.

The three structures noted on the assessor's website as mobile homes were referenced to the "overall limit" curve, a conservative interpretation based on the fact that mobile homes can be easily dislodged by moving water and offer relatively little shelter.

Expected Loss of Life

The total expected downstream life loss was calculated by estimating the statistical fatality rate for each house as described above, multiplying by the occupancy rate of 2.4 persons/house, and summing over all of the potentially affected houses. (Note that the RCEM Interim Guidelines suggest establishing DV zones comprising multiple residences. However, a house-by-house calculation was used in this situation given the small inundation area, limited number of residences, and significant local variability in DV.)

Table 3 lists the calculated expected life loss for the two dam failure scenarios, with and without adequate warning. A spreadsheet showing the calculation of DV, fatality rate, and expected life loss is attached as Appendix D.

Carson Lake Dam Failure Scenario	Expected Number of Lives Lost	
	Little or No Warning	Adequate Warning
Failure at peak of just-overtopping flood, 0.25 foot over embankment crest	6.50	0.26
Failure on rising limb of PMF hydrograph, 0.25 foot over embankment crest	8.88	0.33

Table 3: Expected Loss of Life for Failures of Carson Lake Dam

Tables 4 and 5 list the eight residences with the greatest estimated life loss for each flood and warning scenario. In each case, the top eight residences account for 80 to 90 percent of the total expected life loss. The listed residences are ordered by the no-warning expected life loss value.

House #	House Type	Distance Downstream from Dam (miles)	Maximum DV (ft^2/sec)	Expected Life Loss	
				No Warning	Adequate Warning
10	Conventional Construction	8.7	129	1.25	0.018
1	Mobile Home	7.6	69	1.09	0.064
14	Conventional Construction	7.3	111	1.05	0.016
13	Conventional Construction	7.8	85	0.68	0.012
12	Mobile Home	7.6	54	0.60	0.041
19	Mobile Home	13.4	52	0.55	0.039
2	Conventional Construction	10.5	72	0.47	0.010
5	Conventional Construction	12.7	53	0.21	0.007

Table 4: Expected Life Loss at Most Severely Impacted Homes for Just-Overtopping Flood Failure

House #	House Type	Distance Downstream from Dam (miles)	Maximum DV (ft^2/sec)	Expected Life Loss	
				No Warning	Adequate Warning
1	Mobile Home	7.6	78	1.39	0.079
10	Conventional Construction	8.7	142	1.35	0.020
14	Conventional Construction	7.3	125	1.21	0.018
12	Mobile Home	7.6	67	1.01	0.060
2	Conventional Construction	10.5	96	0.85	0.014
13	Conventional Construction	7.8	96	0.85	0.014
19	Mobile Home	13.4	58	0.73	0.047
11	Conventional Construction	8.2	78	0.56	0.011

Table 5: Expected Life Loss at Most Severely Impacted Homes for Flood Failure during the PMF

Recommendations

Recommended Hydrologic Hazard Classification

Based on the analysis described above, it is Ayres' opinion that the Carson Lake Dam would appropriately be classified as a high hydrologic hazard structure if an effective downstream warning system is implemented and maintained. This is based on a conservative application of the RCEM fatality rate curves, particularly for the adequate-warning condition. Although the published curves leave room to adjust for case-by-case conditions, the life loss estimates shown in Table 3 are much less than one for the adequate-warning case, and much greater than one for the no-warning case. We would not expect modest changes to the fatality rate, made on a structure-by-structure basis, to change this conclusion.

Considerations for Developing a Warning System

The warning times shown in Appendix D represent the time between breach initiation and the first wetting of the ground at the house in question. A flood warning system designed to send alerts when either the spillway discharge or the reservoir level exceeded a certain setpoint would add additional warning time.

Figure 8 presents stage hydrographs to illustrate the progression of flooding at the most upstream house in the modeled reach. This house (#14 on the Appendix B maps) is among the most severely impacted structures, with a maximum DV value of 125 and an expected life loss of 1.2 persons for the PMF failure case. Note that the depth-velocity product uses a depth referenced to the ground elevation at the house, which in this case is several feet below the (elevated) first-floor elevation of the house. While this may produce conservative results, we understand the RCEM fatality curves were developed with reference to ground elevations – not first floor elevations – and should be used in this context.

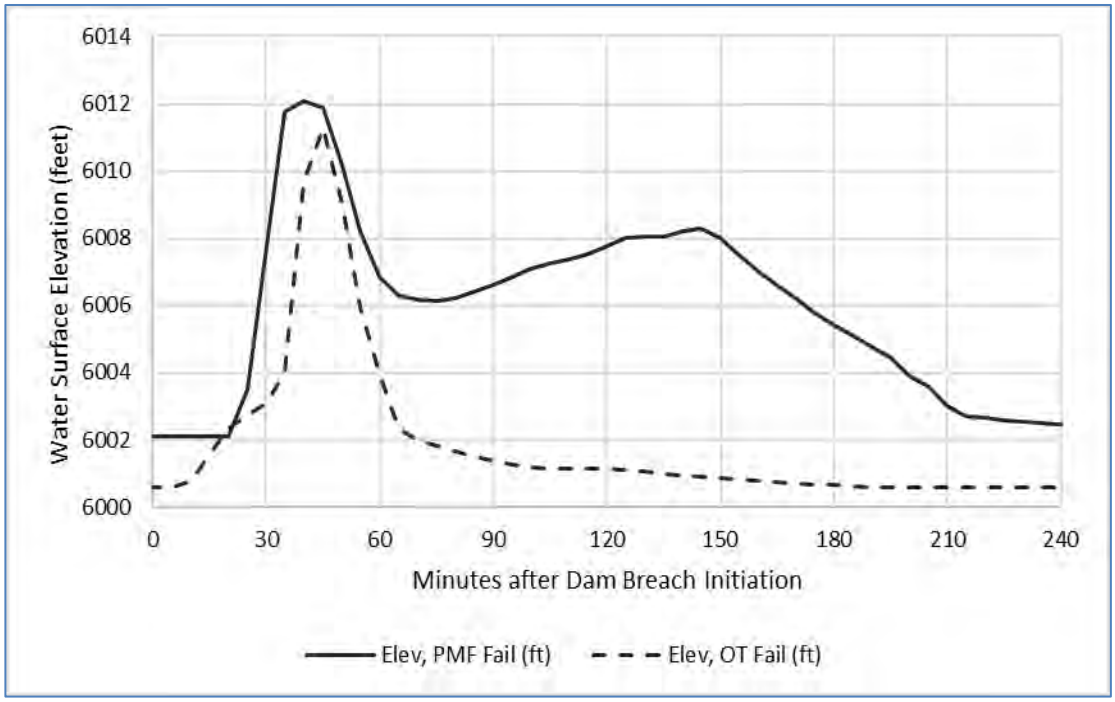


Figure 8: Flood Stage Hydrographs at Most Upstream House (#14)

Figure 9 shows inflow, outflow, and stage hydrographs for the (no-failure) PMF at Carson Lake Reservoir. The spillway crest elevation and maximum normal elevation is 9,895 feet. The total reservoir rise time from normal pool to overtopping is approximately 50 minutes.

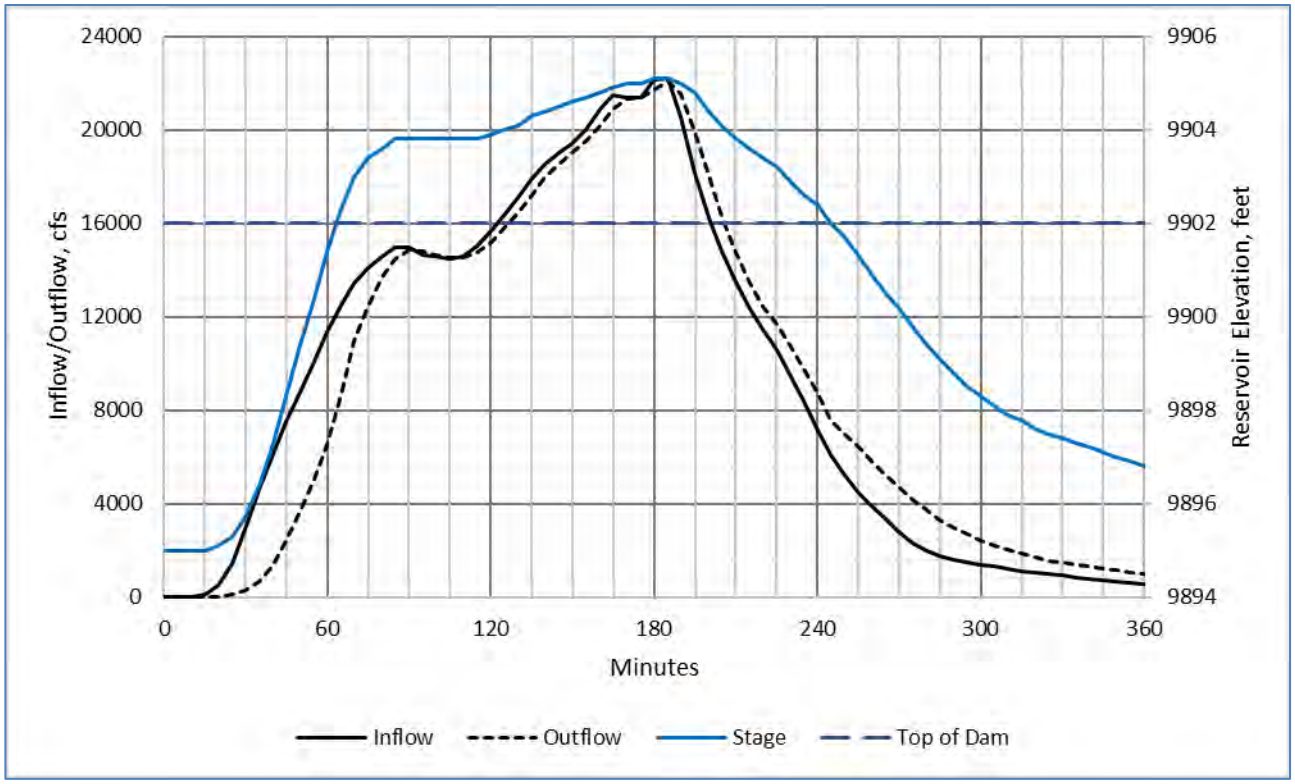


Figure 9: Carson Lake PMF Inflow, Outflow, and Stage Hydrographs (no failure)

A flood warning system would likely be based on redundant measures of reservoir stage and outflow, supplemented by real-time meteorological observations. An optimal triggering flood stage would be rare enough that numerous "false alarms" do not occur, but low enough to provide significant additional warning time relative to the overtopping stage. For example, based on the timing shown in Figure 9, an alarm set to respond when the pool reached elevation 9,897 would:

- Be activated approximately 25 minutes before overtopping for the PMF case, which is a worst-case scenario in terms of rate of rise;
- Produce a total warning time of approximately 45 minutes before failure-induced flooding reached the first house;
- Be activated at an outflow of approximately 1,400 cfs, which is more than three times the 500-year inflow to Carson Lake as estimated in the StreamStats application; and
- Have the added benefit of producing a warning for natural flash flooding with no dam failure, which is a much more likely occurrence than a flood which continues to rise over the crest of the dam.

Further investigation of the optimal detection and warning procedures would be developed as part of a warning system feasibility study.

The above discussion also assumed that a minimum of 45 minutes constitutes "adequate" warning in this setting. We consider this a reasonable assumption based on the small number of affected structures, the proximity of all affected structures to (unflooded) public roads, and the capability of the City of Grand

Junction to develop an effective homeowner education program. This assumption would also be investigated in more detail in a feasibility study.

Appendix A

Breach Parameter Estimation

AYRES ASSOCIATES

① Carson Dam

Height = 56 ft (> 50 ft → Large Dam)

Storage = 637 ac-ft (Between 100 & 4000 ac-ft → SMALL DAM)

CONSIDER LARGE DAM

② Storage Intensity (SI) = $\frac{V_w}{H_w}$

when full to top of dam (9902 ft)

V_w = 750 ac-ft

H_w = ~~56~~ 51 ft (elev-stor curve only allows 51 ft of water)

$SI = \frac{V_w}{H_w} = \frac{750}{\cancel{56}\ 51} = \cancel{13.3}\ 15$ (medium according to Table 3)

Froelich method OK

③ Froehlich Parameters

$B_{avg} = 8.239\, K_o V_w^{0.32} H_b^{0.04}$

K_o = 1.3 for overtopping

H_B = ~~56 ft~~ = 9902 - 9851 = 51 ft

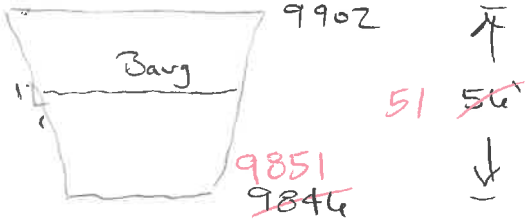
V_w = 750 ac-ft

$B_{avg} = 8.239\,(1.3)(750)^{0.32}(\cancel{56}\ 51)^{0.04} = \cancel{105}\ 104$ ft

Project No.	Remarks	Prepared by ajs	Date 1/16/2020
Project Name Carson		Checked by EBF	Date 1/17/20
Title/Item Dam Breach			Sheet of

AYRES ASSOCIATES

④ Side slope 1:1 for overtopping



Bottom width = $B_{ave} - \left(\frac{56}{2}\right) \times 2$ = 105 ft − 56 ft

= 49 ft

53 ft

⑤ Breach Development Time

$T_f = 3.66t \sqrt{\frac{750}{(32.2)(56)^2}}$ = 0.32 hr

⑥ Results Check

$\frac{B_{avg}}{H_B} = \frac{105\text{ ft}}{56\text{ ft}} = 1.9 > 0.6$, so full breach devel. expected

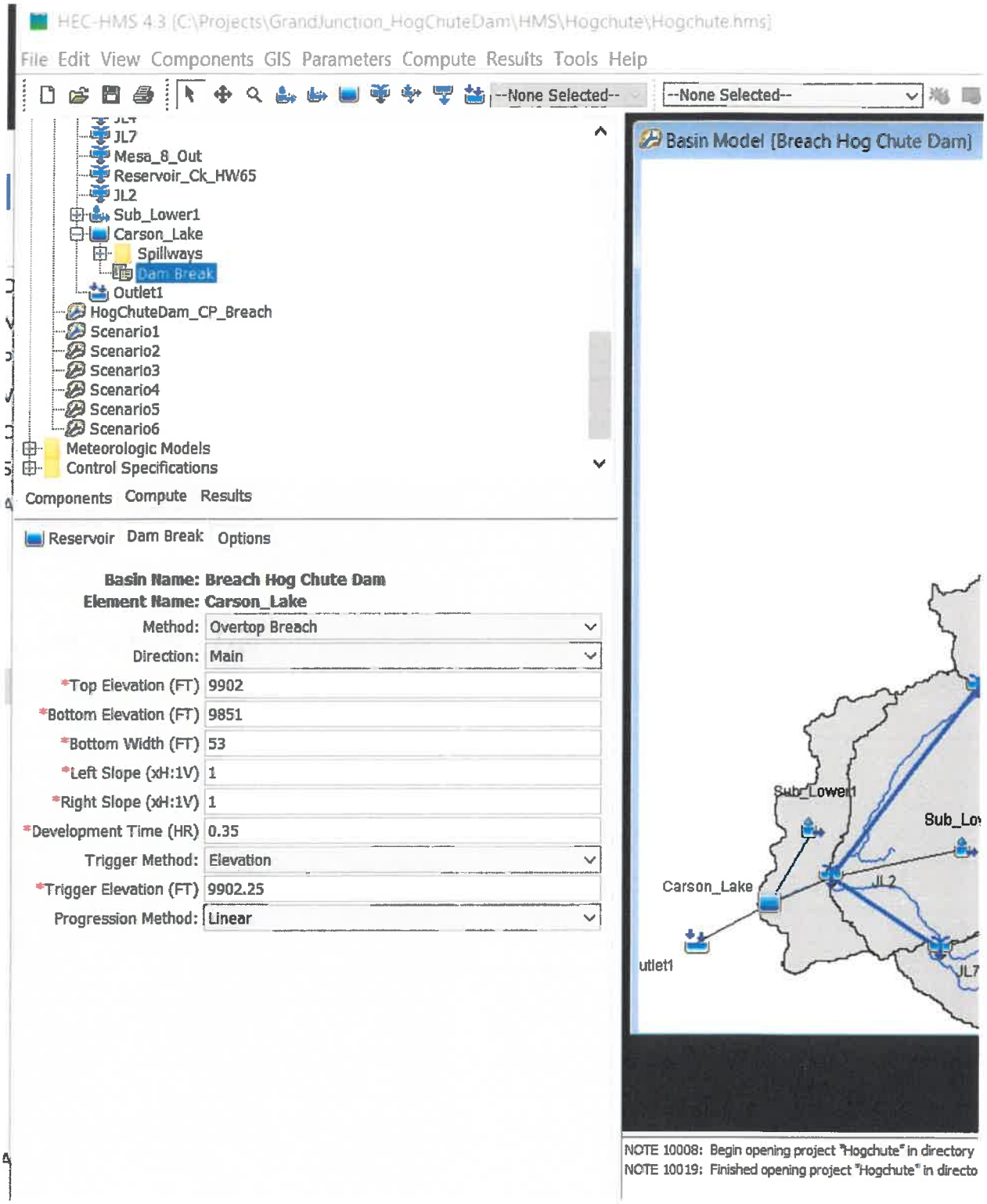
Erosion Rate = ER = $\frac{B_{avg}}{T_f} = \frac{105\text{ ft}}{0.32\text{ hr}} = 328\ \frac{\text{ft}}{\text{hr}}$

$\frac{ER}{H_W} = \frac{328\ \text{ft/hr}}{56\text{ ft}} = 5.9$, since between 1.6 and 21, erosion rate is reasonable

10,000 yr precip × 1.35 = peak pool elev of 9902.26'

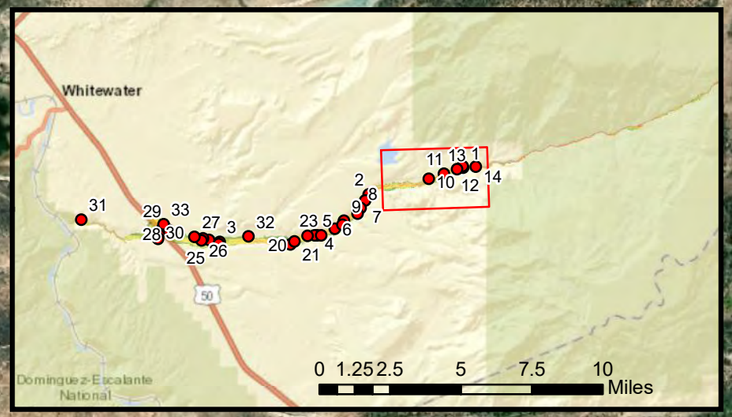
(i.e. 0.26 ft overtopping)

Project No.	Remarks	Prepared by ajp	Date 1/16/2020
Project Name Carson		Checked by EBF	Date 1/17/2020
Title/Item Dam Breach			Sheet of


HEC-HMS 4.3 [C:\Projects\GrandJunction_HogChuteDam\HMS\Hogchute\Hogchute.hms]
File Edit View Components GIS Parameters Compute Results Tools Help
--None Selected--
--None Selected--
JL7
Mesa_8_Out
Reservoir_Ck_HW65
JL2
Sub_Lower1
Carson_Lake
Spillways
Dam Break
Outlet1
HogChuteDam_CP_Breach
Scenario1
Scenario2
Scenario3
Scenario4
Scenario5
Scenario6
Meteorologic Models
Control Specifications
Components Compute Results
Reservoir Dam Break Options
Basin Name: Breach Hog Chute Dam
Element Name: Carson_Lake
Method: Overtop Breach
Direction: Main
*Top Elevation (FT) 9902
*Bottom Elevation (FT) 9851
*Bottom Width (FT) 53
*Left Slope (xH:1V) 1
*Right Slope (xH:1V) 1
*Development Time (HR) 0.35
Trigger Method: Elevation
*Trigger Elevation (FT) 9902.25
Progression Method: Linear
Basin Model [Breach Hog Chute Dam]
Sub_Lower1
Carson_Lake
JL2
JL7
NOTE 10008: Begin opening project "Hogchute" in directory
NOTE 10019: Finished opening project "Hogchute" in directo

Appendix B

Flood Maps (Depth and DV)



Purdy Mesa Rd

Kannah Creek Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

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CITY OF Grand Junction COLORADO

N

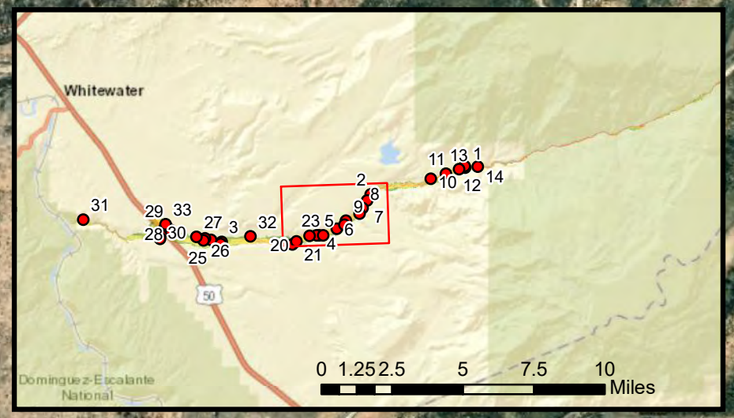
Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 1 of 4

March 5, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Purdy Mesa Rd
Purdy Mesa Rd
Purdy Mesa Rd
Purdy Mesa Rd
Purple Sage Ct
Lands End Rd
Purdy Mesa Rd
Divide Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Cre

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

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Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 2 of 4

March 5, 2020

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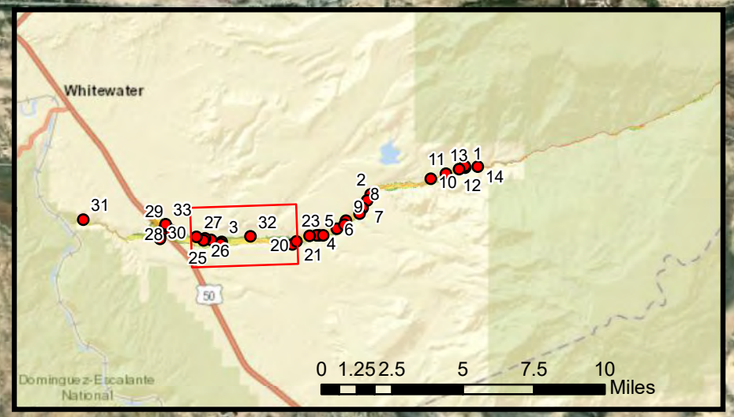

Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Whiting Rd
Whiting Rd
Purple Sage Ct
Lands End Rd
Purdy Mesa Rd
50
0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

AYRES RIVER ENGINEERING + WATER RESOURCES

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Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 3 of 4
March 5, 2020

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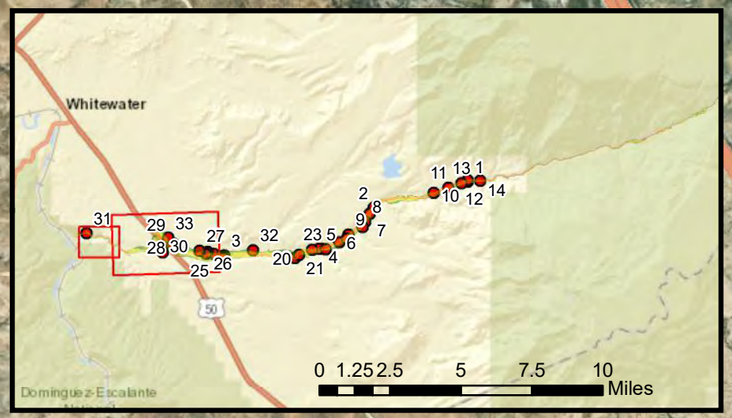

Old Kannah Creek Rd
Blair Rd
Crane
Simpson Rd
Prongh orn Dr
Kannah Creek Rd
Whiting R
Golden Eagle Ct
Eagle Trail Ct
Red Tail Ct
Bean Ranch Rd
50
0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

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Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 4 of 4
March 5, 2020

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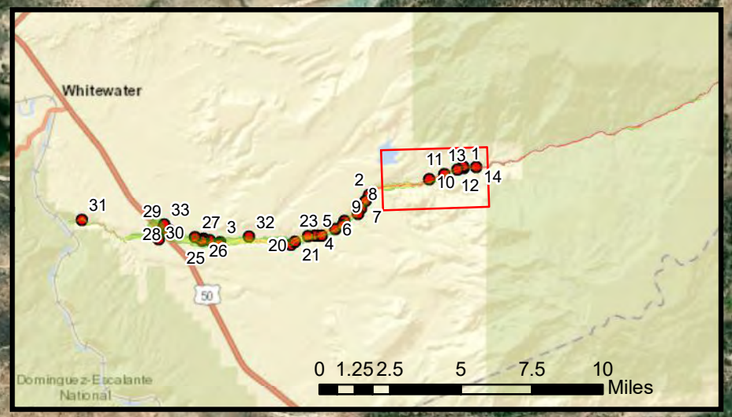

Flood Wave Arrival Time:
0 Hours 35 Minutes

Flood Wave Arrival Time:
0 Hours 30 Minutes

Flood Wave Arrival Time:
0 Hours 25 Minutes

Purdy Mesa Rd

Kannah Creek Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

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Maximum Depth x Velocity, Overtopping Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 1 of 4

February 27, 2020

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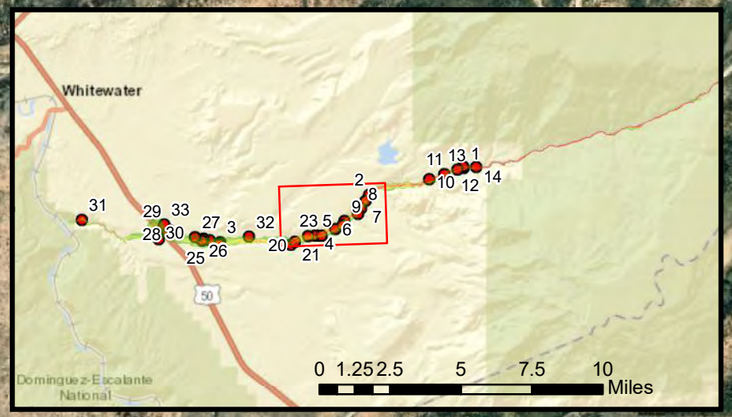

Flood Wave Arrival Time:
0 Hours 45 Minutes

Flood Wave Arrival Time:
0 Hours 55 Minutes

Flood Wave Arrival Time:
1 Hours 05 Minutes

Purdy Mesa Rd

Kannah Creek Rd

Purple Sage Ct

Lands End Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

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Maximum Depth x Velocity, Overtopping Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 2 of 4
February 27, 2020

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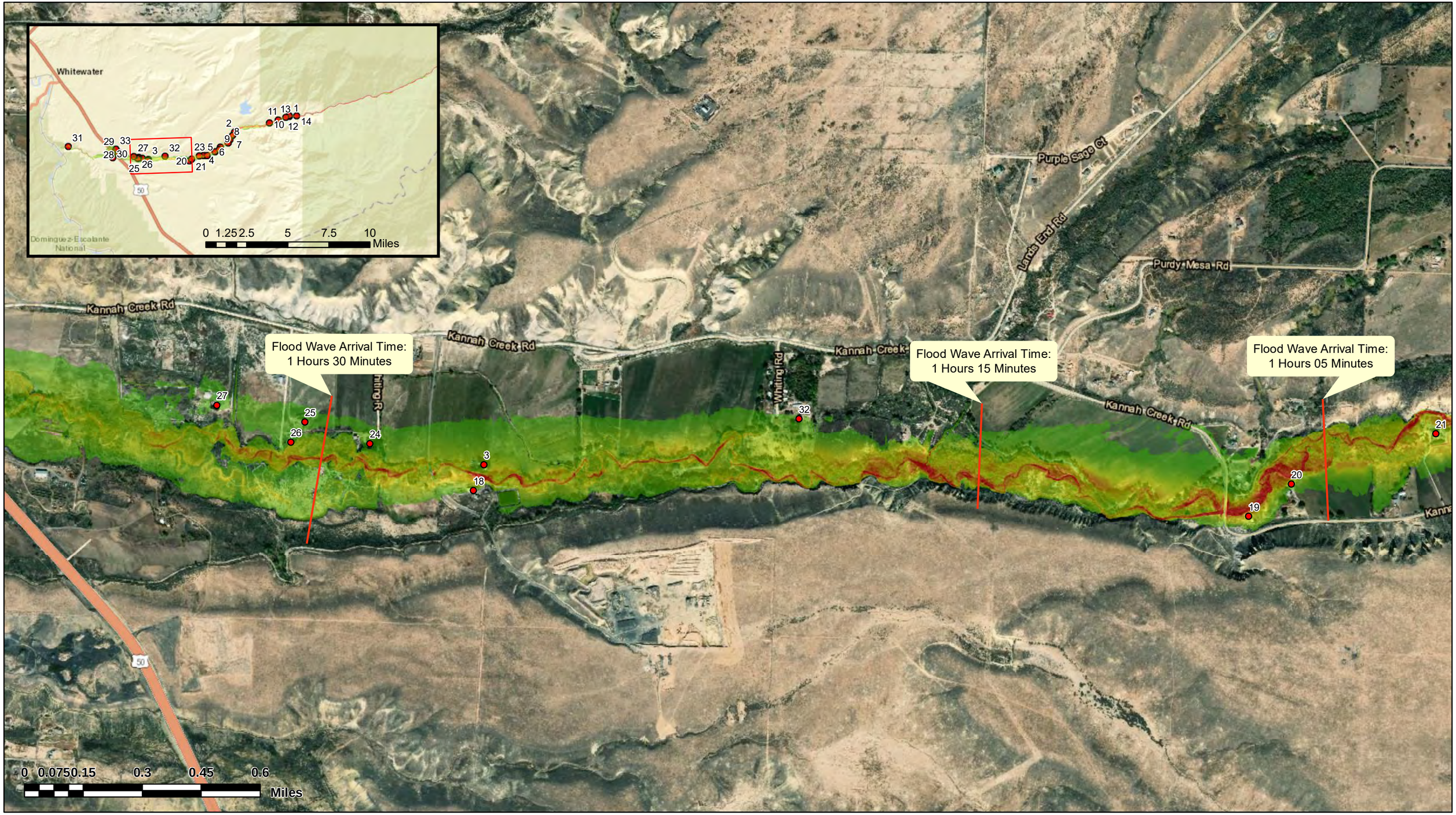

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

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N

Maximum Depth x Velocity, Overtopping Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 3 of 4

February 27, 2020

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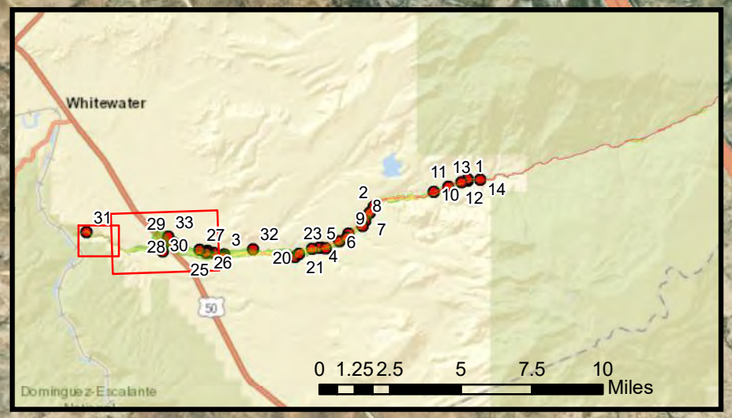

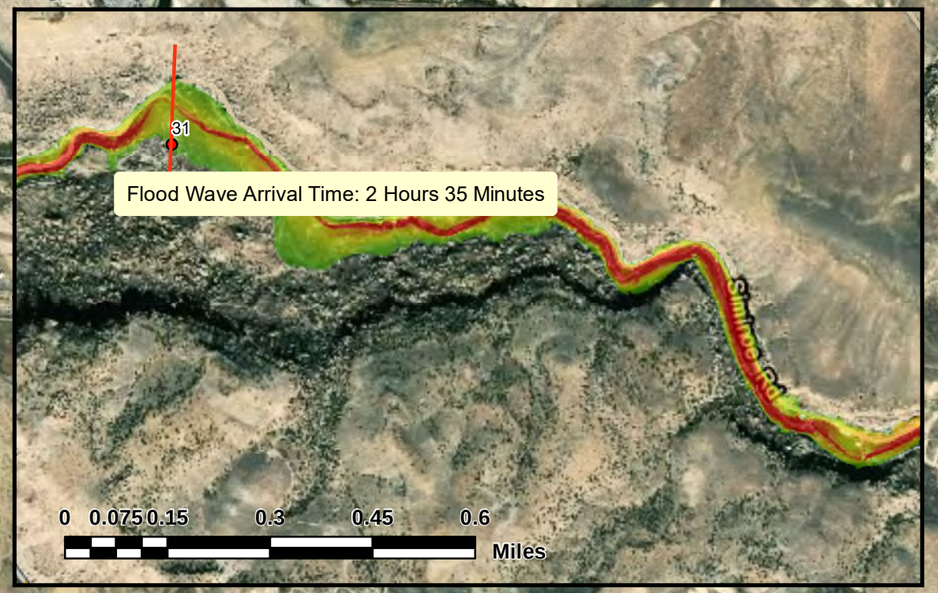

Flood Wave Arrival Time:
1 Hours 45 Minutes

Flood Wave Arrival Time:
2 Hours 05 Minutes

Flood Wave Arrival Time:
1 Hours 30 Minutes

Blair Rd

Kannah Creek Rd

Golden Eagle Ct

Red Tail Ct

Eagle Trail Ct

Bean Ranch Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

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N

Maximum Depth x Velocity, Overtopping Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 4 of 4

February 27, 2020

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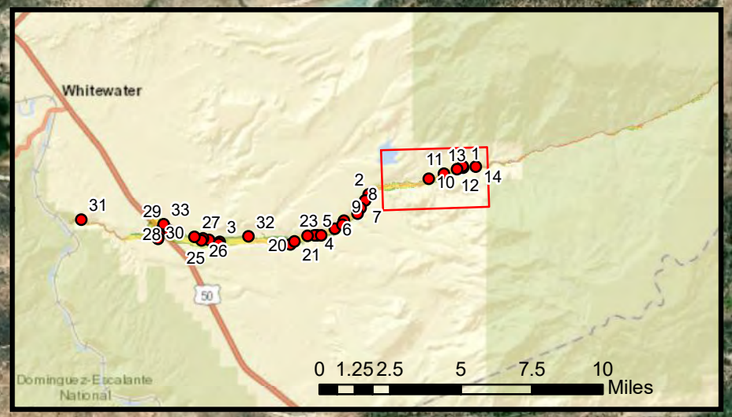

Purdy Mesa Rd

Kannah Creek Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

AYRES RIVER ENGINEERING + WATER RESOURCES

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Maximum Flooding Depth, PMF Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 1 of 4

March 5, 2020

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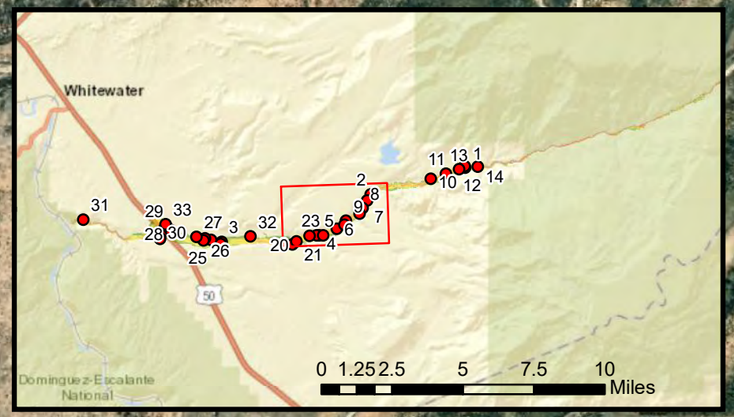

Purdy Mesa Rd
Purdy Mesa Rd
Purdy Mesa Rd
Purdy Mesa Rd
Purple Sage Ct
Lands End Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Divide Rd
Kannah Cre

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Maximum Flooding Depth, PMF Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 2 of 4
March 5, 2020

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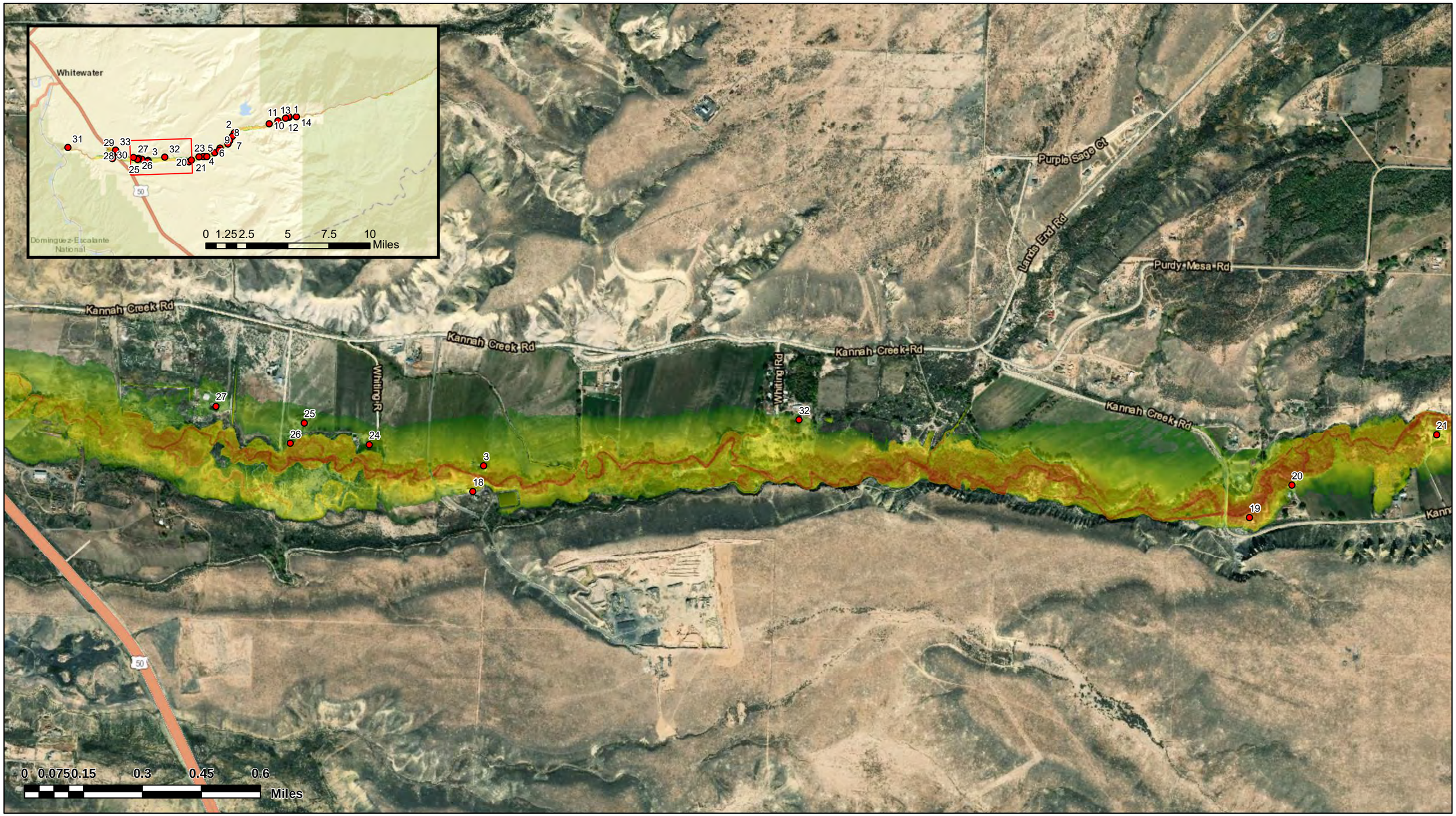

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Maximum Flooding Depth, PMF Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 3 of 4

March 5, 2020

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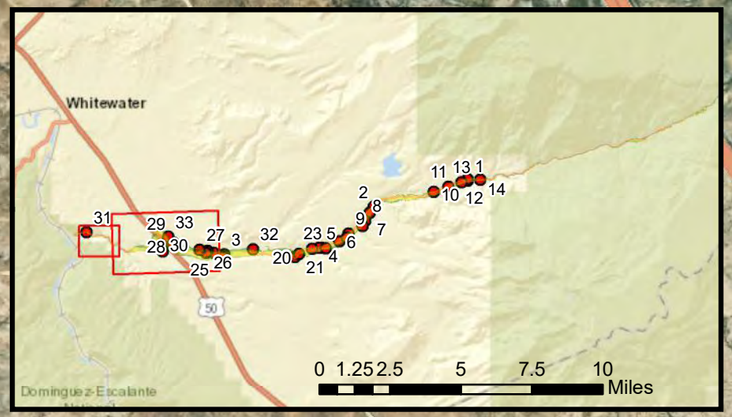
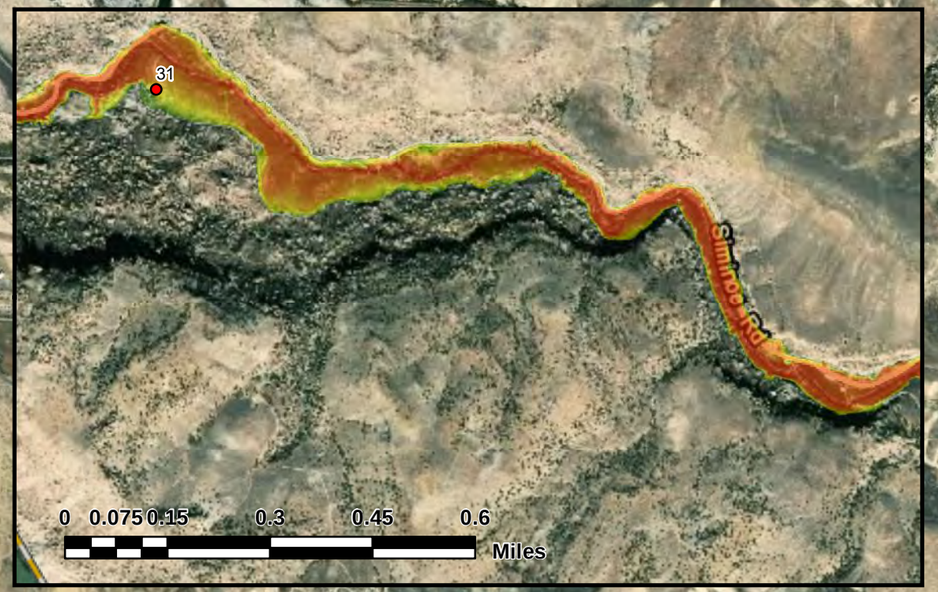




Old Kannah Creek Rd
Blair Rd
Kannah Creek Rd
Pronghorn Dr
Golden Eagle Ct
Red Tail Ct
Eagle Trail Ct
Bean Ranch Rd
Whiting R
0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth (ft)

0 - 1 | 1 - 5 | 5 - 10 | 10 - 15 | > 15

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

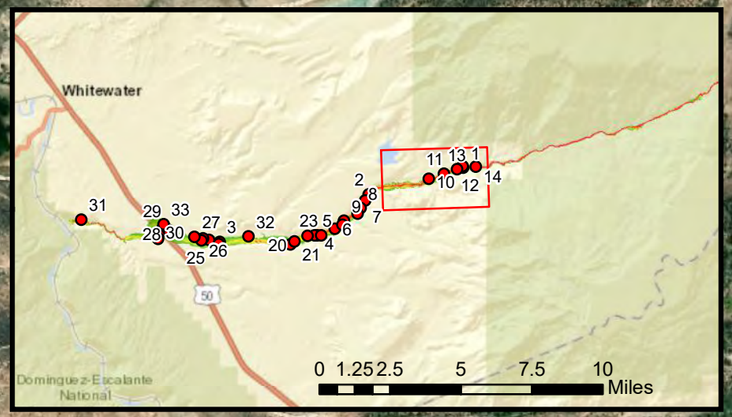
Maximum Flooding Depth, PMF Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 4 of 4

March 5, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Flood Wave Arrival Time:
0 Hours 40 Minutes

Flood Wave Arrival Time:
0 Hours 35 Minutes

Flood Wave Arrival Time:
0 Hours 20 Minutes

Purdy Mesa Rd

Kannah Creek Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

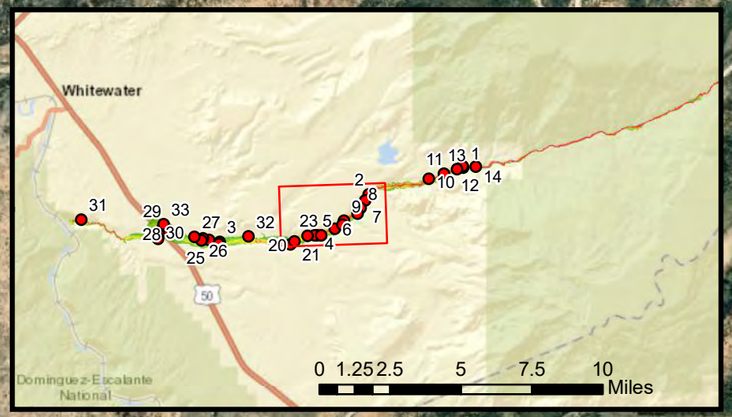
AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Maximum Depth x Velocity, PMF Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 1 of 4
February 27, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Flood Wave Arrival Time:
0 Hours 50 Minutes

Flood Wave Arrival Time:
0 Hours 55 Minutes

Flood Wave Arrival Time:
1 Hours 05 Minutes

Purdy Mesa Rd
Purple Sage Ct
Lands End Rd
Kannah Creek Rd
Kannah Cre

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

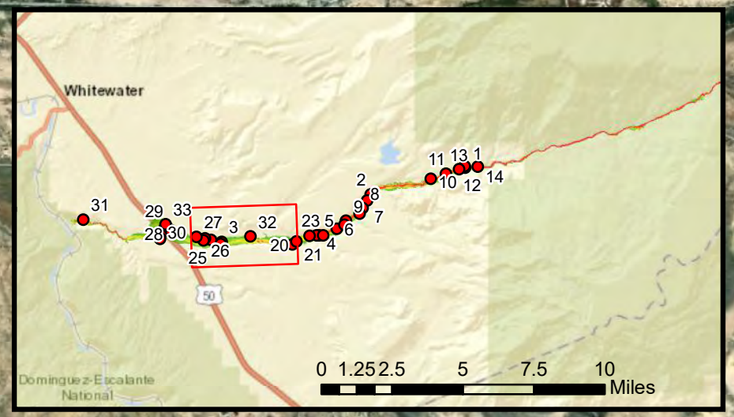
AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Maximum Depth x Velocity, PMF Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 2 of 4
February 27, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Kannah Creek Rd

Flood Wave Arrival Time:
1 Hours 30 Minutes

Flood Wave Arrival Time:
1 Hours 10 Minutes

Flood Wave Arrival Time:
1 Hours 05 Minutes

Purple Sage Ct

Lands End Rd

Purdy Mesa Rd

Whiting Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

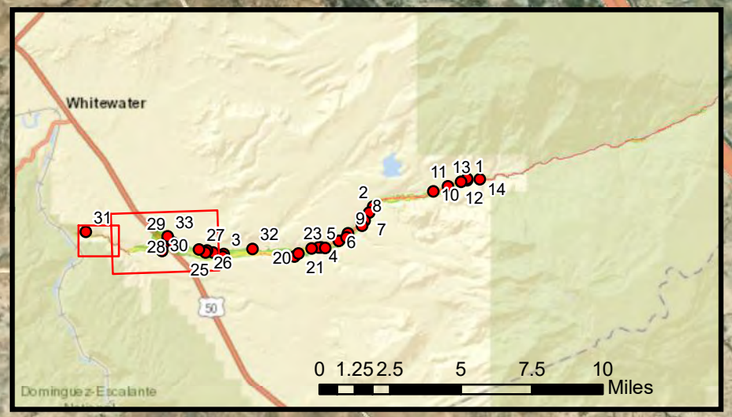
AYRES RIVER ENGINEERING + WATER RESOURCES

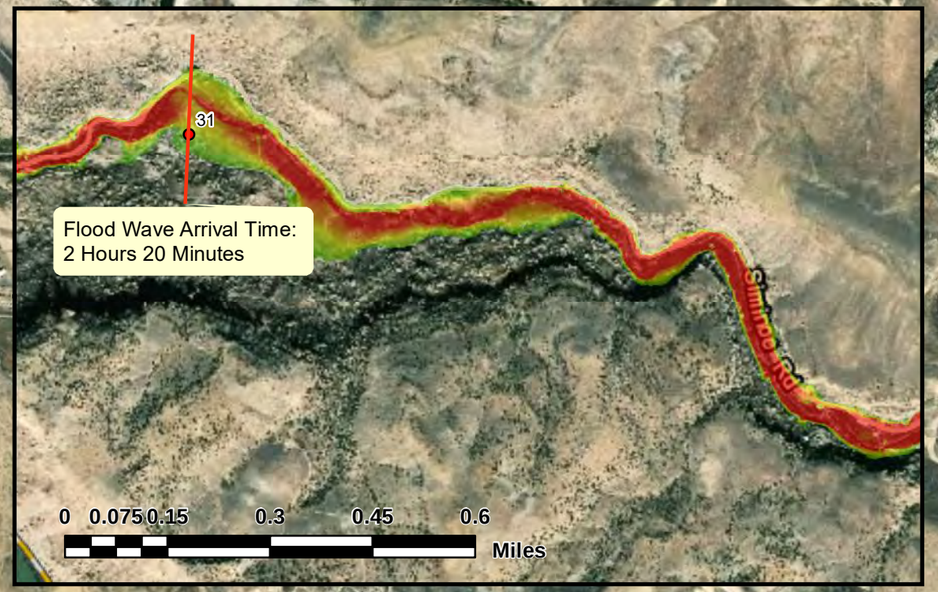
CITY OF Grand Junction COLORADO

N

Maximum Depth x Velocity, PMF Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 3 of 4
February 27, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.





Flood Wave Arrival Time:
1 Hours 40 Minutes

Flood Wave Arrival Time:
2 Hours 00 Minutes

Flood Wave Arrival Time:
1 Hours 30 Minutes

Blair Rd
Kannah Creek Rd
Summer Rd
Golden Eagle Ct
Red Tail Ct
Eagle Trail Ct
Bean Ranch Rd
50

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations

Maximum Depth x Velocity (ft^2 / s)

0 - 25 | 25 - 50 | 50 - 75 | 75 - 100 | > 100

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

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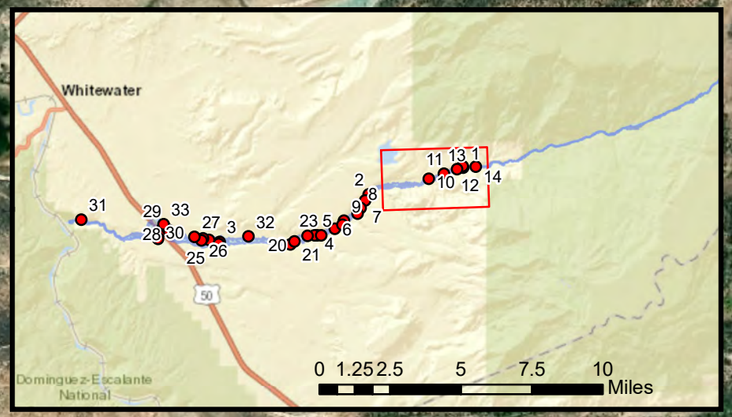
Maximum Depth x Velocity, PMF Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 4 of 4
February 27, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Purdy Mesa Rd

Kannah Creek Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

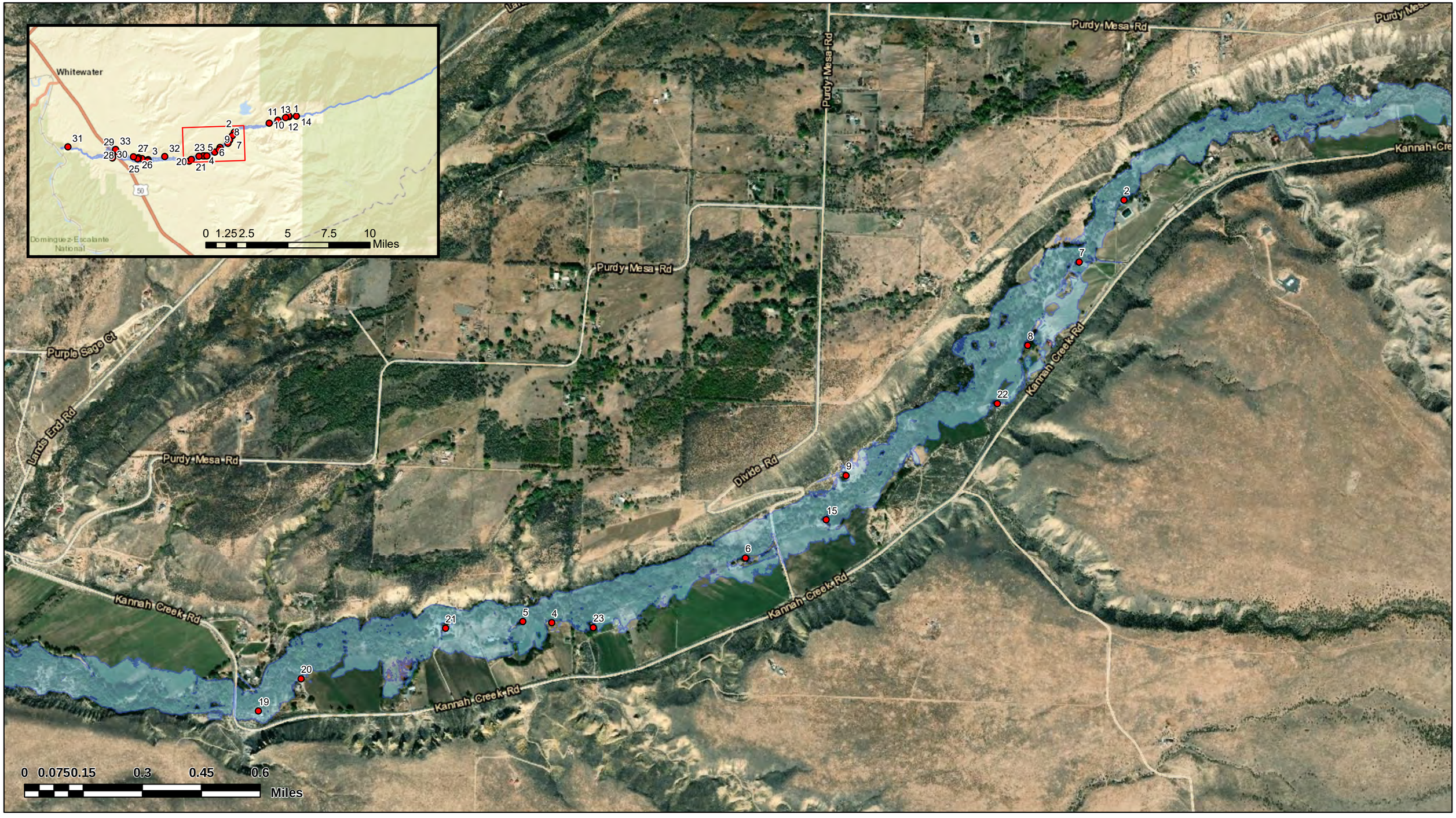
- House Locations
- Overtopping (No Failure) Inundation

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 1 of 4
February 28, 2020



Legend

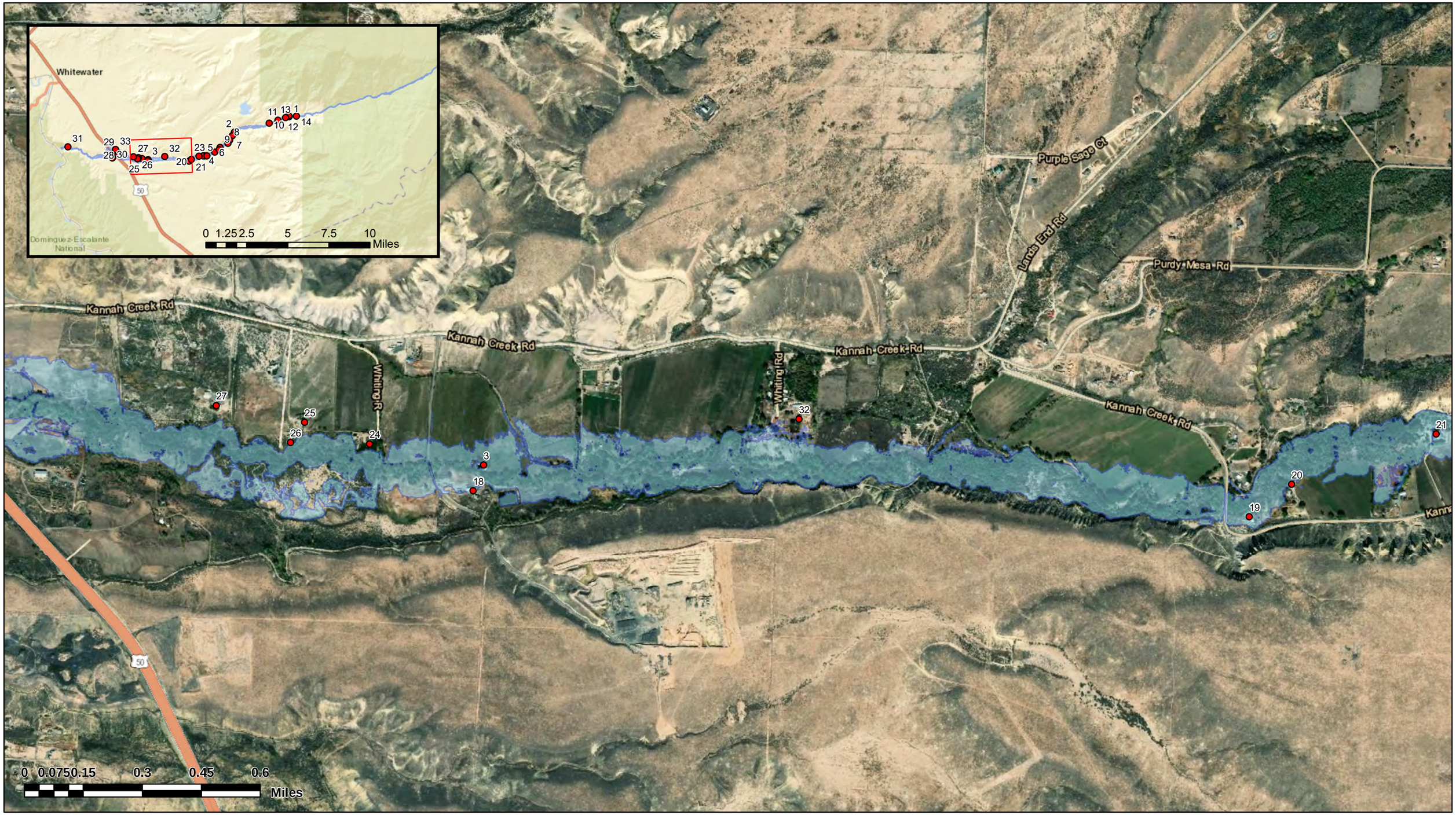
- House Locations
- Overtopping (No Failure) Inundation

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 2 of 4
February 28, 2020



Legend

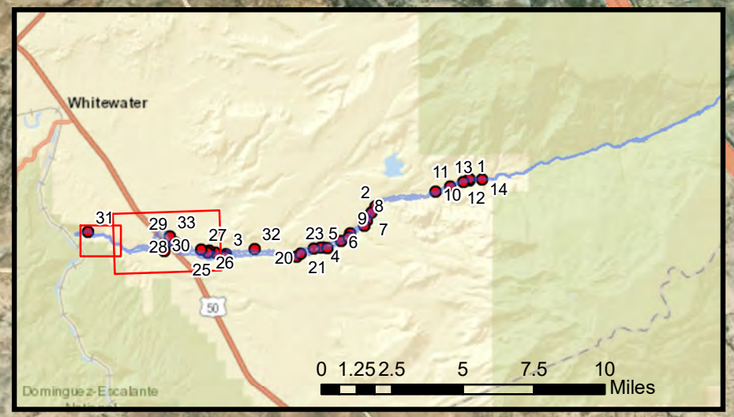
- House Locations
- Overtopping (No Failure) Inundation

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 3 of 4
February 28, 2020



Old Kannah Creek Rd
Blair Rd
Simpson Rd
Pierghom Dr
Kannah Creek Rd
Whitting R
Golden Eagle Ct
Red Tail Ct
Eagle Trail Ct
Bean Ranch Rd
50
0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations
- Overtopping (No Failure) Inundation

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 4 of 4
February 28, 2020

Appendix C

Fatality Rate Charts from RCEM Interim Guidelines

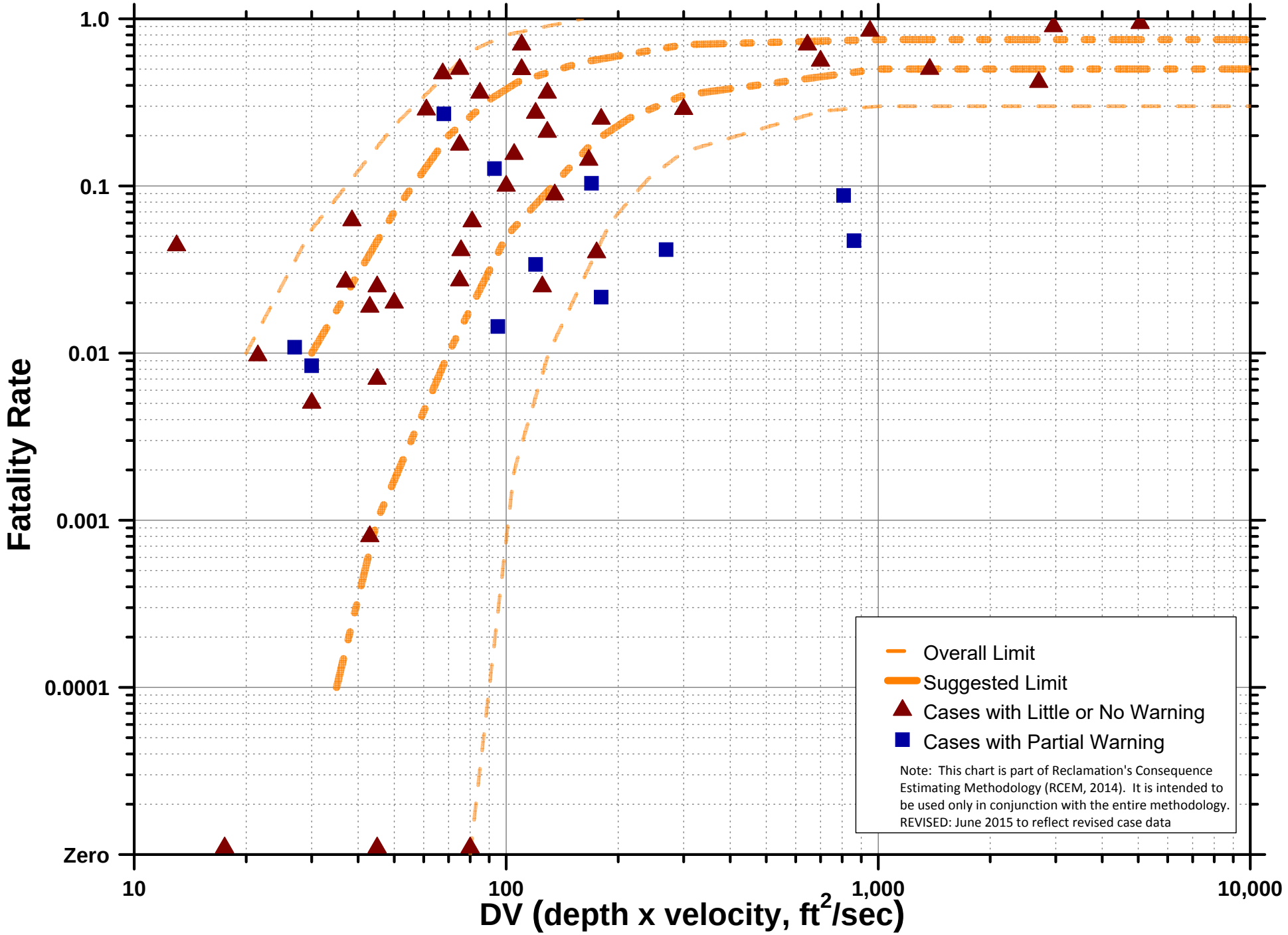


Figure 3 - Fatality Rate vs. DV for Little or No Warning

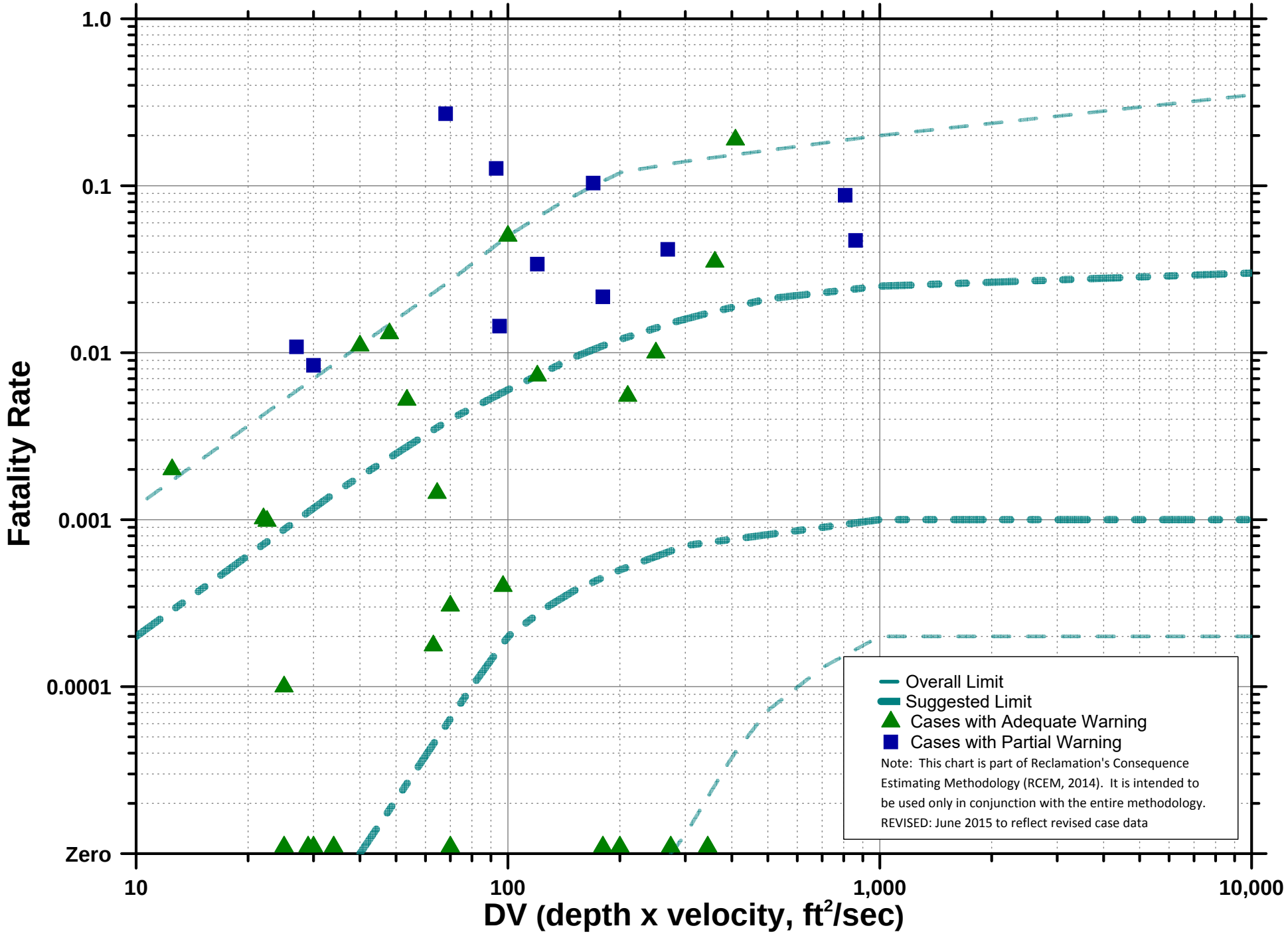


Figure 4 - Fatality Rate vs. DV for Adequate Warning

Appendix D

Estimates of Fatality Rate and Loss of Life at Downstream Homes

Predicted Fatalities Due to PMF Failure (Peak Outflow = 62,000 cfs), Little or No Warning					
House ID #	**House Type**	**Depth x Velocity (ft^2/s)**	**Fatality Rate***	**Predicted Fatalities****	**Wave Arrival Time (H:MM after failure)**
1	Mobile Home	78	0.5781	1.3874	0:20
2	Guest House	96	0.3526	0.8462	0:45
3	Single Family Residence	13	0.0010	0.0024	1:25
4	Single Family Residence	43	0.0407	0.0977	1:00
5	Single Family Residence	59	0.1123	0.2696	1:00
6	Single Family Residence	9	0.0010	0.0024	0:55
7	Single Family Residence	50	0.0689	0.1654	0:45
8	Single Family Residence	5	0.0010	0.0024	0:50
9	Single Family Residence	44	0.0423	0.1016	0:50
10	Single Family Residence / Monastery	142	0.5621	1.3490	0:30
11	Single Family Residence	78	0.2331	0.5595	0:25
12	Mobile Home	67	0.4221	1.0131	0:25
13	Single Family Residence	96	0.3529	0.8469	0:20
14	Single Family Residence	125	0.5061	1.2145	0:20
15	Single Family Residence	33	0.0151	0.0363	0:55
16	Mobile Home	26	0.0327	0.0785	1:45
17	Single Family Residence	8	0.0010	0.0024	1:45
18	Single Family Residence	10	0.0010	0.0024	1:25
19	Mobile Home	58	0.3036	0.7288	1:05
20	Single Family Residence	12	0.0010	0.0024	1:05
21	Single Family Residence	29	0.0093	0.0222	1:00
22	Single Family Residence	10	0.0010	0.0024	0:50
23	Single Family Residence	19	0.0010	0.0024	1:00
24	Single Family Residence	3	0.0010	0.0024	1:30
25	Single Family Residence	1	0.0010	0.0024	1:30
26	Mobile Home	6	0.0100	0.0240	1:30
27	Single Family Residence	1	0.0010	0.0024	1:35
28	Single Family Residence	1	0.0010	0.0024	1:45
29	Mobile Home	1***	0.0100	0.0240	1:50
30	Single Family Residence	0***	0.0010	0.0024	1:55
31	Mobile Home	24***	0.0227	0.0545	2:25
32	Single Family Residence	0	0.0010	0.0024	1:20
33	Mobile Home	0***	0.0100	0.0240	3:00
	Total Fatalities: 8.88				

*** Fatality rate estimates are based on the Depth x Velocity relationship found on Figures 3 and 4 of USBR's *Reclamation Consequence Estimating Methodology* (July 2015). The high "Suggested Limit" curve is used for all homes except mobile homes which use the high "Overall Limit" curve to account for the greater risk expected at lower quality shelters.**

**** Based on 2.4 persons per household (US Census Bureau records for Mesa County, Colorado, 2018); "Single Family Residence and Townhouse" treated as two households.**

***** Maximum Depth x Velocity occurs during PMF peak following breach flow peak.**

Predicted Fatalities Due to PMF Failure (Peak Outflow = 62,000 cfs), Adequate Warning					
House ID #	**House Type**	**Depth x Velocity (ft^2/s)**	**Fatality Rate***	**Predicted Fatalities****	**Wave Arrival Time (H:MM after failure)**
1	Mobile Home	78	0.0328	0.0788	0:20
2	Guest House	96	0.0058	0.0139	0:45
3	Single Family Residence	13	0.0003	0.0007	1:25
4	Single Family Residence	43	0.0020	0.0048	1:00
5	Single Family Residence	59	0.0031	0.0074	1:00
6	Single Family Residence	9	0.0002	0.0005	0:55
7	Single Family Residence	50	0.0025	0.0060	0:45
8	Single Family Residence	5	0.0002	0.0005	0:50
9	Single Family Residence	44	0.0020	0.0049	0:50
10	Single Family Residence / Monastery	142	0.0085	0.0203	0:30
11	Single Family Residence	78	0.0045	0.0108	0:25
12	Mobile Home	67	0.0251	0.0603	0:25
13	Single Family Residence	96	0.0058	0.0139	0:20
14	Single Family Residence	125	0.0076	0.0182	0:20
15	Single Family Residence	33	0.0013	0.0032	0:55
16	Mobile Home	26	0.0054	0.0129	1:45
17	Single Family Residence	8	0.0002	0.0005	1:45
18	Single Family Residence	10	0.0002	0.0005	1:25
19	Mobile Home	58	0.0197	0.0474	1:05
20	Single Family Residence	12	0.0002	0.0006	1:05
21	Single Family Residence	29	0.0011	0.0027	1:00
22	Single Family Residence	10	0.0002	0.0005	0:50
23	Single Family Residence	19	0.0005	0.0013	1:00
24	Single Family Residence	3	0.0002	0.0005	1:30
25	Single Family Residence	1	0.0002	0.0005	1:30
26	Mobile Home	6	0.0011	0.0026	1:30
27	Single Family Residence	1	0.0002	0.0005	1:35
28	Single Family Residence	1	0.0002	0.0005	1:45
29	Mobile Home	1***	0.0011	0.0026	1:50
30	Single Family Residence	0***	0.0002	0.0005	1:55
31	Mobile Home	24***	0.0045	0.0109	2:25
32	Single Family Residence	0	0.0002	0.0005	1:20
33	Mobile Home	0***	0.0011	0.0026	3:00
	Total Fatalities: 0.33				

*** Fatality rate estimates are based on the Depth x Velocity relationship found on Figures 3 and 4 of USBR's *Reclamation Consequence Estimating Methodology* (July 2015). The high "Suggested Limit" curve is used for all homes except mobile homes which use the high "Overall Limit" curve to account for the greater risk expected at lower quality shelters.**

**** Based on 2.4 persons per household (US Census Bureau records for Mesa County, Colorado, 2018); "Single Family Residence and Townhouse" treated as two households.**

***** Maximum Depth x Velocity occurs during PMF peak following breach flow peak.**

Predicted Fatalities Due to Overtopping Failure (Peak Outflow = 53,000 cfs), No Warning					
House ID #	House Type	Depth x Velocity (ft^2/s)	Fatality Rate*	Predicted Fatalities**	Wave Arrival Time (H:MM after failure)
1	Mobile Home	69	0.4535	1.0883	0:25
2	Guest House	72	0.1964	0.4713	0:40
3	Single Family Residence	8	0.0010	0.0024	1:25
4	Single Family Residence	39	0.0281	0.0673	1:00
5	Single Family Residence	53	0.0849	0.2038	1:00
6	Single Family Residence	6	0.0010	0.0024	0:55
7	Single Family Residence	44	0.0427	0.1026	0:40
8	Single Family Residence	4	0.0010	0.0024	0:45
9	Single Family Residence	40	0.0296	0.0710	0:50
10	Single Family Residence / Monastery	129	0.5220	1.2529	0:35
11	Single Family Residence	52	0.0759	0.1822	0:30
12	Mobile Home	54	0.2490	0.5976	0:25
13	Single Family Residence	85	0.2844	0.6826	0:25
14	Single Family Residence	111	0.4392	1.0540	0:20
15	Single Family Residence	30	0.0107	0.0257	0:55
16	Mobile Home	22	0.0185	0.0444	1:45
17	Single Family Residence	5	0.0010	0.0024	1:45
18	Single Family Residence	8	0.0010	0.0024	1:25
19	Mobile Home	52	0.2274	0.5457	1:05
20	Single Family Residence	10	0.0010	0.0024	1:05
21	Single Family Residence	23	0.0023	0.0056	1:00
22	Single Family Residence	7	0.0010	0.0024	0:45
23	Single Family Residence	16	0.0010	0.0024	1:00
24	Single Family Residence	2	0.0010	0.0024	1:30
25	Single Family Residence	1	0.0010	0.0024	-
26	Mobile Home	4	0.0100	0.0240	1:30
27	Single Family Residence	0	0.0000	0.0000	1:40
28	Single Family Residence	0	0.0010	0.0024	1:45
29	Mobile Home	0	0.0100	0.0240	1:55
30	Single Family Residence	-	0.0000	0.0000	-
31	Mobile Home	5	0.0100	0.0240	2:30
32	Single Family Residence	-	0.0010	0.0024	-
33	Mobile Home	-	0.0000	0.0000	-
	Total Fatalities: 6.5				

*** Fatality rate estimates are based on the Depth x Velocity relationship found on Figures 3 and 4 of USBR's *Reclamation Consequence Estimating Methodology* (July 2015). The high "Suggested Limit" curve is used for all homes except mobile homes which use the high "Overall Limit" curve to account for the greater risk expected at lower quality shelters.**

**** Based on 2.4 persons per household (US Census Bureau records for Mesa County, Colorado, 2018); "Single Family Residence and Townhouse" treated as two households.**

Predicted Fatalities Due to Overtopping Failure (Peak Outflow = 53,000 cfs), Adequate Warning					
House ID #	**House Type**	**Depth x Velocity (ft^2/s)**	**Fatality Rate***	**Predicted Fatalities****	**Wave Arrival Time (H:MM after failure)**
1	Mobile Home	69	0.0266	0.0638	0:25
2	Guest House	72	0.0041	0.0098	0:40
3	Single Family Residence	8	0.0002	0.0005	1:25
4	Single Family Residence	39	0.0017	0.0042	1:00
5	Single Family Residence	53	0.0027	0.0065	1:00
6	Single Family Residence	6	0.0002	0.0005	0:55
7	Single Family Residence	44	0.0021	0.0049	0:40
8	Single Family Residence	4	0.0002	0.0005	0:45
9	Single Family Residence	40	0.0018	0.0042	0:50
10	Single Family Residence / Monastery	129	0.0078	0.0188	0:35
11	Single Family Residence	52	0.0026	0.0062	0:30
12	Mobile Home	54	0.0172	0.0414	0:25
13	Single Family Residence	85	0.0051	0.0121	0:25
14	Single Family Residence	111	0.0067	0.0162	0:20
15	Single Family Residence	30	0.0012	0.0028	0:55
16	Mobile Home	22	0.0042	0.0100	1:45
17	Single Family Residence	5	0.0002	0.0005	1:45
18	Single Family Residence	8	0.0002	0.0005	1:25
19	Mobile Home	52	0.0163	0.0390	1:05
20	Single Family Residence	10	0.0002	0.0005	1:05
21	Single Family Residence	23	0.0007	0.0018	1:00
22	Single Family Residence	7	0.0002	0.0005	0:45
23	Single Family Residence	16	0.0004	0.0010	1:00
24	Single Family Residence	2	0.0002	0.0005	1:30
25	Single Family Residence	1	0.0002	0.0005	-
26	Mobile Home	4	0.0011	0.0026	1:30
27	Single Family Residence	0	0.0000	0.0000	1:40
28	Single Family Residence	0	0.0002	0.0005	1:45
29	Mobile Home	0	0.0011	0.0026	1:55
30	Single Family Residence	-	0.0000	0.0000	-
31	Mobile Home	5	0.0011	0.0026	2:30
32	Single Family Residence	-	0.0002	0.0005	-
33	Mobile Home	-	0.0000	0.0000	-
	Total Fatalities: 0.26				

*** Fatality rate estimates are based on the Depth x Velocity relationship found on Figures 3 and 4 of USBR's *Reclamation Consequence Estimating Methodology* (July 2015). The high "Suggested Limit" curve is used for all homes except mobile homes which use the high "Overall Limit" curve to account for the greater risk expected at lower quality shelters.**

**** Based on 2.4 persons per household (US Census Bureau records for Mesa County, Colorado, 2018); "Single Family Residence and Townhouse" treated as two households.**

Appendix B
Hydrology Study

MEMORANDUM

To:	John Eklund, PE		
From:	Dale Mathison, PE and Adam Schneider, PE (WI, CA, MO)		
Date:	June 16, 2020	Project No.:	26-1144.00
Re:	Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado		

Purpose of Memorandum

Our October 1, 2019 report (Ayres 2019) assigned the inflow design flood (IDF) for Carson Dam to be the probable maximum flood (PMF), assuming that the structure is classified as an extreme hydrologic hazard. Later correspondence with the State Engineer's Office (SEO), however, revealed that the hydrologic hazard for Carson Dam had never been officially determined.

Ayres then completed a Reclamation Consequence Estimation Methodology (RCEM) study (Ayres 2020) for the City of Grand Junction to determine the hydrologic hazard for Carson Dam. Ayres concluded through this study that the Carson Dam spillway presented a high hydrologic hazard (not extreme as previously thought) and would therefore only need to be sized to convey runoff from a 1 in 10,000-yr (0.01%) annual exceedance probability (AEP) storm (not the PMF). SEO's independent review of Ayres's study confirmed these findings. The high hydrologic hazard determination does require that an effective early warning system be implemented to reduce the risk posed to individuals downstream.

This memorandum accompanies Ayres (2019) and documents the computation of the 0.01% AEP IDF for Carson Lake. This memorandum also describes our computation of the 1 in 100-yr (1% AEP), 1 in 10-yr (10% AEP), and 1 in 2-yr (50% AEP) inflow hydrographs for Carson Lake, which we used as supplemental information to support design of the dam's spillway.

Watershed Model

Ayres (2019) describes the HEC-HMS model that we developed to compute the PMF inflow hydrograph to Carson Lake. To compute the additional inflow hydrographs described in this memorandum, we used the same HEC-HMS model with the same initial conditions and parameters. We only changed the precipitation applied to the HEC-HMS model to compute inflows for the frequency-based events of interest here.

Initial conditions and parameters used to estimate extreme events such as the PMF and the 0.01% AEP inflow may not be appropriate for computing smaller floods like the 1%, 10%, and 50% AEP events. Therefore, we checked our results against peak flow-frequency estimates derived from US Geological Survey (USGS) regression equations. These comparisons are also described in this memorandum.

Precipitation Input

As prescribed by the SEO, we obtained 0.01% AEP rainfall for the watershed above Carson Dam from the online MetPortal PF Tool (https://conm-reps-gui.shinyapps.io/metportal/). We found 2-hr local storms to be controlling for this application.

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www.AyresAssociates.com

Using the MetPortal PF Tool, we also extracted 1% and 10% AEP, 2-hr storms for the watershed of interest. 50% AEP storms are not available from the MetPortal PF Tool, so we derived an inflow hydrograph for this event using a scaling procedure described later in this document.

Consistent with guidance from the SEO (DWR 2020), we scaled up the 0.01%, 1%, and 10% AEP hyetographs by 7 percent to account for future climate change. Final hyetographs for these three events are presented in Table 1. The scaled hyetographs shown in the table were input directly into the HEC-HMS model to compute the various flood inflows.

Table 1. 0.01%, 1%, and 10% AEP, 2-hr rainfall hyetographs for watershed above Carson Dam

	Incremental Precipitation (in)					
	0.01% AEP		1% AEP		10% AEP	
Time (min)	Raw[1]	Scaled[2]	Raw[1]	Scaled[2]	Raw[1]	Scaled[2]
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0629	0.0674	0.0331	0.0354	0.0210	0.0225
10	0.0929	0.0994	0.0489	0.0523	0.0310	0.0332
15	0.1322	0.1414	0.0695	0.0743	0.0441	0.0472
20	0.2452	0.2624	0.1289	0.1380	0.0819	0.0876
25	0.5532	0.5919	0.2908	0.3112	0.1846	0.1976
30	0.3025	0.3236	0.1590	0.1702	0.1010	0.1080
35	0.2343	0.2508	0.1232	0.1318	0.0782	0.0837
40	0.1771	0.1895	0.0931	0.0996	0.0591	0.0633
45	0.1362	0.1458	0.0716	0.0766	0.0455	0.0487
50	0.1303	0.1394	0.0685	0.0733	0.0435	0.0465
55	0.1169	0.1251	0.0615	0.0658	0.0390	0.0418
60	0.1046	0.1120	0.0550	0.0589	0.0349	0.0374
65	0.0823	0.0881	0.0433	0.0463	0.0275	0.0294
70	0.0722	0.0773	0.0380	0.0406	0.0241	0.0258
75	0.0545	0.0583	0.0287	0.0307	0.0182	0.0195
80	0.0463	0.0496	0.0244	0.0261	0.0155	0.0165
85	0.0398	0.0426	0.0209	0.0224	0.0133	0.0142
90	0.0341	0.0364	0.0179	0.0192	0.0114	0.0122
95	0.0281	0.0300	0.0148	0.0158	0.0094	0.0100
100	0.0232	0.0248	0.0122	0.0130	0.0077	0.0083
105	0.0191	0.0204	0.0100	0.0107	0.0064	0.0068
110	0.0158	0.0169	0.0083	0.0089	0.0053	0.0056
115	0.0131	0.0140	0.0069	0.0074	0.0044	0.0047
120	0.0082	0.0087	0.0043	0.0046	0.0027	0.0029
125	0.0068	0.0073	0.0036	0.0038	0.0023	0.0024
130	0.0054	0.0058	0.0029	0.0031	0.0018	0.0019
135	0.0054	0.0058	0.0029	0.0031	0.0018	0.0019
140	0.0054	0.0058	0.0029	0.0031	0.0018	0.0019
145	0.0054	0.0058	0.0029	0.0031	0.0018	0.0019
150	0.0054	0.0058	0.0029	0.0031	0.0018	0.0019
155	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

1. 2-hr hyetographs as extracted from the MetPortal PF Tool
2. Scaled up by 7 percent to account for future climate change (DWR 2020)

Results

We applied the scaled hyetographs shown in Table 1 to the HEC-HMS basin model developed for the PMF simulation described in Ayres (2019). We kept all initial conditions, modeling parameters, and upstream dam breach assumptions unchanged from those used for the PMF. We implemented the commonly employed "design storm assumption" for all simulations, where the AEP of the applied precipitation is assigned to the computed hydrographs. Results are summarized in Table 2.

Table 2. Results of Carson Dam frequency storm inflow simulations

Result	0.01% AEP	1% AEP	10% AEP
Total precipitation depth (in)	2.95	1.55	0.99
Peak inflow (cfs)	5,930	1,802	295
Total inflow volume (ac-ft)	1,048	279	42
Maximum lake elevation (ft, NAVD88)	9,900.40	9,897.16	9,895.45
Peak spillway discharge (cfs)	5,035	1,317	179

To check the reasonableness of the computed 1%, and 10% AEP inflow hydrographs, we derived peak flow-frequency estimates for Carson Dam using regional regression equations published in Capesius et al. (2009). The regression equations for the Northwest Hydrologic Region, to where we assigned the Carson Dam watershed, require as input the drainage area (11.4 mi^2), the percentage of the drainage area above 7,500 ft (100%), and the mean annual precipitation (38.17 in, which we estimated using the USGS StreamStats tool). Resulting peak flow-frequency estimates are presented in Table 3.

Table 3. Peak inflow-frequency estimates for Carson Dam computed using USGS regional regression equations

AEP (%)	**Peak (cfs)**
1	503
10	306
50	167

As can be seen, the 1% AEP peak of 503 cfs is significantly less than the 1% AEP peak 1,802 cfs computed using rainfall-runoff modeling. For the 10% AEP event, peak inflows computed using the two methods compare favorably, with 306 cfs computed using the regression equation and 295 cfs computed using rainfall-runoff modeling. These comparisons indicate that our rainfall-runoff modeling results may be conservative, but reasonable.

For the 50% AEP event, we developed a full inflow hydrograph by scaling the 10% AEP rainfall by 0.925 and then rerunning the HEC-HMS simulation using the decreased rainfall to compute a peak inflow to Carson Lake of 170 cfs. We determined the scale factor of 0.925 through trial and error to produce a peak inflow approximately equal to the peak determined using the regional regression equation.

All inflow hydrographs described in this memorandum are tabulated, along with the corresponding outflow hydrographs, in Table 4. The 0.01% AEP inflow serves as the IDF for Carson Dam based on its assigned high hydrologic hazard rating. The 1%, 10%, and 50% AEP inflows provide supplemental information on which to base design of the dam's spillway.

Table 4. Tabulated inflow and outflow hydrographs for computed frequency storms

	HEC-HMS Simulation Results							
	0.01% AEP		1% AEP		10% AEP		50% AEP	
Time (min)	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow
0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0
10	1	0	0	0	0	0	0	0
15	2	0	1	0	1	0	1	0
20	7	1	4	0	2	0	2	0
25	30	2	9	1	5	0	5	0
30	100	7	18	2	10	1	9	1
35	310	23	39	4	17	2	15	2
40	873	68	89	9	26	4	24	3
45	1768	169	207	20	32	6	29	5
50	2676	333	415	43	32	8	29	7
55	3388	683	668	83	32	10	27	9
60	3906	1128	890	139	38	12	25	10
65	4361	1580	1057	206	64	15	28	11
70	4843	2184	1202	280	112	21	42	13
75	5315	2763	1351	360	169	30	68	17
80	5712	3336	1503	486	216	43	99	22
85	5928	3924	1648	650	242	58	124	29
90	**5930**	4399	1755	808	253	73	138	37
95	5766	4744	**1802**	954	260	88	145	46
100	5479	4953	1785	1080	269	102	149	54
105	5120	**5035**	1716	1181	282	116	155	62
110	4729	5009	1613	1254	293	130	162	69
115	4326	4894	1489	1299	**295**	143	169	77
120	3929	4712	1354	**1317**	288	155	**170**	85
125	3548	4480	1217	1312	270	165	166	91
130	3190	4216	1083	1288	245	172	154	97
135	2865	3933	958	1248	217	177	139	101
140	2569	3644	846	1196	189	**179**	122	103
145	2307	3357	747	1135	163	179	106	**104**
150	2075	3099	663	1071	141	177	91	104
155	1869	2873	590	1004	122	173	80	102
160	1690	2653	529	937	107	168	70	100
165	1532	2444	476	871	94	163	61	97
170	1396	2248	432	809	84	157	54	94
175	1279	2065	394	749	75	151	49	91
180	1179	1897	361	693	67	144	44	87
185	1094	1745	332	641	61	138	40	83
190	1017	1606	306	592	56	132	36	80
195	951	1507	281	547	51	125	33	76
200	894	1419	255	505	46	119	30	73
205	844	1336	230	466	43	113	28	69

1 of 4

	HEC-HMS Simulation Results							
	0.01% AEP		1% AEP		10% AEP		50% AEP	
Time (min)	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow
210	800	1259	209	429	39	107	26	66
215	777	1188	190	397	36	102	24	62
220	801	1128	173	380	33	96	22	59
225	856	1083	158	363	30	91	20	56
230	907	1052	145	346	28	86	18	53
235	932	1032	134	329	26	81	17	50
240	935	1018	123	313	24	77	16	48
245	923	1004	114	297	22	72	15	45
250	902	990	106	282	20	68	14	43
255	875	975	99	268	19	64	13	40
260	846	958	93	254	17	61	12	38
265	815	939	87	241	16	57	11	36
270	781	918	82	229	15	54	10	34
275	746	894	78	217	14	51	9	32
280	713	870	74	205	13	48	9	30
285	683	844	70	195	12	45	8	28
290	655	818	67	185	12	42	8	27
295	629	791	64	175	11	40	7	25
300	604	765	61	166	10	37	7	24
305	582	739	59	157	10	35	7	22
310	561	714	57	149	9	33	6	21
315	542	689	55	142	9	31	6	20
320	523	666	53	135	8	29	6	19
325	504	643	52	128	8	28	5	18
330	487	621	50	122	7	26	5	17
335	472	599	49	116	7	25	5	16
340	457	579	47	111	7	23	5	15
345	443	560	46	106	6	22	5	14
350	430	541	45	101	6	21	4	13
355	419	524	44	96	6	19	4	13
360	407	507	44	92	6	18	4	12
365	397	491	43	88	6	17	4	11
370	384	476	43	85	5	16	4	11
375	371	461	42	81	5	15	4	10
380	358	447	42	78	5	15	4	10
385	346	432	41	75	5	14	4	9
390	336	419	40	72	5	13	3	9
395	326	405	39	70	5	13	3	8
400	317	396	39	67	5	12	3	8
405	308	389	38	65	4	11	3	8
410	299	383	38	63	4	11	3	7
415	291	376	38	61	4	10	3	7

	HEC-HMS Simulation Results							
	0.01% AEP		1% AEP		10% AEP		50% AEP	
Time (min)	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow
420	284	369	38	59	4	10	3	6
425	276	361	38	57	3	9	3	6
430	269	354	38	56	3	9	3	6
435	262	347	38	54	3	8	3	6
440	256	340	38	53	3	8	3	5
445	250	333	38	52	3	8	2	5
450	244	326	38	51	3	7	2	5
455	238	319	39	50	3	7	2	5
460	233	313	39	49	3	7	2	5
465	228	306	39	48	3	6	2	4
470	223	300	39	47	3	6	2	4
475	218	293	39	47	3	6	2	4
480	214	287	39	46	3	6	2	4
485	210	281	40	46	3	5	2	4
490	208	275	40	45	3	5	2	4
495	207	270	40	45	3	5	2	4
500	206	265	40	44	3	5	2	4
505	205	260	40	44	3	5	2	3
510	204	256	40	44	3	5	2	3
515	203	251	41	43	3	5	2	3
520	202	248	41	43	3	4	2	3
525	201	244	41	43	3	4	2	3
530	200	240	41	43	3	4	2	3
535	199	237	41	43	3	4	2	3
540	198	234	42	43	3	4	2	3
545	196	231	42	43	3	4	2	3
550	195	228	42	42	3	4	2	3
555	194	226	42	42	3	4	2	3
560	193	223	42	42	3	4	2	3
565	191	220	42	42	3	4	2	3
570	190	218	42	42	3	4	2	3
575	189	216	42	42	3	4	2	3
580	187	214	42	42	3	4	2	3
585	186	211	42	42	3	4	2	3
590	184	209	42	42	3	4	2	3
595	183	207	42	42	3	4	2	3
600	181	205	43	42	3	4	2	3
605	180	203	43	42	3	4	2	3
610	178	201	43	42	3	4	2	3
615	177	199	43	42	3	4	2	3
620	175	197	43	42	3	3	2	3
625	174	196	43	42	3	3	2	3

	HEC-HMS Simulation Results							
	0.01% AEP		1% AEP		10% AEP		50% AEP	
Time (min)	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow
630	172	194	42	42	3	3	2	3
635	170	192	42	42	3	3	2	3
640	169	190	42	42	3	3	2	3
645	167	188	42	42	3	3	2	3
650	166	187	42	42	3	3	2	3
655	164	185	42	42	3	3	2	3
660	163	183	42	42	3	3	2	3
665	161	181	42	42	3	3	2	2
670	160	180	42	42	3	3	2	2
675	158	178	42	42	3	3	2	2
680	157	176	42	42	3	3	2	2
685	155	175	42	42	3	3	2	2
690	154	173	42	42	3	3	2	2
695	152	172	41	42	3	3	2	2
700	151	170	41	42	3	3	2	2
705	149	168	41	42	3	3	2	2
710	148	167	41	42	3	3	2	2
715	146	165	41	42	3	3	2	2
720	145	164	41	42	3	3	2	2

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Carson Lake

Dam ID 420127
Improvements to Dam, Spillway, and Outlet Works

Hydrology Update

DRAFT

Prepared for:

City of Grand Junction, CO

October 1, 2019

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Carson Lake

Dam ID 420127
Improvements to Dam, Spillway, and Outlet Works

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Contents

Page No.

Executive Summary 1
Background 1
Tasks 1
Actions 1
Results 2
Watershed Delineation 3
Runoff Volume (Losses) 4
Initial loss 4
Constant Loss Rate 5
Percent Impervious Area 7
Rainfall-Runoff Transform 8
Hydrologic Routing 9
Upstream Storage 9
Reservoirs 9
Upstream Dam Failures 10
Highway 65 Attenuation Potential 10
Modeling Carson Reservoir and Dam 11
Probable Maximum Precipitation 13
Results and Sensitivity 14
PMF Simulation Results 14
Sensitivity of Results 14
References 16

List of Appendices

Appendix A 8-Point Cross Sections
Appendix B Upstream Reservoir Information
Appendix C Dam Breach Configuration
Appendix D Carson Dam and Reservoir Configuration

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List of Figures

Page No.

Figure 1. Carson Lake PMF results 2
Figure 2. Carson Dam watershed delineation 3
Figure 3. HEC-HMS basin model representation of the Carson Dam watershed (not to scale) 4
Figure 4. XKSAT values by map unit (internal areas shown as white are classified as open water) 6
Figure 5. Ponding areas and total volume potential upstream of the Highway 65 embankment 12

List of Tables

Page No.

Table 1. Adopted initial loss (STRTL) values 5
Table 2. Final constant loss rate (XKSAT) derivation for subbasins 7
Table 3. Percent directly connected impervious area 7
Table 4. Basin lag times 8
Table 5. Routing reach parameters 9
Table 6. Upstream reservoir information 10
Table 7. Dam breach parameters 10
Table 8. Pertinent information for Carson Dam and Reservoir 11
Table 9. 2-hr Local PMP Storm Pattern 13
Table 10. Summary of Carson Lake PMF results 14
Table 11. Sensitivity Analysis Results 15

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Executive Summary

Background

The City of Grand Junction owns and operates Carson Lake (also known as Hogchute Dam Reservoir), located in Mesa County, CO. Carson Lake, which has a normal storage capacity of 637 ac-ft, is created by a 56-ft high earthen embankment impounding Kannah Creek. Originally designed and constructed in 1947, a recent Comprehensive Dam Safety Evaluation (CDSE) identified potential failure modes (PFMs) to serve as the basis for future dam rehabilitation design. To date, a preliminary hydrology study of the contributing watershed and a seepage and geotechnical investigation of the dam site have been completed. Now, Ayres Associates has been tasked with 1) finalizing the hydrology, and 2) designing improvements to the dam and spillway. Details of the design will be guided by the updated hydrology.

Based on a high hazard (and extreme hydrologic hazard) classification, RJH (2019) identified Carson Lake's inflow design flood (IDF) as the probable maximum flood (PMF). The preliminary hydrology study determined conclusively that the controlling PMF inflow was the result of a local (or thunderstorm-driven) event. This modeled storm produced a peak inflow of 35,097 cfs, a total inflow volume of 4,937 ac-ft, a peak outflow of 34,629 cfs, and a maximum pool elevation of 9,906.9 ft, which overtops Carson Dam by a maximum depth of 4.9 ft for a duration of 2.5 hours (RJH 2019). RJH submitted this hydrology report to the Colorado Division of Water Resources (DWR) for review.

DWR reviewed RJH's hydrology report and provided comments in a January 2019 memorandum titled *Hogchute (DAMID: 420127) Hydrologic Modeling Discussion* (DWR 2019a). The memorandum states that "much conservatism was built into the Consultant's model and that some parameters may be adjusted or sensitivity analyses performed to better reflect actual site conditions. Doing so may produce less conservative, albeit more representative results." Further details are provided in DWR (2019a) indicating how specific transform and loss parameters may be modified to achieve this objective.

Tasks

We have been tasked with finalizing the Carson Lake hydrology to serve as the basis for dam and spillway rehabilitation design. Based on DWR (2019a) guidance, the objective of the hydrology update is to compute a PMF inflow hydrograph using best estimates of initial conditions and modeling parameters that are representative of expected watershed conditions, which may not necessarily be the most conservative values possible.

Actions

To update the Carson Lake hydrology, we:

1. Reviewed RJH's hydrology study (RJH 2019) and DWR's review comments (DWR 2019a).
2. Delineated the watershed above Carson Lake Dam.
3. Developed an HEC-HMS model of the watershed, implementing guidance from DWR (2019b) and Sabol (2008).
4. Developed probable maximum precipitation (PMP) storms for the watershed using the Colorado-New Mexico Regional Extreme Precipitation Study (REPS) tool, the use of which is described in DWR (2019c).

5. Applied the PMP storms to the HEC-HMS model of the watershed to compute the PMF inflow to Carson Lake.
6. Analyzed the sensitivity of results to changes in initial conditions and parameters.

Results

Results from the controlling PMF inflow to Carson Lake are shown in Figure 1. The controlling inflow resulted from the 2-hr local PMP generated by the REPS tool. The inflow hydrograph has a peak of 22,214 cfs and a total volume of 4,703 ac-ft. The pool elevation rises to a maximum value of 9,905.14 ft, which overtops the dam crest elevation of 9,902 ft for approximately 3 hours. The resulting peak discharge from the lake, which includes flow through both the spillway and over the top of dam, is 22,107 cfs.

The peak inflow presented here (22,214 cfs) is significantly less than the 35,097 cfs documented in RJH (2019). Total storm volumes compare more closely, however, with RJH's inflow volume of 4,937 ac-ft only slightly exceeding the 4,703 ac-ft computed for this update. For both studies, dam overtopping occurs for about 2.5 to 3 hours.

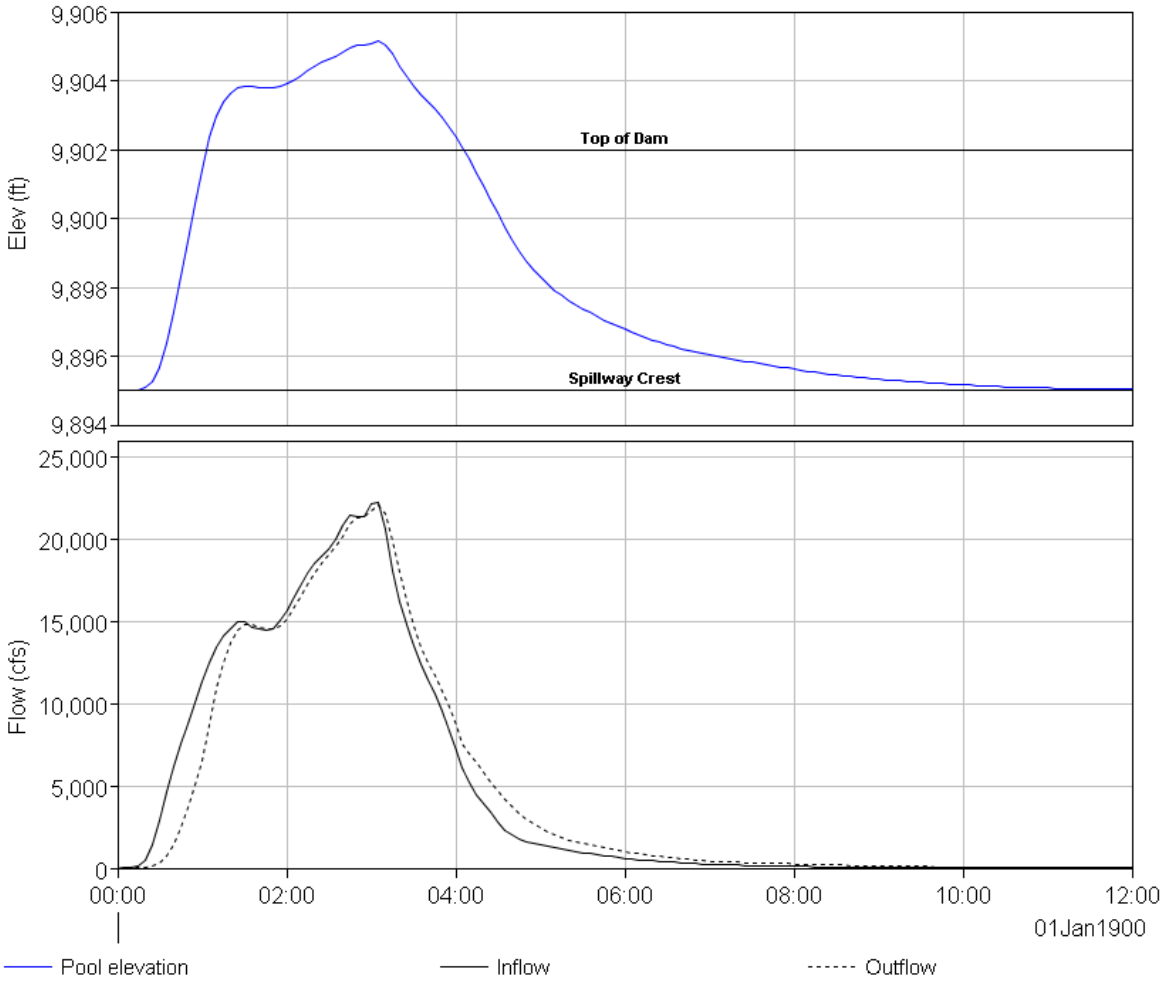

Figure 1. Carson Lake PMF results

Watershed Delineation

We delineated the watershed above Carson Dam using HEC-GeoHMS tools and the US Geological Survey (USGS) 10-m digital elevation model (DEM). Prior to subdividing the watershed, we imposed onto the terrain the Highway 65 embankment as a "wall". This raised the elevations of the DEM grid cells along the road embankment so that water can only pass through at designated locations—similar to how a levee would be represented. We also imposed an ortho-corrected streamline onto the DEM to "burn in" known flow paths. This "burn in" procedure allowed water to pass through the Highway 65 embankment at 2 locations: Kannah Creek between Grand Mesa #8 and Grand Mesa #9 reservoirs and Reservoir Creek upstream of Grand Mesa #6 reservoir. We placed subbasin outlets at the dams for Grand Mesa #6, #8, and #9 reservoirs, at the Kannah Creek and Reservoir Creek crossings of Highway 65, and at significant stream confluences in the watershed. The final subbasin delineation, covering a total watershed area of 11 mi^2, is shown in Figure 2. Subbasin labels shown in the figure are the names we assigned for use in HEC-HMS.

Figure 2. Carson Dam watershed delineation

After delineating the watershed, we developed a basin model using HEC-HMS version 4.3—a screenshot of which is shown in Figure 3. The basin model includes subbasins, reservoirs (Grand Mesa #6, #8, and

#9 and Carson Lake), routing reaches (depicted by the solid blue lines), and junctions. We describe model parameterization in the following sections.

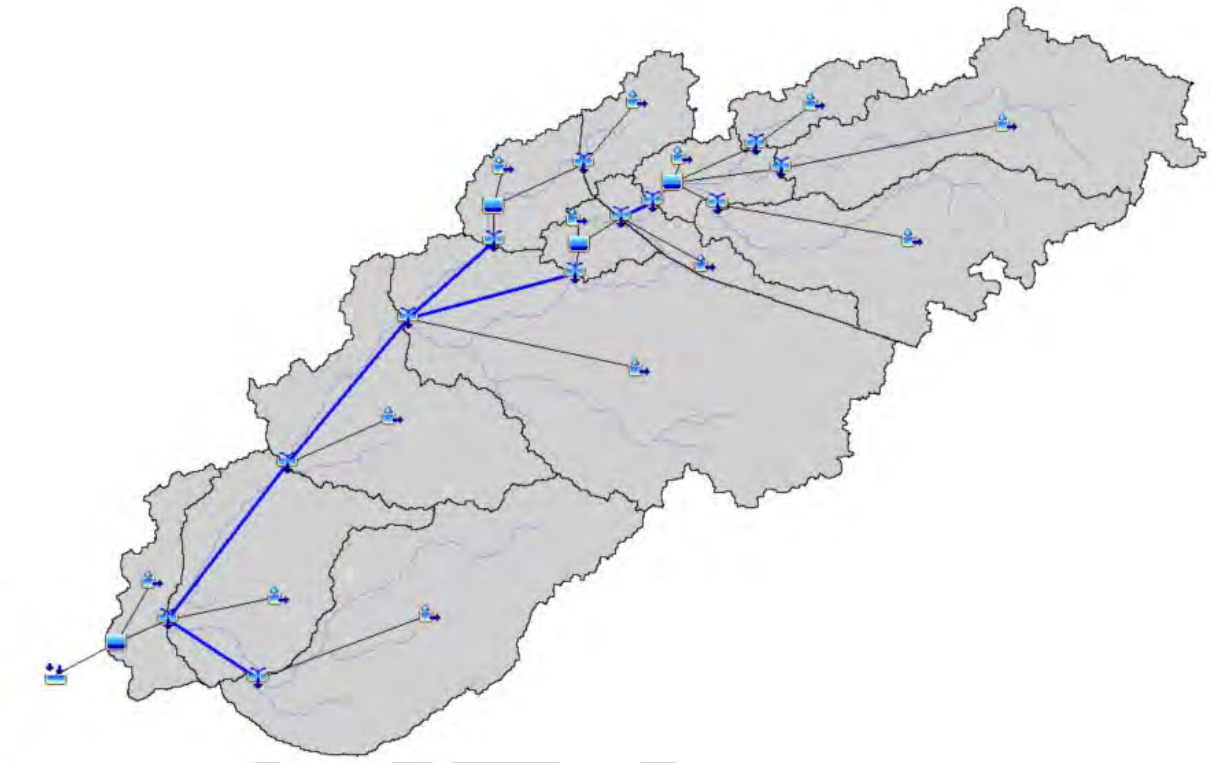

Figure 3. HEC-HMS basin model representation of the Carson Dam watershed (not to scale)

Runoff Volume (Losses)

Consistent with guidance from Sabol (2008), we used the initial and constant loss method in HEC-HMS. This method requires that one initial condition (the initial loss) and two parameters (the constant loss rate and percent directly connected impervious) be specified.

Initial loss

The initial loss is referred to in Sabol (2008) as STRTL and consists of two components, a surface retention loss (*Ia*) and an initial infiltration (*Il*). For each subbasin, we estimated *Ia* using Table 8 from Sabol (2008), which assigns a value based on the average slope and the percent vegetation. We computed average slopes using GIS tools and a 1-m LiDAR DEM of the watershed area. Our percent vegetation estimates are based on available land cover data and aerial imagery. We estimated *Il* for each subbasin using Table 12 from Sabol (2008), which provides *Il* values as a function of constant loss rates and watershed conditions (dry, normal, or saturated). Derivation of constant loss rates is described in the following section and, based on review comments provided in DWR (2019a), we chose *Il* values for each subbasin assuming normal watershed conditions.

Table 1. Adopted initial loss (STRTL) values

Subbasin	*Ia* (in)	*Il* (in)	STRTL (in)
Sub_Lower1	0.4	0.3	0.7
Sub_Lower2	0.4	0.3	0.7
Sub_Lower3	0.4	0.3	0.7
Sub_Lower4	0.4	0.3	0.7
Sub_Lower5	0.3	0.3	0.6
Sub_Lower6	0.3	0.3	0.6
Sub_Lower7	0.6	0.3	0.9
Sub_Upper1	0.3	0.3	0.6
Sub_Upper2	0.4	0.3	0.7
Sub_Upper3	0.4	0.3	0.7
Sub_Upper4	0.6	0.3	0.9
Sub_Upper5	0.6	0.3	0.9
Sub_Upper6	0.6	0.3	0.9

Constant Loss Rate

After initial loss volumes are satisfied for each subbasin, further losses are computed using a constant loss rate. Sabol (2008) describes how these constant loss rates can be computed using the same technique as the Green-Ampt bare-ground saturated hydraulic conductivity (XKSAT). We followed this procedure here.

We obtained a shapefile of Soil Survey Geographic Database (SSURGO) soil map units over the watershed and identified the map units for which soil data would be needed. Then, from the SSURGO database tables, we generated the Engineering Properties and Component Legend reports for all map units within the watershed.

We used the Component Legend report to identify the major soil components found in each map unit of interest and the percent area of each of those components. Then we used the Engineering Properties report to identify the controlling soil texture for each component in each map unit. Controlling soil textures were determined by considering the top 16 inches of the soil horizon. Sabol (2008) states that "for less frequent floods including the PMF, examine the soil to … at least 12 inches but no more than 18 inches" when selecting the controlling horizon. We considered the top 16 inches to avoid identifying a controlling horizon of clay found in many soil components at depths greater than 16 inches but less than 18 inches. We judged these clay layers to be deep enough below the ground surface that considering them to be controlling would overly conservative.

After identifying controlling horizons for each component in each map unit, we related those controlling textures to bare-ground hydraulic conductivities using Table 10 from Sabol (2008). Then, we computed an area-weighted average hydraulic conductivity for each map unit. The area-weighted XKSAT values assigned to each map unit are shown in Figure 4.

Using the coverage of XKSAT values for the watershed soil map units, we computed area-weighted XKSAT values for each subbasin. Then, we applied Figure 8 from Sabol (2008) to adjust these area-

weighted XKSAT values based on the percent canopy estimated for each subbasin. The final adjusted XKSAT values, shown in Table 2, were applied to the HEC-HMS model as the constant loss rates.

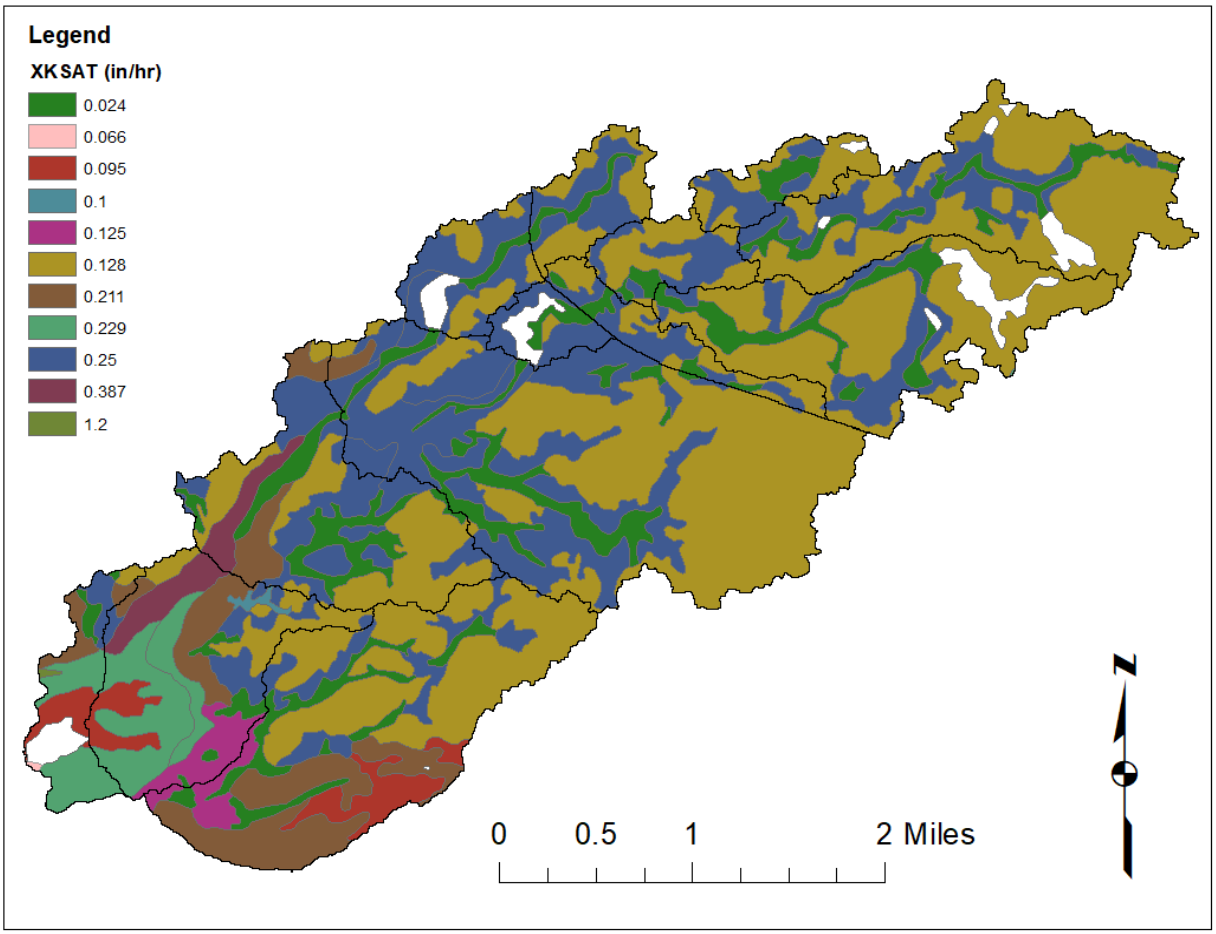

Figure 4. XKSAT values by map unit (internal areas shown as white are classified as open water)

Table 2. Final constant loss rate (XKSAT) derivation for subbasins

Subbasin	*Unadjusted XKSAT*[1] (in/hr)	Vegetation[2] (%)	Adjustment Factor[3] (in)	Final XKSAT[4] (in/hr)
Sub_Lower1	0.17	63	1.59	0.27
Sub_Lower2	0.18	50	1.44	0.26
Sub_Lower3	0.13	33	1.26	0.16
Sub_Lower4	0.14	33	1.26	0.17
Sub_Lower5	0.19	25	1.17	0.23
Sub_Lower6	0.15	10	1.00	0.15
Sub_Lower7	0.13	50	1.44	0.18
Sub_Upper1	0.14	33	1.26	0.18
Sub_Upper2	0.14	40	1.33	0.18
Sub_Upper3	0.15	20	1.11	0.16
Sub_Upper4	0.12	50	1.44	0.18
Sub_Upper5	0.12	50	1.44	0.18
Sub_Upper6	0.11	50	1.44	0.16

1. Area-weighted averages computed from XKSAT values shown in Figure 4.
2. Estimated percent canopy from land cover data and aerial imagery.
3. Computed using Figure 8 in Sabol (2008).
4. Entered in HEC-HMS as the constant loss rate.

Percent Impervious Area

The percent directly connected impervious area (RTIMP) must be specified for each subbasin. Because the watershed is undeveloped, the only areas we considered to be directly connected impervious are those that correspond with open water. These estimates by subbasin are provided in Table 3.

Table 3. Percent directly connected impervious area

Subbasin	*RTIMP* (%)
Sub_Lower1	13.5
Sub_Lower2	0.0
Sub_Lower3	0.0
Sub_Lower4	0.0
Sub_Lower5	9.4
Sub_Lower6	23.3
Sub_Lower7	0.0
Sub_Upper1	0.0
Sub_Upper2	0.0
Sub_Upper3	0.0
Sub_Upper4	2.3
Sub_Upper5	4.0
Sub_Upper6	6.6

Rainfall-Runoff Transform

Sabol (2008) recommends that dimensionless unit hydrographs or S-graphs, both of which are documented in the US Bureau of Reclamation (USBR) *Flood Hydrology Manual* (Cudworth 1989), be selected as rainfall-runoff transform methods for design flood inflows in both the Rocky Mountain and Plateau regions of Colorado. Consistent with RJH (2019), we chose the S-graph method from Cudworth (1989) here. Based on topography and vegetation, we characterized the Carson Dam watershed as being within the Colorado Plateau region.

The selected unit hydrograph method requires that both an S-graph and a basin lag time be specified for each subbasin. For the S-graph, we used the Colorado Plateau curve presented in Table 4-14 of Cudworth (1989). We computed lag times using the following equation:

$$Lag = 26 \times K_n \times \left(\frac{LL_{ca}}{S^{0.5}}\right)^{0.33} \qquad \textit{(Equation 1)}$$

Where *Lag* is the basin lag time (in hours), K_n is a lumped parameter representing resistance to overland flow along the longest flow path, *L* is the length of the longest flow path (in miles), L_{ca} is the length of the centroidal longest flow path (in miles), and *S* is the slope of *L* (in feet per mile).

Sabol (2008) states that for the Colorado Plateau region, a reasonable range for K_n is 0.04 to 0.06 for the local storm PMF. Examining aerial imagery over the longest flow path in each subbasin, we assigned K_n toward the low end of that range if there appeared to be little impediment to flow and toward the high end if there appeared to be more storage and attenuation potential. Selected K_n values, other variables from Equation 1, and computed basin lag times are shown in Table 4.

Table 4. Basin lag times

Subbasin	K_n	L (mi)	L_{ca} (mi)	S (ft/mi)	Lag (hr)
Sub_Lower1	0.047	1.58	0.52	327.42	0.44
Sub_Lower2	0.047	2.25	0.58	291.56	0.52
Sub_Lower3	0.05	1.86	0.73	203.44	0.60
Sub_Lower4	0.05	3.66	1.69	118.09	1.08
Sub_Lower5	0.04	1.15	0.47	120.70	0.38
Sub_Lower6	0.04	0.93	0.31	48.51	0.37
Sub_Lower7	0.05	3.13	1.21	112.38	0.93
Sub_Upper1	0.06	1.20	0.57	92.09	0.65
Sub_Upper2	0.05	1.67	0.62	119.95	0.60
Sub_Upper3	0.04	1.07	0.37	66.67	0.38
Sub_Upper4	0.06	1.43	0.61	68.86	0.74
Sub_Upper5	0.06	3.63	1.94	50.55	1.56
Sub_Upper6	0.06	3.43	1.44	54.36	1.37

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Hydrologic Routing

We used Muskingum-Cunge routing for the reaches configured in the HEC-HMS model. This method requires that the length, slope, Manning's *n*, and cross section shape be specified for each modeled reach. We computed lengths and slopes in GIS using the ortho-corrected streamline and the LiDAR terrain data. For estimating Manning's *n* values in channel and overbank areas of each reach, we used guidance from Chow (1959) and aerial imagery. Lengths, slopes, and Manning's *n* values are listed in Table 5.

Table 5. Routing reach parameters

Reach	Length (ft)	Slope (ft/ft)	*n* (left)	*n* (channel)	*n* (right)
R_Blw_Mesa6	3674.9	0.022	0.05	0.04	0.05
R_Blw_Mesa9	6944.9	0.023	0.05	0.04	0.05
R_JL3_to_JL2	6507.1	0.058	0.05	0.05	0.05
R_JL4_to_JL3	6243.1	0.028	0.05	0.04	0.05
R_JL7_to_JL2	4045.9	0.107	0.06	0.04	0.06
R_Mesa8_to_HW65	1157	0.010	0.05	0.04	0.05

Cross sections are represented with 8-point geometries. We developed 8-point geometries using GIS tools and the 1-m LiDAR terrain by cutting cross sections in the upstream, middle, and downstream portions of each reach. Based on the shapes of the cross sections cut throughout each reach, we developed an 8-point idealized geometry for each and input those into the HEC-HMS model. Cross section shapes are provided in Appendix A.

Upstream Storage

Reservoirs

Three upstream reservoirs are represented explicitly in the HEC-HMS model: Grand Mesa #6, Grand Mesa #8, and Grand Mesa #9. Other reservoirs in the watershed—for example, Scales reservoirs #1 and #3—clearly have storage and attenuation potential. However, these reservoirs are in the far headwater portion of the watershed and, rather than modeling them as reservoirs in HEC-HMS, we accounted for them by increasing subbasin lag times.

Grand Mesa #8 and Grand Mesa #9 were included in the RJH (2019) model, so all information and data used for that effort were carried over for use here. Grand Mesa #6 was not included in the RJH (2019) model. We represented Grand Mesa #6 using an elevation-storage curve, dam crest information, and spillway crest information estimated from the 1-m LiDAR terrain. Pertinent information for all 3 upstream reservoirs is provided in Table 6. We estimated broad-crested weir coefficients for both the Grand Mesa #6 dam top and spillway crest using guidance from Brater and King (1976). Additional information and data for all 3 upstream reservoirs are included in Appendix B.

Table 6. Upstream reservoir information

Reservoir	Dam Top Elevation (ft)	Dam Length (ft)	Spillway Crest Elevation (ft)	Spillway Length (ft)
Grand Mesa #6	10,539	450	10,535.4	50
Grand Mesa #8	10,647.6	550.6	10,642.6	40
Grand Mesa #9	10,616	600	10,613	15

Upstream Dam Failures

Consistent with RJH (2019), we judged it reasonable to assume that Grand Mesa #6, #8, and #9 may undergo significant damage during the PMF. Therefore, we configured HEC-HMS to breach these dams in response to overtopping that could occur. RJH configured their breach models so that dam breaks occurred as soon as reservoir pools exceeded dam top elevations. Because all 3 dams appear to be armored with riprap, immediate failure upon overtopping is a conservative assumption. Therefore, we configured our dam breach models to initiate dam failure only when overtopping exceeds 1 foot. To serve as input to the HEC-HMS model, we calculated breach parameters using Froehlich (2008) equations. These parameters are shown in Table 7. Configuration of all dam breach parameters in the HEC-HMS model is provided in Appendix C.

Table 7. Dam breach parameters

Reservoir	Top Elevation[1] (ft)	Trigger Elevation[2] (ft)	Height of Breach[3] (ft)	Width of Breach Bottom[4] (ft)	Breach Development Time[5] (hr)
Grand Mesa #6	10,539	10,540	15	50.6	0.62
Grand Mesa #8	10,647.6	10,648.6	21	78.3	0.82
Grand Mesa #9	10,616	10,617	15	58.3	0.73

1. Dam crest elevation.
2. Failure is triggered when pool elevation exceeds 1 foot above the dam crest elevation.
3. Dam height (assume breach extends over entire height of dam).
4. Computed using Froehlich (2008), using breach side slopes of 1H:1V for overtopping failure.
5. Computed using Froehlich (2008).

Highway 65 Attenuation Potential

The Highway 65 embankment separates the upper and lower portions of the watershed, and discharge from the upper watershed area is conveyed into the lower watershed area through the embankment via culverts on Kannah Creek and Reservoir Creek. During the PMF, these culverts will be overwhelmed by flood flows, and water will back up significantly behind the Highway 65 embankment along both creeks. RJH (2019) did not model attenuation that could occur as a result of this storage.

Kannah Creek passes under Highway 65 through 2 culverts (38 and 48 inches in diameter) between Grand Mesa #8 and Grand Mesa #9 reservoirs. Using the 1-m LiDAR terrain, we estimated the ponding storage potential behind the highway embankment at this location. About 3.5 ac-ft of water is capable of being stored upstream of the embankment before road overtopping occurs. Given this small storage potential, and that the PMF discharges greatly exceed the capacities of the 2 culverts, water will rapidly

back up behind the embankment and then flow over it during the inflow design flood event. Because of the small storage potential, this will result in a near inflow equals outflow situation at the embankment, and further consideration of any attenuation potential is not warranted.

Details regarding the mechanism by which Reservoir Creek passes under Highway 65 just upstream of Grand Mesa #6 are not known from information available to us. While aerial imagery suggests that a culvert is present here, the Colorado Department of Transportation (CDOT) culvert inventory does not include this location. Presuming a small culvert is installed here (comparable in size to those on Kannah Creek), PMF flows will exceed the capacity of the culvert and water will back up behind the embankment. From the 1-m LiDAR data, we know that storage potential on Reservoir Creek upstream of the embankment is significant—about 113.6 ac-ft of water can be stored before the road overtops. This volume is comparable to the entire PMF excess volume originating from the watershed area upstream. However, the contributing watershed area upstream of the embankment is only about 3 percent of the total watershed area above Carson Lake. So, while the attenuation potential is high here, the overall impact of that attenuation on the PMF inflow hydrograph to Carson Lake is negligible. Therefore, further accounting of attenuation potential here is not needed.

Both areas of ponding upstream of the Highway 65 embankment are depicted in Figure 5.

Modeling Carson Reservoir and Dam

We modeled Carson Reservoir in HEC-HMS using information and physical data obtained from RJH (2019). The reservoir is represented with an elevation-storage curve and separate discharges resulting from flow over the spillway and flow over the dam top. We did not include releases from the reservoir's low-level outlet in the analysis. Pertinent information is shown in Table 8, and details of the model's configuration can be found in Appendix D.

Table 8. Pertinent information for Carson Dam and Reservoir

Dam Top Elevation (ft)	9,902
Dam Length (ft)	620
Normal Maximum Capacity (ac-ft)	520 ac-ft
Spillway Crest Elevation (ft)	9,895
Spillway Crest Length (ft)	142

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Figure 5. Ponding areas and total volume potential upstream of the Highway 65 embankment

Probable Maximum Precipitation

RJH (2019) analyzed PMPs for both long-duration general storms and shorter duration local storms. RJH developed PMP storms using both Hydrometeorological Report No. 49 (HMR 49) (Hansen et al. 1984) and the CO-NM REPS tool. Using both procedures, RJH found that the local storms, by far, were the controlling events for Carson Reservoir. Therefore, we only considered the local storm PMP here.

For local storms, RJH (2019) found that the PMF resulting from the REPS PMP was 36 percent greater than from the HMR 49 PMP with respect to peak inflow and 26 percent greater with respect to total inflow volume. The Colorado State Engineer's Office has fully adopted the use of the REPS tool for computing PMPs, superseding HMR 49. We obtained the REPS GIS application tools from https://drive.google.com/drive/folders/1BwS5VLnQ6_5N_NWjfgbLUPV42Ietex4n on September 3, 2019. Using the REPS PMP evaluation tool and a shapefile of the delineated watershed, we developed 2-hr local and 6-hr local PMPs for the contributing area above Carson Lake. REPS outputs the PMP as an actual storm with a synthetic temporal distribution applied at 5-min time intervals. Per Colorado Dam Safety guidance, we scaled up the PMP storm uniformly by a factor of 1.07 "to account for expected increases in temperature and associated increases in atmospheric moisture availability over the 50-year period 2020 to 2070" (DWR 2019b). The 2-hr local PMP storm pattern, both directly from REPS and scaled by 1.07, are provided in Table 9. We applied the scaled PMP storm to the HEC-HMS model as specified hyetographs for each subbasin. Because the total watershed area above Carson Dam is only 11 mi^2, we did not apply depth-area reduction factors to the PMP storm depths.

Table 9. 2-hr Local PMP Storm Pattern

Time Elapsed (min)	Incremental Rainfall Depth from REPS (in)	Incremental Rainfall Depth Increased by 7 Percent (in)
5	0.692	0.740
10	0.608	0.651
15	0.535	0.572
20	0.469	0.502
25	0.412	0.441
30	0.363	0.388
35	0.324	0.347
40	0.292	0.312
45	0.270	0.289
50	0.255	0.273
55	0.249	0.266
60	0.253	0.271
65	0.243	0.260
70	0.245	0.262
75	0.245	0.262
80	0.245	0.262
85	0.245	0.262
90	0.245	0.262
95	0.245	0.262

Time Elapsed (min)	Incremental Rainfall Depth from REPS (in)	Incremental Rainfall Depth Increased by 7 Percent (in)
100	0.245	0.262
105	0.245	0.262
110	0.245	0.262
115	0.245	0.262
120	0.245	0.262

Results and Sensitivity

PMF Simulation Results

We applied the configured 2-hr PMP storm to the HEC-HMS model to compute the PMF inflow. We used a 5-min computational time interval, and all reservoirs—both upstream and Carson—were initialized with starting pools at the spillway crest elevations. Key results are shown in Table 10 and hydrographs are plotted earlier in this document in Figure 1. The pool elevation rises to a maximum value of 9,905.14 ft, which overtops the dam crest elevation of 9,902 ft for approximately 3 hours.

Table 10. Summary of Carson Lake PMF results

Total Precipitation Depth (in)	8.20
Total Precipitation Excess (in)	7.05
Peak Inflow (cfs)	22,214
Total Inflow Volume (ac-ft)	4,703
Maximum Pool Elevation (ft)	9,905.14
Peak Outflow (cfs)	22,107

The peak inflow presented here (22,214 cfs) is significantly less than the 35,097 cfs documented in RJH (2019). Total storm volumes compare more closely, however, with RJH's inflow volume of 4,937 ac-ft only slightly exceeding the 4,703 ac-ft computed for this update. For both studies, dam overtopping occurs for about 2.5 to 3 hours.

We also ran the HEC-HMS model using the REPS 6-hr local storm PMP as input. However, we found that the 2-hr PMP controlled by a significant margin.

Sensitivity of Results

We analyzed the sensitivity of results to various changes in initial conditions and parameters. We varied dam failure assumptions, S-curves and basin lag times, and initial losses and constant loss rates to assess model sensitivity and to maximize PMF inflows. Results are summarized in Table 11. The baseline results (Scenario 0) shown in the table are from the 2-hr local PMF storm simulation described previously and adopted as final.

The table shows that model results are not highly sensitive to changes in initial losses and constant loss rates (see scenarios 3 and 4). Furthermore, changing the S-curves and basin lag times from Colorado Plateau to Rocky Mountain (Scenario 2) decreases the PMF inflow and maximum pool elevation. The model is highly sensitive to upstream dam failure assumptions (see Scenario 1 for no dam failure and scenarios 5 and 6 for failure at 0 ft of overtopping). Lowering basin lag times to the lowest reasonable value (computed using K_n = 0.04 for all subbasins) also greatly impacts the results (see Scenario 6). Scenario 6 assumptions, which are inherently conservative in all regards, are like those employed in RJH (2019), and the results for Scenario 6 are close in magnitude to those reported there. This updated hydrology study is intended to replace those highly conservative assumptions with ones more representative of expected watershed conditions, as suggested in DWR (2019a).

Table 11. Sensitivity Analysis Results

Scenario	Description	Peak Inflow (cfs)	Total Inflow Volume (ac-ft)	Maximum Pool Elevation (ft)
0	Baseline	22,214	4,703	9,905.14
1	Baseline with no upstream dam failures	17,475	4,116	9,904.30
2	Baseline with Rocky Mountain S-curves and lag times	19,747	4,705	9,904.64
3	Baseline with constant loss rates computed considering top 18 inches of soil	22,532	4,811	9,905.19
4	Baseline with constant loss rates computed considering top 18 inches of soil and initial infiltration set to 0 inches	23,815	4,981	9,905.37
5	Baseline with constant loss rates computed considering top 18 inches of soil, initial infiltration set to 0 inches, and dam failure triggered at 0-ft overtopping.	27,860	5,139	9,905.94
6	Baseline with constant loss rates computed considering top 18 inches of soil, initial infiltration set to 0 inches, dam failure triggered at 0-ft overtopping, and basin lag times set to lowest reasonable value (K_n = 0.04).	33,556	5,147	9,906.75

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Appendix A
8-Point Cross Sections

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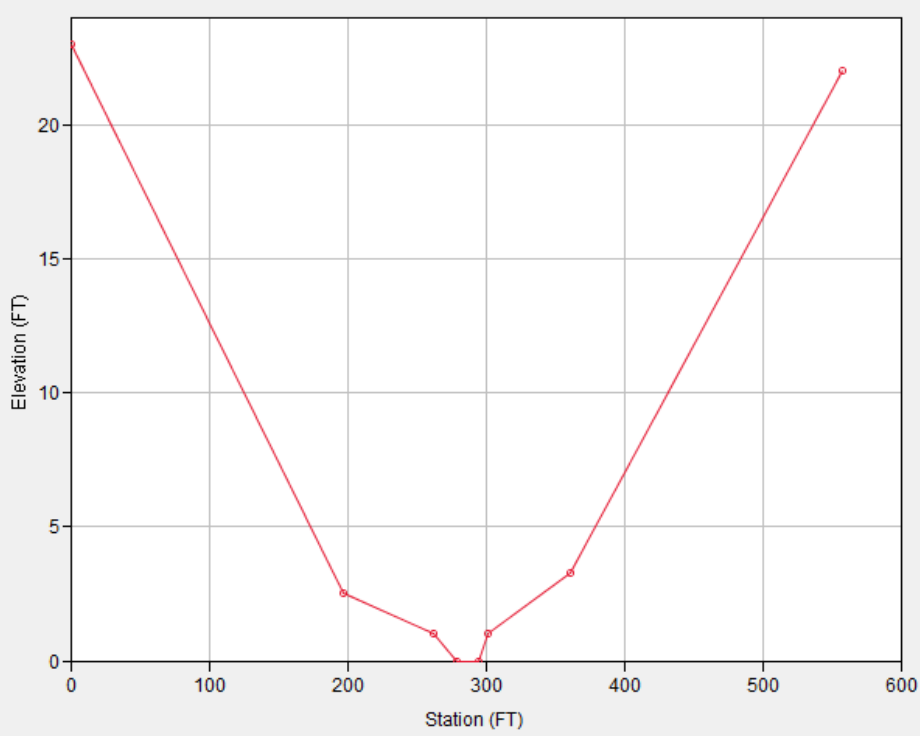
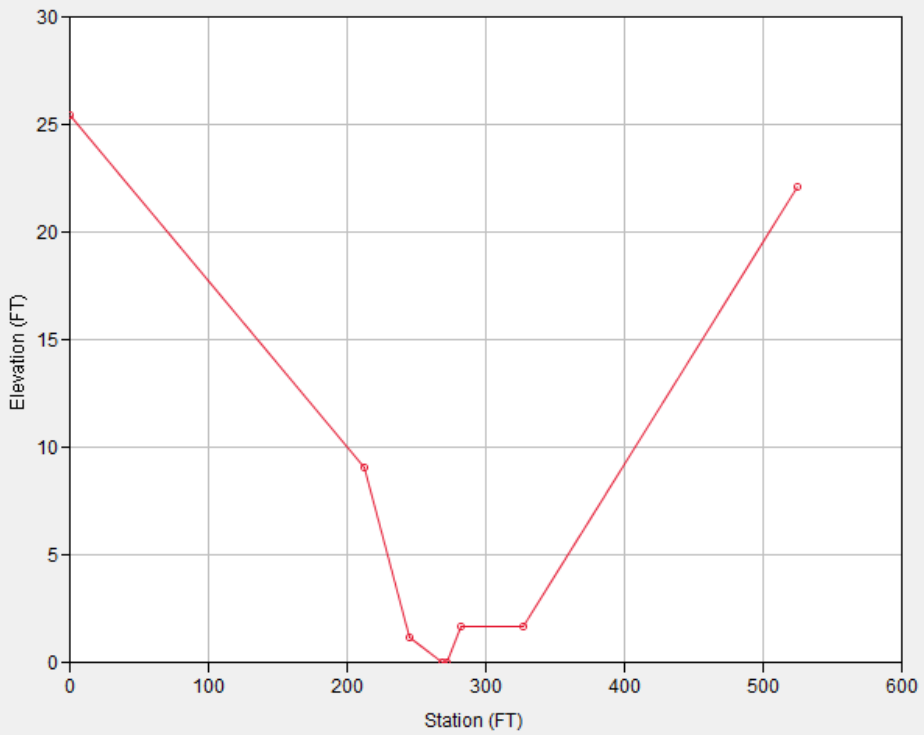
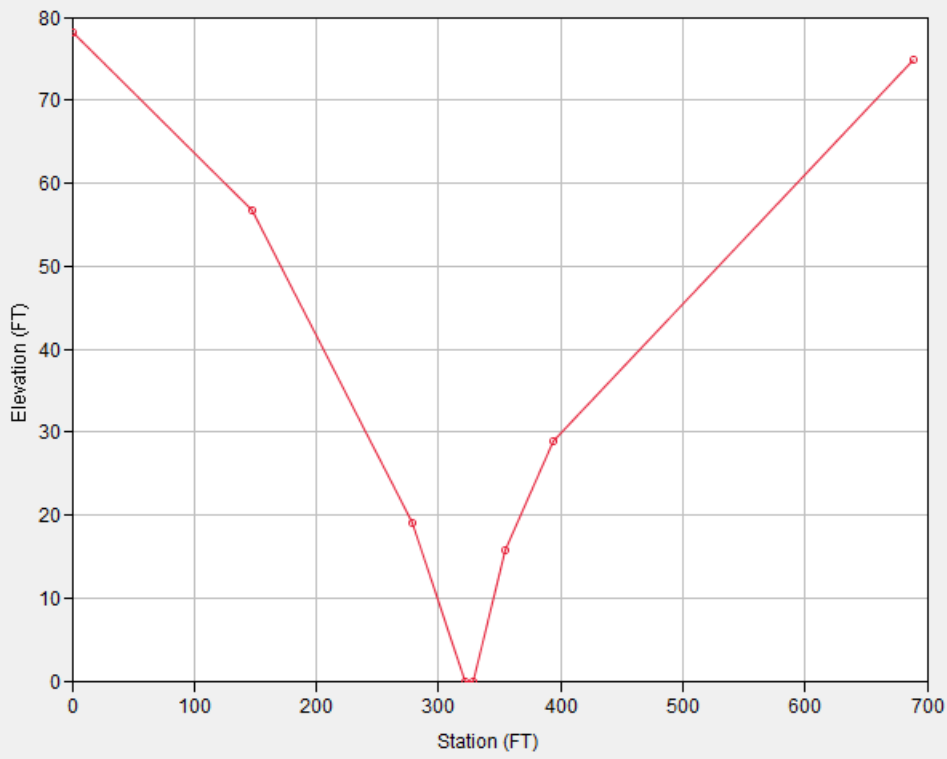
Reach: R_Blw_Mesa6



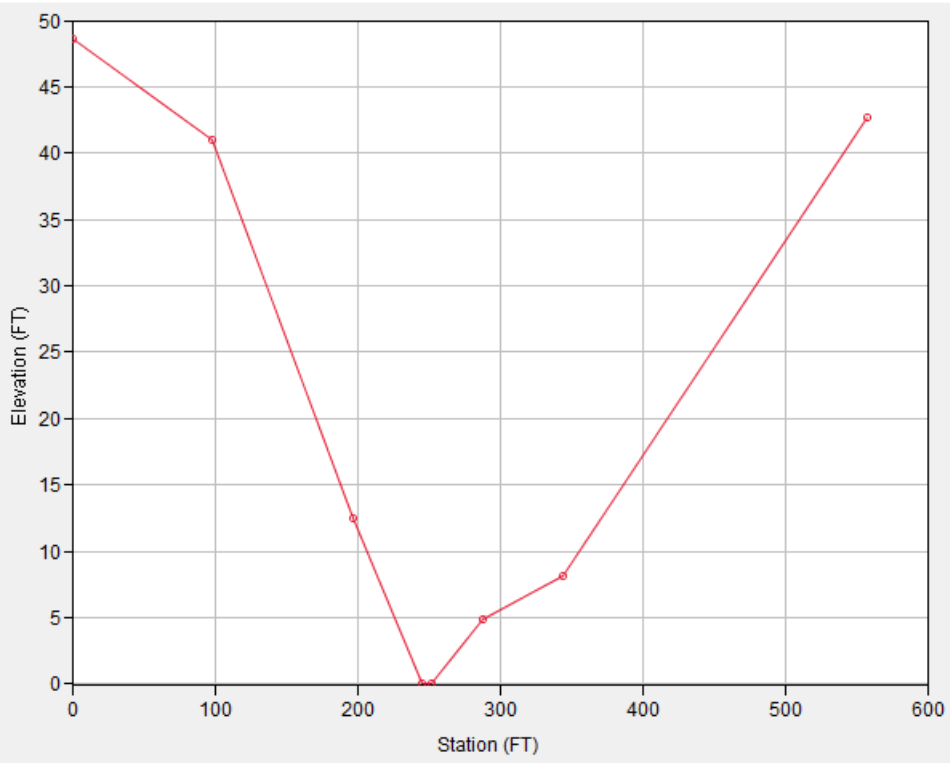
Reach: R_Blw_Mesa9



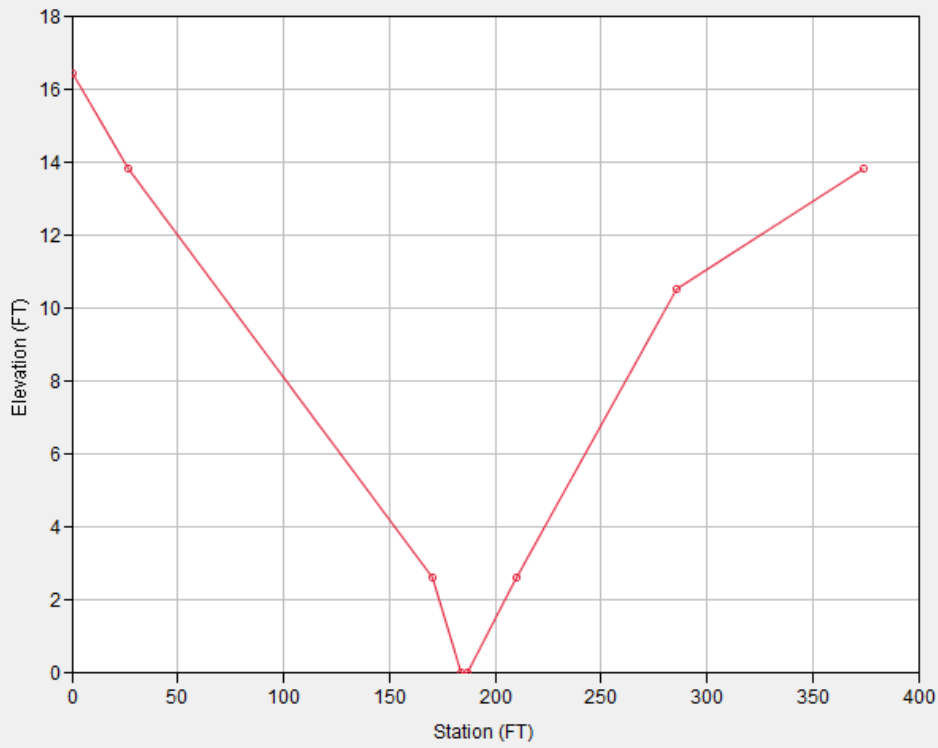
Reach: R_JL3_to_JL2



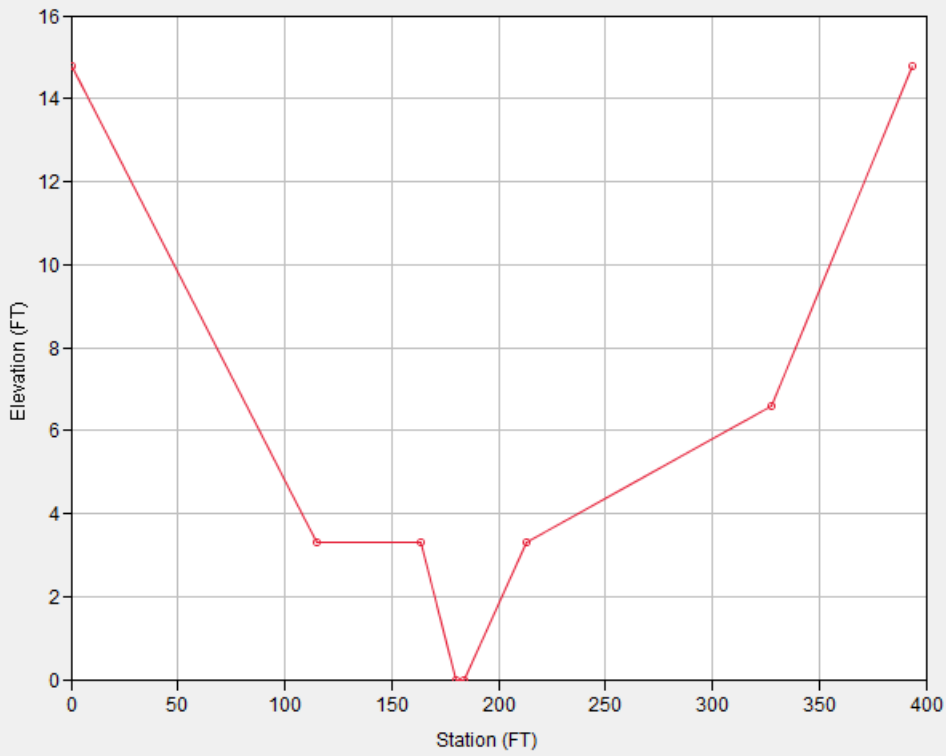
Reach: R_JL4_to_JL3



Reach: R_JL7_to_JL2



Reach: R_Mesa8_to_HW65



Appendix B

Upstream Reservoir Information

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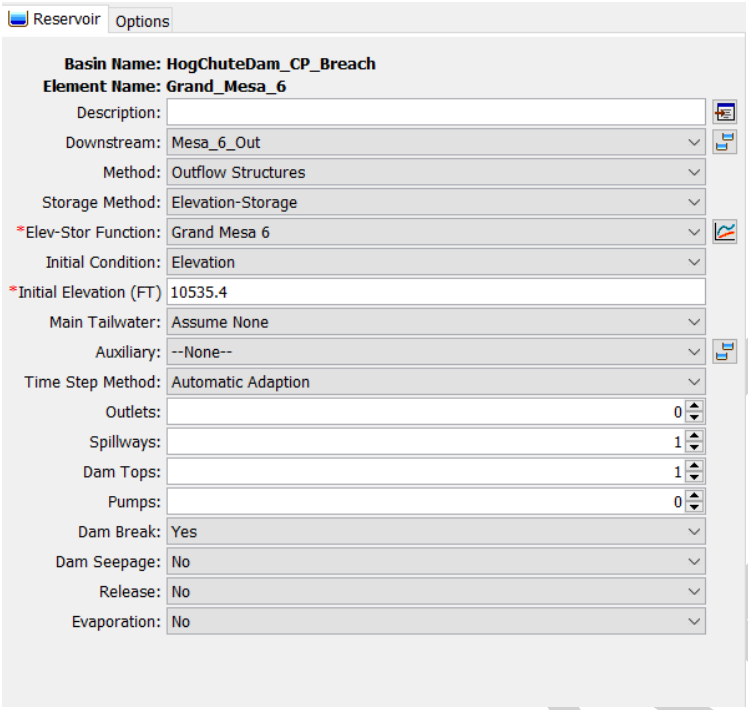
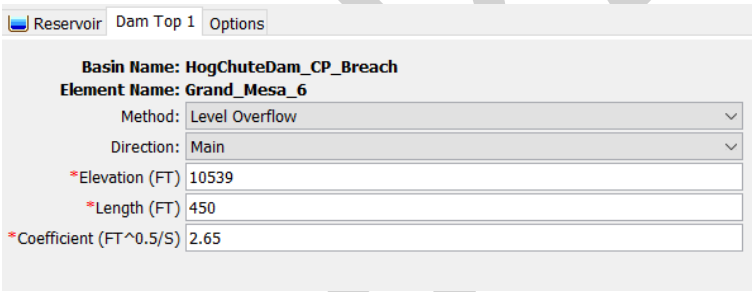
Grand Mesa #6

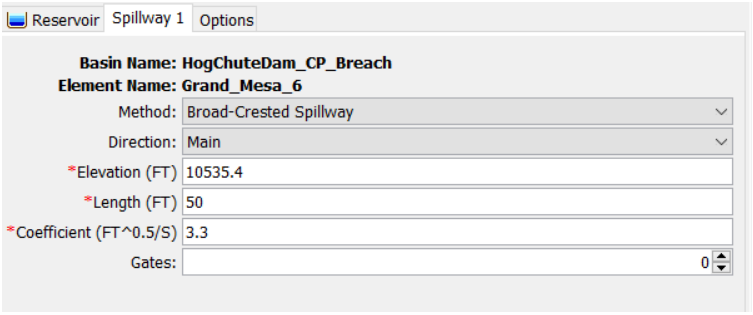
Reservoir: Source 1-m LiDAR data



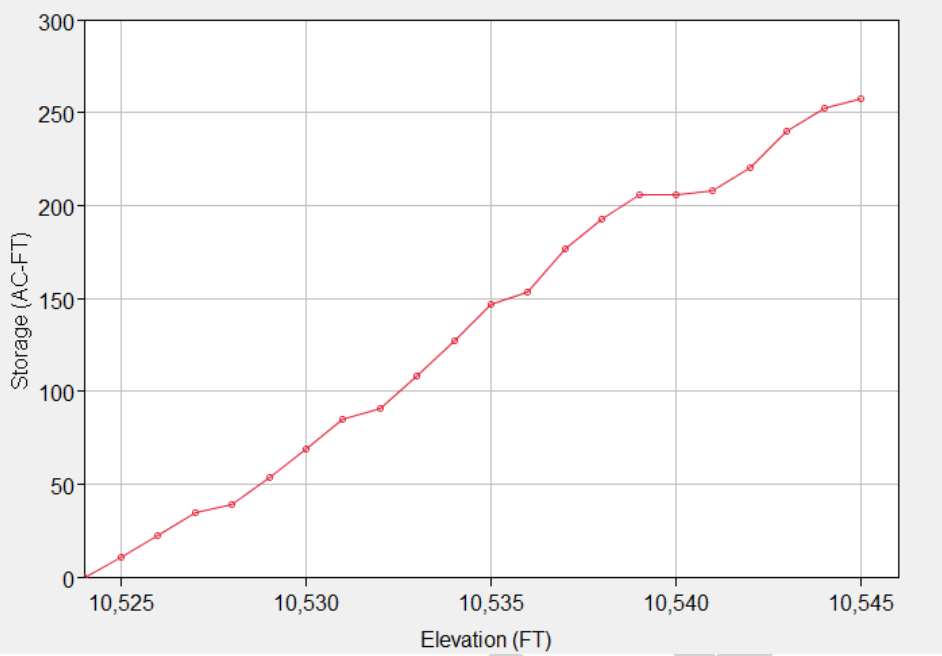
Dam top: Source 1-m LiDAR data



Spillway: Source 1-m LiDAR data



Elevation-storage curve: Source 1-m LiDAR data



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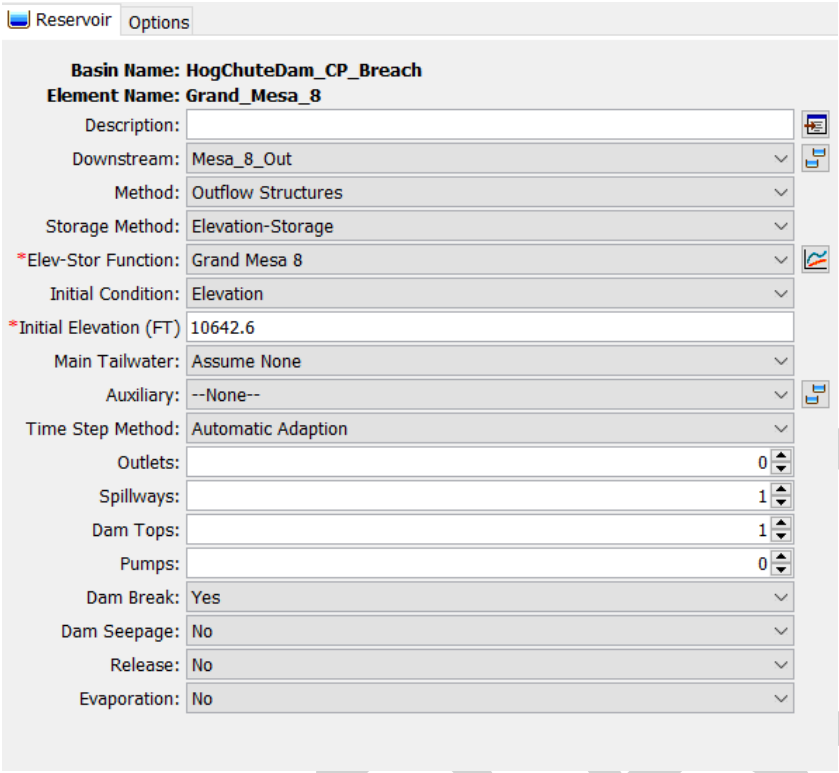
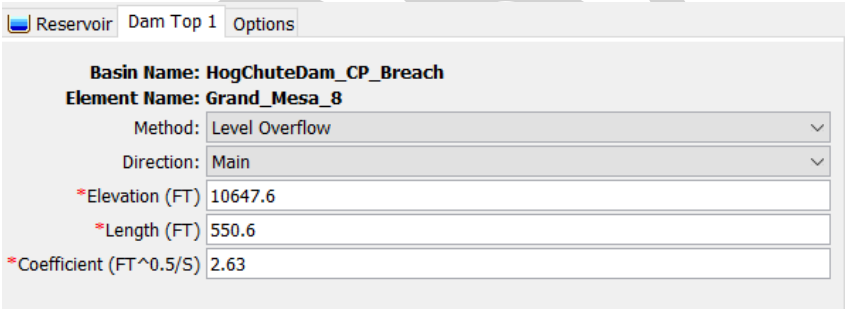
Grand Mesa #8

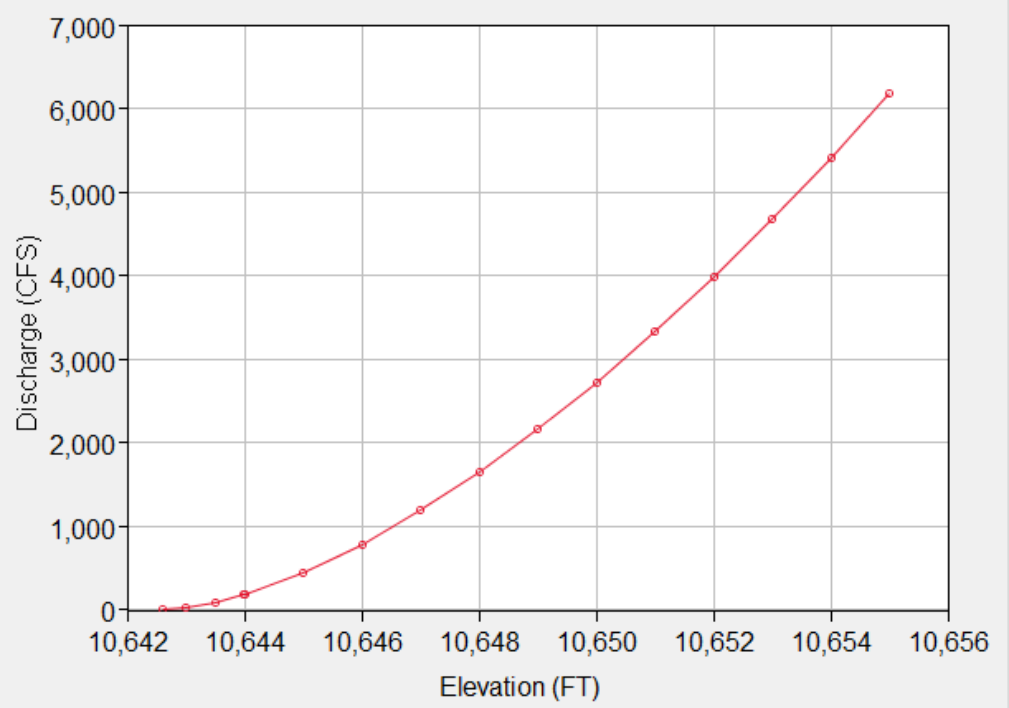
Reservoir: Source RJH (2019)



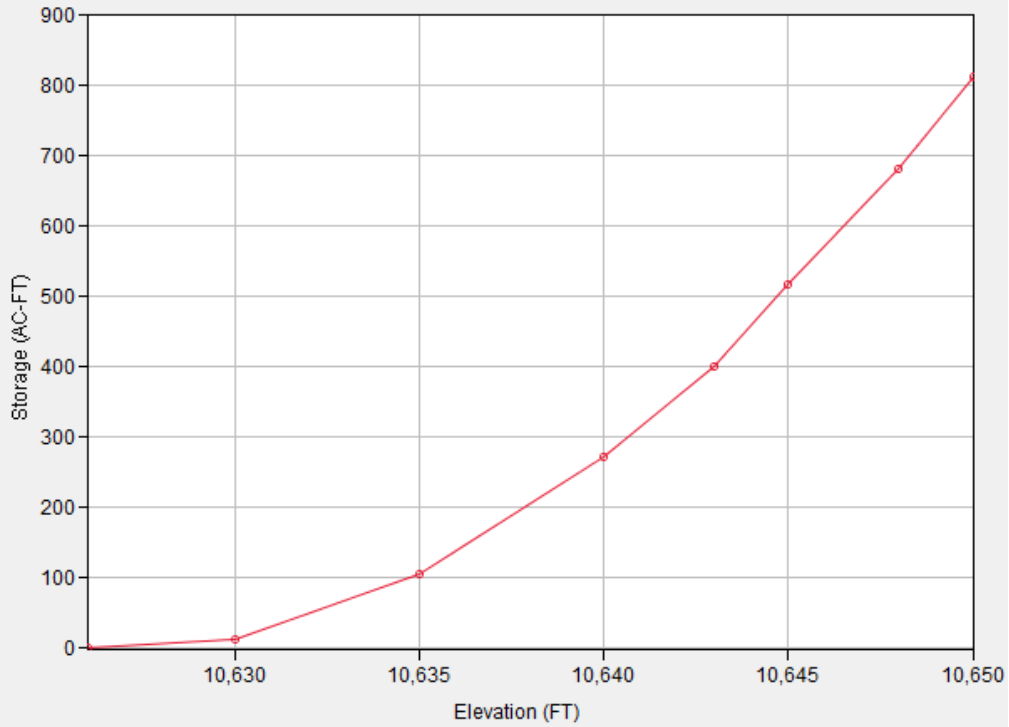
Dam top: Source RJH (2019)



Spillway rating curve: Source RJH (2019)

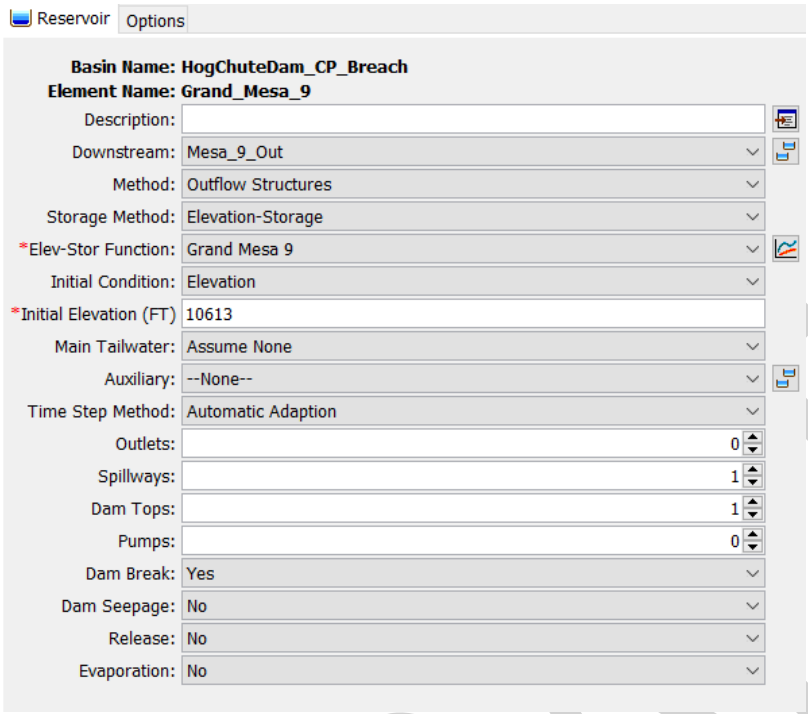


Elevation-storage curve: Source RJH (2019)

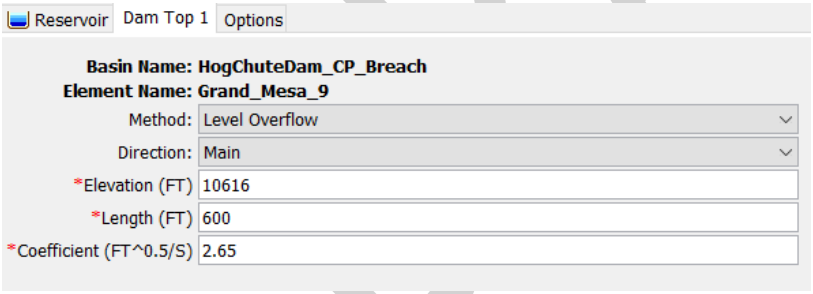


Grand Mesa #9

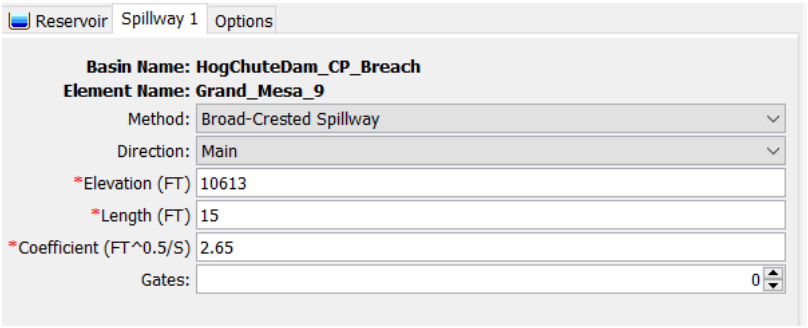
Reservoir: Source RJH (2019)



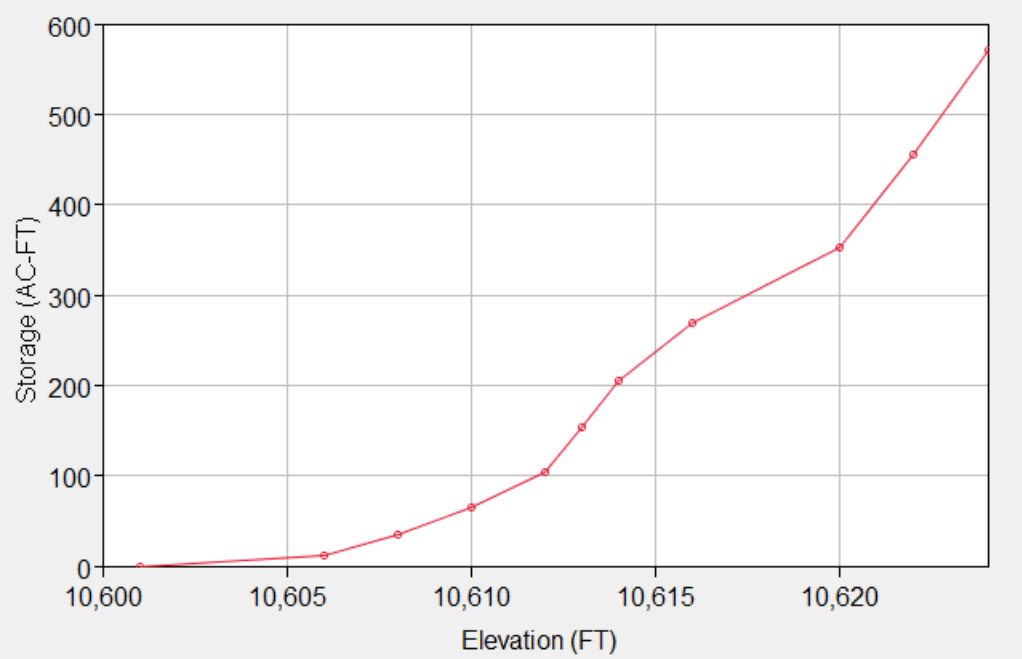
Dam top: Source RJH (2019)



Spillway: Source RJH (2019)



Elevation-storage curve: Source RJH (2019)

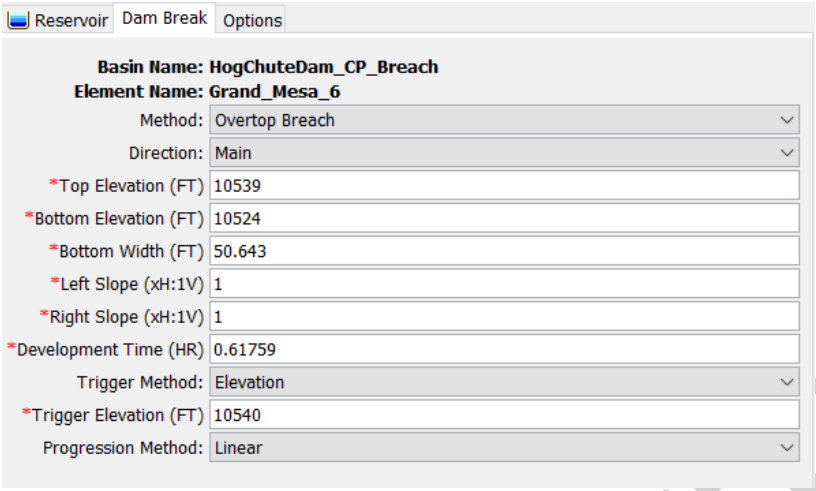


DRAFT

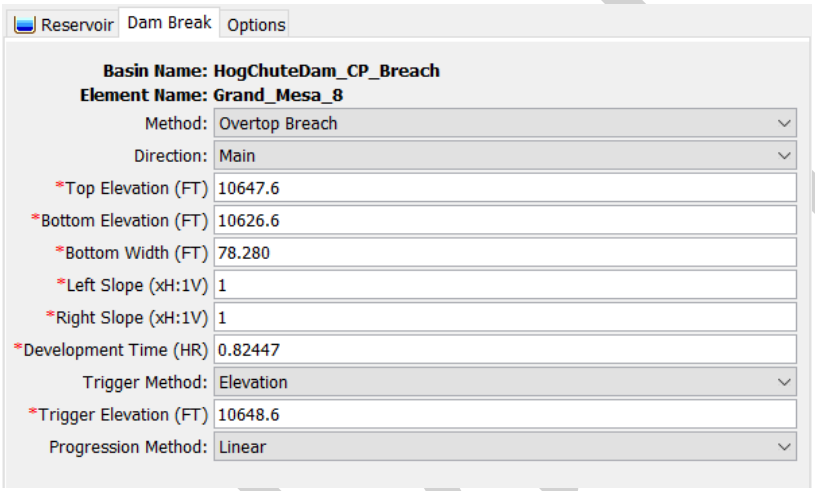
Appendix C
Dam Breach Configuration

DRAFT

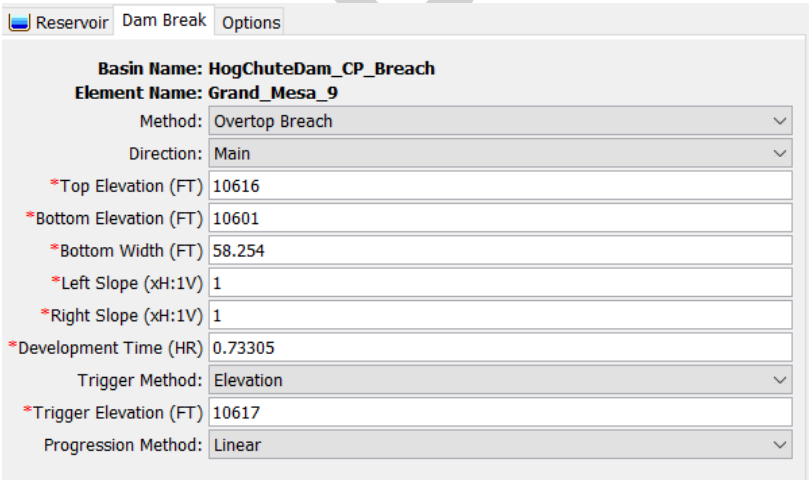
Grand Mesa #6



Grand Mesa #8



Grand Mesa #9

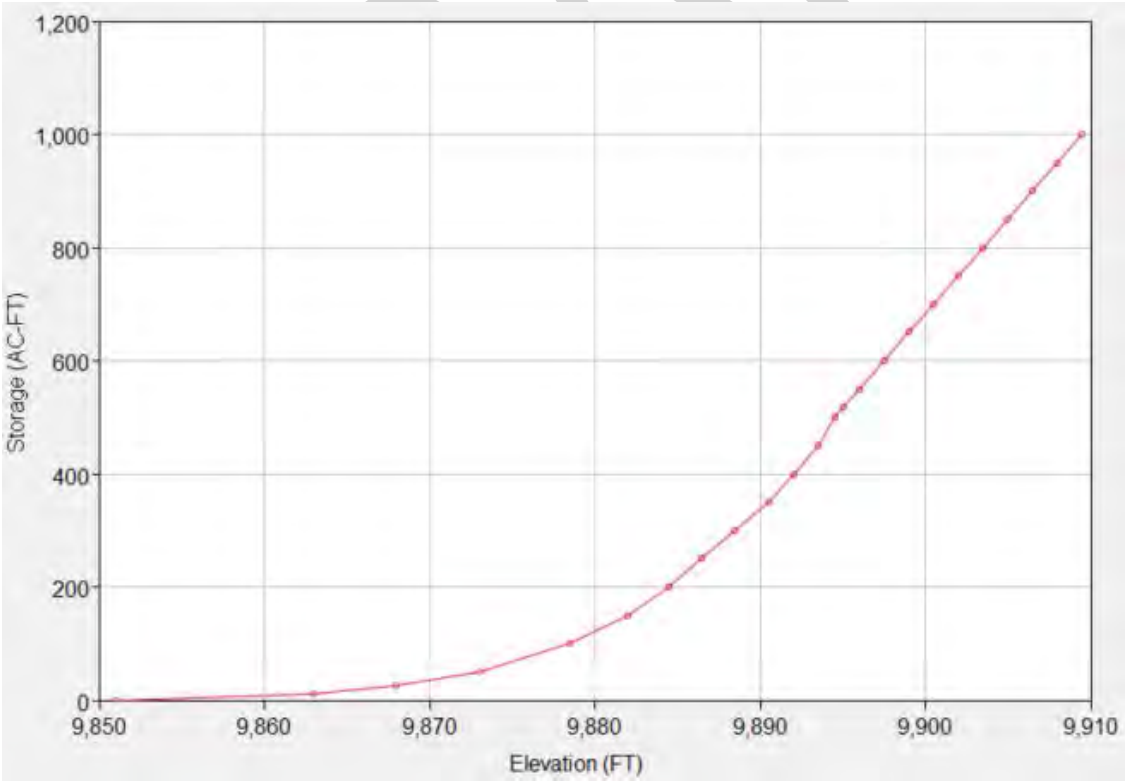


Appendix D
Carson Dam and Reservoir Configuration

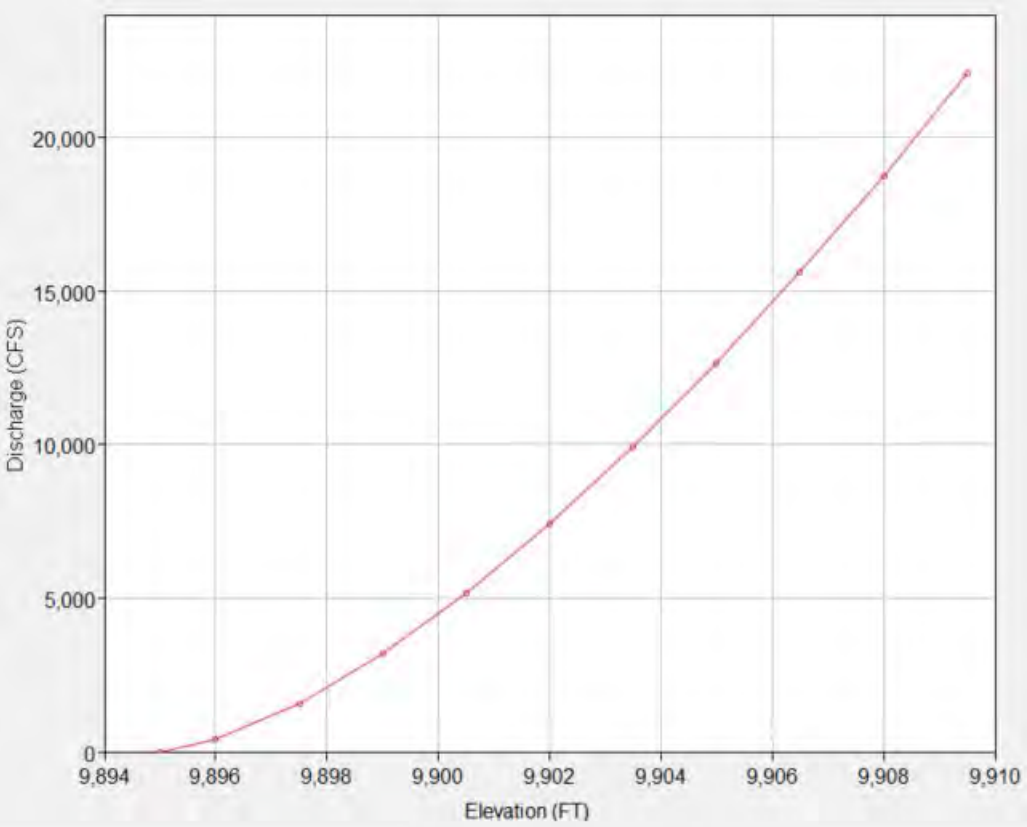
DRAFT

Reservoir: Source RJH (2019)

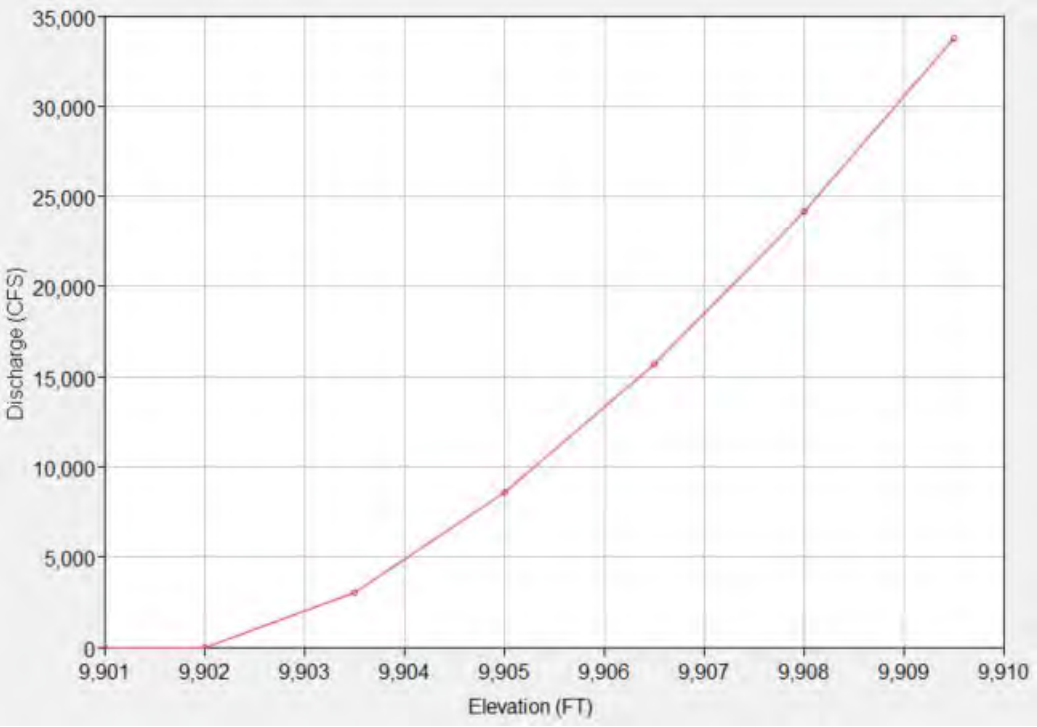
Elevation-storage curve: Source RJH (2019)



Spillway rating curve: Source RJH (2019)



Dam overtopping rating curve: Source RJH (2019)



Appendix C
Geotechnical Reports

Huddleston-Berry
Engineering & Testing, LLC

2789 Riverside Parkway
Grand Junction, Colorado 81501
Phone: 970-255-8005
Info@huddlestonberry.com

November 4, 2019
Project#01806-0002

Ayres Associates
3433 Oakwood Hills Parkway
Eau Claire, Wisconsin 54701

Attention: Mr. Chris Goodwin

Subject: Geotechnical Investigation
Carson Lake
Mesa County, Colorado

Dear Mr. Goodwin,

This letter presents the results of a geotechnical investigation conducted by Huddleston-Berry Engineering & Testing, LLC (HBET) at Carson Lake on the Grand Mesa in Mesa County, Colorado. The purpose of the investigation was to evaluate the subsurface conditions along the spillway chute to assist in designing spillway improvements for the reservoir.

Site Conditions

At the time of the investigation, the site was generally open. The existing spillway channel was fairly heavily vegetated; however, the area beyond the channel to the south and east was vegetated with weeds, grasses, and scattered low brush. The spillway channel was dry and the bottom of the channel was heavily armored with cobbles and boulders. Boulders were also observed to scattered on the ground surface across the site.

Subsurface Investigation

The subsurface investigation included three test pits as shown on Figure 2 – Site Plan. The test pits were excavated to depths of between 5.5 and 9.0 feet below the existing ground surface. Typed test pit logs are included in Appendix A.

As indicated on the logs, the subsurface conditions were slightly variable. However, the test pits generally encountered 1.5 to 2.0 feet of organic rich topsoil materials above brown, moist, dense to very dense clayey sand with gravel to clayey gravel with sand soils to the bottoms of the excavations. The sand and gravel soils contained varying quantities of cobbles and boulders. However, both TP-2 and TP-3 contained a higher percentage of cobbles and boulders than TP-1. This can be seen in test pit photographs included in Appendix B. Groundwater was not encountered in the test pits at the time of the investigation.

Laboratory Testing

Due to the presence of significant quantities of larger particles, representative samples of the overall soil mass were unable to be collected. However, HBET conducted laboratory testing on samples of the matrix soils. Laboratory testing included grain size analysis, natural moisture content determination, and Atterberg limits determination. The laboratory testing results are included in Appendix C.

The laboratory testing results indicate that native matrix soils are slightly to moderately plastic. The natural moisture content of the soils was very low at 9%.

Spillway Structure Recommendations

In general, the native cobble and boulder materials are in a dense to very dense condition and will provide excellent support for any new spillway structure. As a result, HBET recommends that any new structure bear directly on the native soils. However, where numerous boulders are present in the subgrade, it may be preferable to pour a concrete leveling pad to interlock with the boulders and provide a uniform bearing surface for the base of the structure.

Although the native soils will provide excellent structural support, as observed along the existing channel bottom, the fine grained portion of the soils will tend to wash out. The clays in the matrix have some plasticity; however, the quantity of clay is insufficient to limit hydraulic conductivity and prevent migration of the fines. As a result, any new spillway structure should include measures to limit/prevent seepage below the structure.

Prior to placement of concrete for any new structure, it is recommended that the bottom of the foundation excavation be moisture conditioned and proofrolled to the Engineer's satisfaction. Large particles may need to be removed at the direction of the Geotechnical and/or Structural Engineer.

For construction above properly prepared dense native soils, a maximum allowable bearing capacity of 3,000 psf may be used. In addition, a modulus of subgrade reaction of 250 pci may be used for the dense native soils. In general, for construction in accordance with the above recommendations, HBET anticipates that post-construction differential settlements will be less than 0.5-inch and total settlements will be less than 1.0-inch.

Engineering Properties

As discussed previously, due to the presence of large particles, representative samples of the overall native soil mass were unable to be collected. However, the samples collected still contained a significant quantity of gravels. Considering the grain-size limitations for the test, direct-shear testing on samples from the site would be even less representative of the natural soil mass. As a result, HBET developed engineering parameters for the overall soil mass based upon our observations of the in-situ material and upon our geotechnical engineering experience. The following conservative parameters are recommended for the native soils:

- $\gamma = 125$ pcf
- $\phi' = 32^{o}$
- $c' = 0$ psf

General Notes

The recommendations included above are based upon the results of the subsurface investigation and upon our experience. These conclusions and recommendations are valid only for the subject project.

As discussed previously, the subsurface conditions at the site were slightly variable. However, the precise nature and extent of subsurface variability may not become evident until construction. Therefore, HBET should be retained to provide engineering oversight during ***ALL*** phases of the construction to verify the subsurface conditions and/or provide alternative recommendations in the event that significant subsurface variability is encountered during construction.

We are pleased to be of service to your project. Please contact us if you have any questions or comments regarding the contents of this report.

Respectfully Submitted:
Huddleston-Berry Engineering and Testing, LLC

Michael A. Berry, P.E.
Vice President of Engineering

FIGURES

FIGURE 1
Site Plan

APPENDIX A
Typed Boring Logs

Huddleston-Berry Engineering & Testing, LLC
640 White Avenue, Unit B
Grand Junction, CO 81501
970-255-8005
970-255-6818

TEST PIT NUMBER TP-1

CLIENT Ayres Associates
PROJECT NAME Carson Lake
PROJECT NUMBER 01806-0002
PROJECT LOCATION Grand Mesa, CO
DATE STARTED 9/30/19 **COMPLETED** 9/30/19
GROUND ELEVATION _____ **TEST PIT SIZE** _____
EXCAVATION CONTRACTOR Client
GROUND WATER LEVELS:
EXCAVATION METHOD Backhoe
AT TIME OF EXCAVATION dry
LOGGED BY MAB **CHECKED BY** MAB
AT END OF EXCAVATION dry
NOTES _____
AFTER EXCAVATION ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		Clayey SAND with Gravel and Organics (TOPSOIL)										
2.5		Clayey SAND with Gravel (sc) to Clayey GRAVEL with Sand (gc), some cobbles, trace boulders, brown, moist, dense to very dense	GB 1					9	30	19	11	19
5.0												
7.5		Bottom of test pit at 9.0 feet.										

GEOTECH BH COLUMNS 01806-0002 CARSON LAKE.GPJ GINT US LAB.GDT 10/16/19

Huddleston-Berry Engineering & Testing, LLC
640 White Avenue, Unit B
Grand Junction, CO 81501
970-255-8005
970-255-6818

TEST PIT NUMBER TP-2

CLIENT Ayres Associates
PROJECT NAME Carson Lake
PROJECT NUMBER 01806-0002
PROJECT LOCATION Grand Mesa, CO
DATE STARTED 9/30/19 **COMPLETED** 9/30/19
GROUND ELEVATION _____ **TEST PIT SIZE** _____
EXCAVATION CONTRACTOR Client
GROUND WATER LEVELS:
EXCAVATION METHOD Backhoe
AT TIME OF EXCAVATION dry
LOGGED BY MAB **CHECKED BY** MAB
AT END OF EXCAVATION dry
NOTES _____
AFTER EXCAVATION ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		Clayey SAND with Gravel and Organics (TOPSOIL)										
2.5		Clayey SAND with Gravel (sc) to Clayey GRAVEL with Sand (gc), abundant cobbles, trace boulders, brown, dry to moist, dense to very dense										
		Higher percentage of cobbles and boulders than TP-1	GB 1					9	29	21	8	29
5.0		Bottom of test pit at 7.0 feet.										

Huddleston-Berry Engineering & Testing, LLC
640 White Avenue, Unit B
Grand Junction, CO 81501
970-255-8005
970-255-6818

TEST PIT NUMBER TP-3

CLIENT Ayres Associates
PROJECT NAME Carson Lake
PROJECT NUMBER 01806-0002
PROJECT LOCATION Grand Mesa, CO
DATE STARTED 9/30/19 **COMPLETED** 9/30/19
GROUND ELEVATION _____ **TEST PIT SIZE** _____
EXCAVATION CONTRACTOR Client
GROUND WATER LEVELS:
EXCAVATION METHOD Backhoe
AT TIME OF EXCAVATION dry
LOGGED BY MAB **CHECKED BY** MAB
AT END OF EXCAVATION dry
NOTES _____
AFTER EXCAVATION ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		Clayey SAND with Gravel and Organics (TOPSOIL)										
		Clayey SAND with Gravel (sc) to Clayey GRAVEL with Sand (gc), abundant cobbles, trace boulders, brown, dry to moist, dense to very dense										
2.5		***Higher percentage of cobbles and boulders than TP-1***										
5.0		Bottom of test pit at 5.5 feet.										

GEOTECH BH COLUMNS 01806-0002 CARSON LAKE.GPJ GINT US LAB.GDT 10/16/19

APPENDIX B
Test Pit Photographs

Profile of TP-1

Spoils pile from TP-1

Profile of TP-2

Spoils from TP-2

Profile of TP-3

Spoils from TP-3

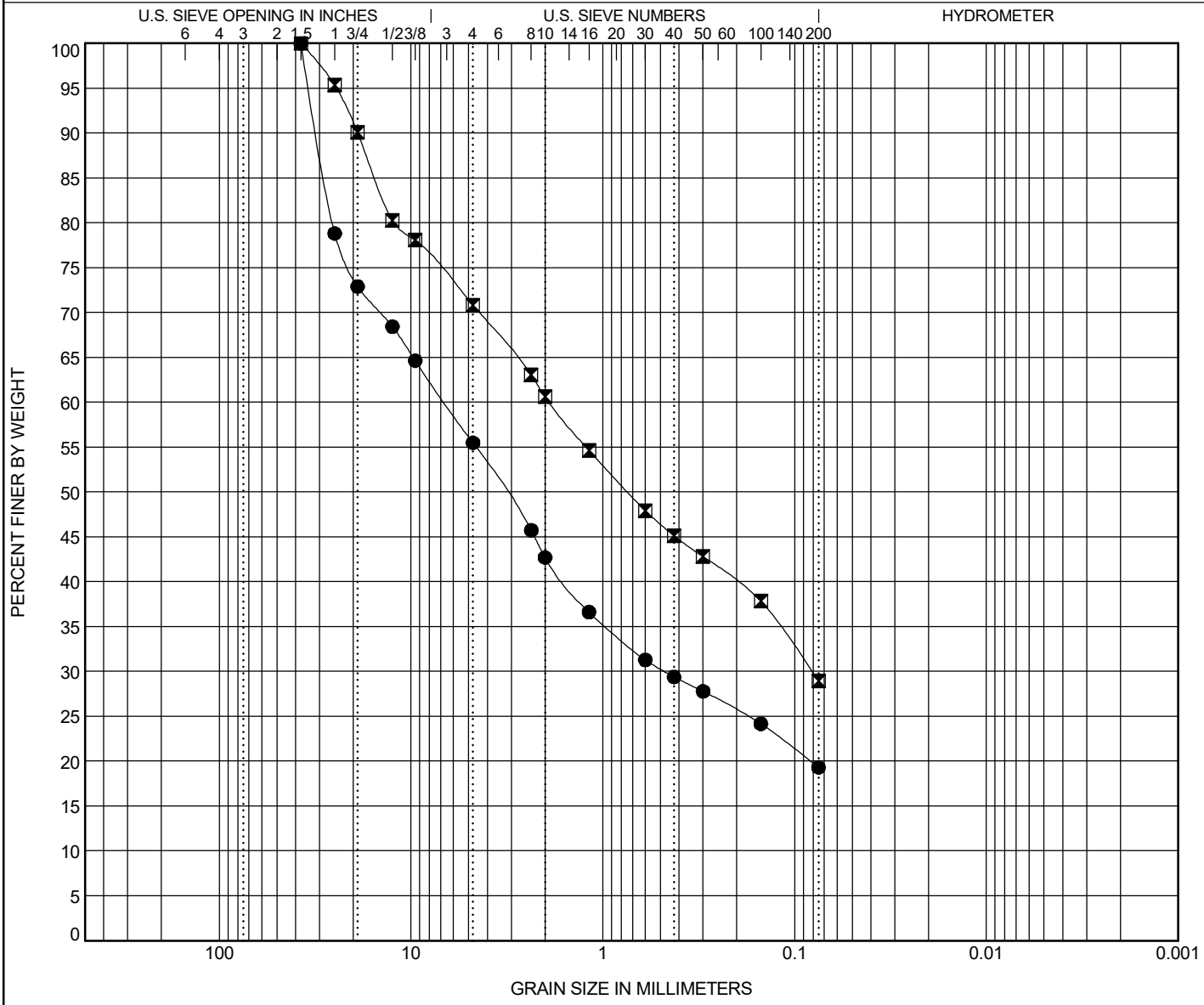
APPENDIX C
Laboratory Testing Results

Huddleston-Berry Engineering & Testing, LLC
640 White Avenue, Unit B
Grand Junction, CO 81501
970-255-8005
970-255-6818

GRAIN SIZE DISTRIBUTION

CLIENT Ayres Associates **PROJECT NAME** Carson Lake

PROJECT NUMBER 01806-0002 **PROJECT LOCATION** Grand Mesa, CO



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	**TP-1, GB1 9/19**	**CLAYEY GRAVEL with SAND(GC)**	**30**	**19**	**11**		
⊠	**TP-2, GB1 9/19**	**CLAYEY SAND with GRAVEL(SC)**	**29**	**21**	**8**		

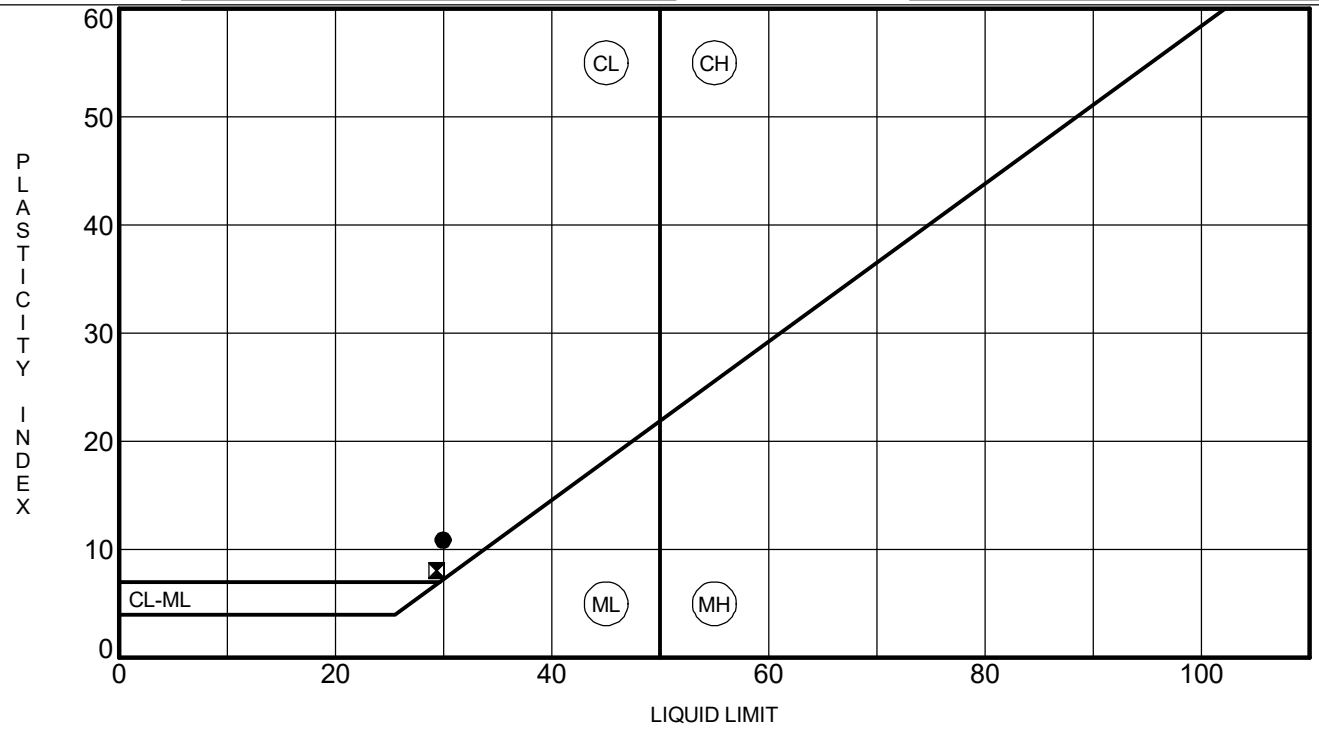
	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-1, GB1 9/19**	**37.5**	**6.687**	**0.475**		**44.5**	**36.2**	**19.3**	
⊠	**TP-2, GB1 9/19**	**37.5**	**1.895**	**0.082**		**29.2**	**41.9**	**28.9**	

Huddleston-Berry Engineering & Testing, LLC
640 White Avenue, Unit B
Grand Junction, CO 81501
970-255-8005
970-255-6818

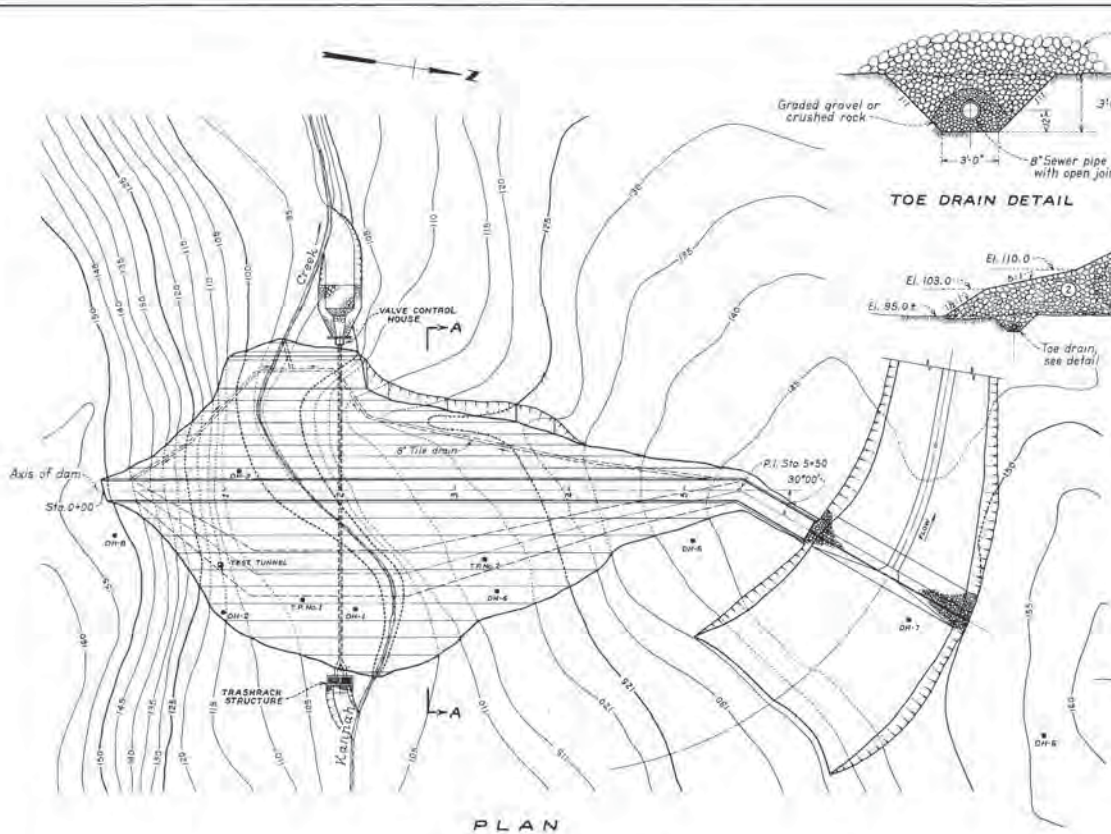
ATTERBERG LIMITS' RESULTS

CLIENT Ayres Associates **PROJECT NAME** Carson Lake

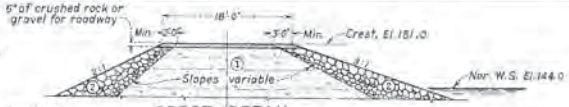
PROJECT NUMBER 01806-0002 **PROJECT LOCATION** Grand Mesa, CO



	Specimen Identification		LL	PL	PI	#200	Classification
●	**TP-1, GB1**	**9/30/2019**	**30**	**19**	**11**	**19**	**CLAYEY GRAVEL with SAND(GC)**
⊠	**TP-2, GB1**	**9/30/2019**	**29**	**21**	**8**	**29**	**CLAYEY SAND with GRAVEL(SC)**

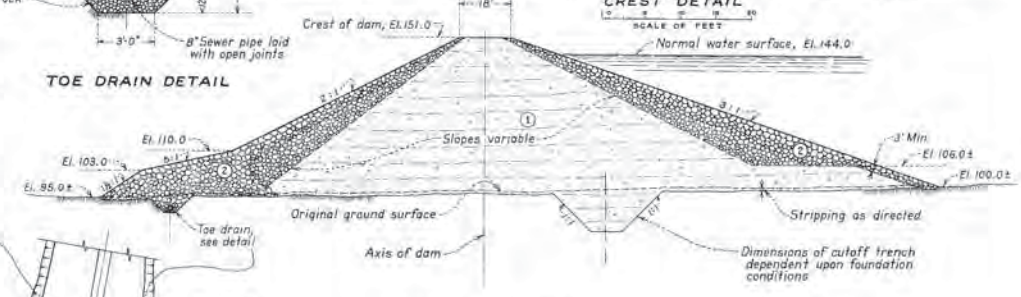


TOE DRAIN DETAIL



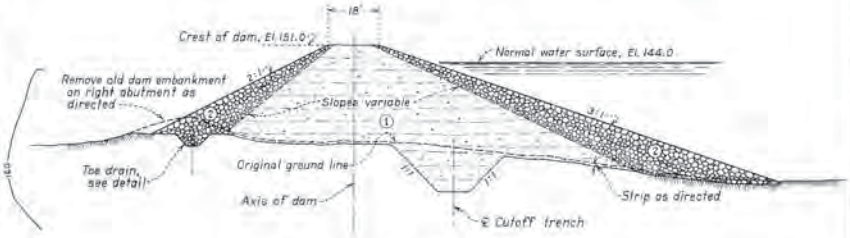
CREST DETAIL

SCALE OF FEET



MAXIMUM SECTION

SCALE OF FEET



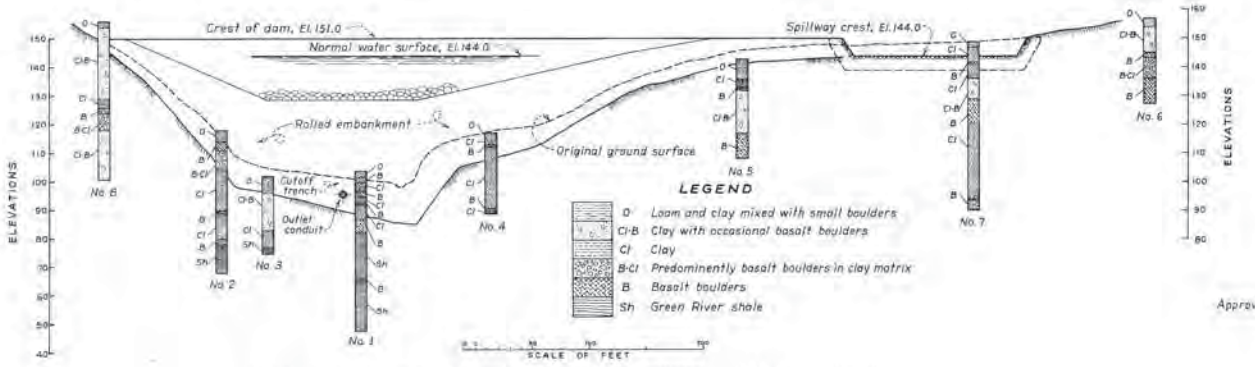
SECTION A-A

EMBANKMENT NOTES

① Impervious fill of clay or clay, sand and gravel graded with coarser material on outer slopes and compacted in 6" layers.
② Cobble and rock fill graded with coarser material on outer slopes.

NOTES

All elevations refer to an assumed datum
Final location of outlet works and appurtenant structures are dependent on foundation conditions encountered.



LEGEND

O	Loam and clay mixed with small boulders
Cl-B	Clay with occasional basalt boulders
Cl	Clay
B-Cl	Predominently basalt boulders in clay matrix
B	Basalt boulders
Sh	Green River shale

PROFILE ON ℄ OF CUTOFF TRENCH AND SPILLWAY - LOG OF DRILL HOLES

Approved on the 24 day of May 1947
By M. C. Hinderlider STATE ENGINEER
By DEPUTY

CITY OF GRAND JUNCTION
R.J. TIPTON AND ASSOCIATES INC., ENGINEERS
DENVER, COLORADO

IMPROVEMENTS TO WATER SUPPLY SYSTEM
HOGCHUTE RESERVOIR
PLAN, PROFILE AND SECTIONS

DRAWN: W.L.C. APPROVED:
TRACED: L.W.M. APPROVED:
CHECKED: N.A.E. APPROVED:

DENVER, COLORADO MAY 1947 | 705-31

C-454

RJH CONSULTANTS, INC.

GEOTECHNICAL AND
WATER RESOURCES ENGINEERING

GEOTECHNICAL DATA REPORT

HOGCHUTE DAM (AKA CARSON LAKE)
DAM ID 420127
MESA COUNTY, COLORADO

Submitted to
City of Grand Junction
250 North 5th Street
Grand Junction, CO 81501

Submitted by
RJH Consultants, Inc.
9800 Mt. Pyramid Court, Suite 330
Englewood, Colorado 80112
303-225-4611
www.rjh-consultants.com

January 2019
Project 181155

COLORADO REGISTERED
GARRETT O. JACKSON
29302
PROFESSIONAL ENGINEER

Garrett O. Jackson, P.E.
Project Manager

TABLE OF CONTENTS

TABLE OF CONTENTS I

SECTION 1 - INTRODUCTION 1
1.1 OBJECTIVE AND PURPOSE 1
1.2 BACKGROUND 1
1.3 RJH SCOPE OF WORK 2
1.4 AUTHORIZATION 3
1.5 PROJECT PERSONNEL 3

SECTION 2 – SITE INVESTIGATION 4
2.1 GENERAL 4
2.2 SURVEYING 4
2.3 BORINGS 5
2.4 DAILY SITE REPORTS 7
2.5 LOGGING AND SAMPLING PROCEDURES 7
2.6 PERMEABILITY TESTING IN SOIL 8
2.7 MONITORING WELLS 9
2.7.1 Monitoring Well Installation 9
2.7.2 Monitoring Well Readings 10

SECTION 3 – LABORATORY TESTING 11

SECTION 4 – SITE AND SUBSURFACE CONDITIONS 14
4.1 GENERAL GEOLOGY 14
4.2 SITE GEOLOGY 14
4.3 SUBSURFACE CONDITIONS 15
4.3.1 General Subsurface Profile 15
4.3.2 Embankment Fill 15
4.3.3 Colluvium (Qc) 16
4.3.4 Groundwater 18

SECTION 5 - LIMITATIONS 20

SECTION 6 – REFERENCES 21

LIST OF TABLES

Table 2.1 Summary of Borings
Table 2.2 Soil Hydraulic Conductivity Test Results
Table 3.1 Summary of Index Laboratory Test Results
Table 3.2 Summary of Permeability and Strength Laboratory Test Results

LIST OF FIGURES

Figure 1.1 Site Vicinity Map
Figure 1.2 Dam Plan
Figure 2.1 Monitoring Well Installation – B-101(P)
Figure 2.2 Monitoring Well Installation – B-102A(P)
Figure 2.3 Monitoring Well Installation – B-103(P)
Figure 2.4 Monitoring Well Installation – B-104(P)
Figure 2.5 Monitoring Well Installation – B-105A(P)
Figure 2.6 Monitoring Well Installation – B-105B(P)
Figure 2.7 Field Investigation Monitoring Well Data
Figure 3.1 Colluvium Drained Strength
Figure 3.2 Colluvium Undrained Strength
Figure 4.1 Published Site Geology
Figure 4.2 Dam Profile
Figure 4.3 Dam Section A – South
Figure 4.4 Dam Section B – Middle
Figure 4.5 Dam Section C – North

APPENDICES

Appendix A Soil Descriptors
Appendix B Boring Logs
Appendix C Sample Photographs
Appendix D Daily Site Reports
Appendix E Permeability Results
Appendix F Field Investigation Water Level Data
Appendix G Laboratory Testing Results

SECTION 1 - INTRODUCTION

1.1 Objective and Purpose

The City of Grand Junction (City) retained RJH Consultants, Inc. (RJH) to provide engineering services for the Hogchute Dam Safety Evaluation Project (Project). The purpose of this Geotechnical Data Report (Report) is to present data collected by RJH to support engineering evaluations of potential dam safety issues that have been identified by the Colorado Office of the State Engineer (SEO) as part of a Comprehensive Dam Safety Evaluation (CDSE). The engineering evaluations are presented in a separate dam safety evaluation report.

1.2 Background

Hogchute Dam (DAMID 420127) is located in Mesa County, Colorado, approximately 22 miles east-southeast of Grand Junction (Site). The dam is a 56-foot-high earth structure that impounds Carson Lake on Kannah Creek at an elevation (El.) of about 9,900 feet in the Grand Mesa National Forest. The reservoir provides water storage for domestic use, irrigation, and fishing recreation. A Site vicinity map is shown on Figure 1.1.

Based on design records, the dam was constructed in 1947, with a low-permeability earthen core protected by upstream and downstream rock shells of gravels, cobbles, and boulders. The embankment was designed to have an 18-foot-wide crest, 3 horizontal to 1 vertical (H:V) upstream slope, and 2H:1V downstream slope. A plan of the dam is shown on Figure 1.2. The outlet works consists of two 20-inch welded steel pipes with hydraulic slide gates at the upstream toe of the dam. The 20-inch pipes converge within the dam into a single 30-inch conduit that discharges into a rock-lined basin at the downstream toe of the dam. There appears to also be a 12-inch outlet gate installed on a 12-inch pipe between the two 20-inch conduits, but the configuration and use of this gate are not clear. The unlined emergency spillway is located at the north (right) end of the dam.

In 1988, the City relocated the outlet control structure from the downstream toe to the crest of the dam. At about the same time, the City extended the 8-inch toe drain discharge pipe into the outlet discharge basin. The work to move the outlet controls and extend the toe drain discharge is described in a 1988 letter, which also includes some photographs of the toe drain work. There are no other construction records for the dam. The City has a four-sheet plan set, dated 1947, that appears to show the original design.

In 2015, the SEO changed the dam's hazard classification to high hazard, based on inundation mapping performed by the City to assess the impacts of a potential dam failure on downstream development that had occurred since construction of the dam. Several SEO dam safety inspection reports over the years have mentioned concerns for undocumented seepage (not collected and not monitored), the absence of any filtering of the embankment core material, apparently broken outlet gate air vents, and the deteriorated condition of the spillway.

In 2017, the SEO performed a CDSE to assess the overall safety of the dam and provide the City with guidance in planning needed dam improvements.

1.3 RJH Scope of Work

RJH performed the following for the data collection phase of the Project:

- Reviewed documents provided by City and SEO.
- Prepared a base topographic map of the Site based on survey data provided by the City.
- Prepared for fieldwork, which included preparing a Project-specific Health and Safety Plan (HASP), coordinating utility clearances, and developing a Drilling and Site Investigation Plan for SEO review and approval.
- Drilled, sampled, and logged seven borings. Six borings were completed as open-standpipe monitoring wells and one boring was backfilled with cement-bentonite grout.
- Surveyed the locations of RJH's borings and monitoring wells using a handheld Global Positioning System (GPS).
- Prepared Daily Site Reports to document field activities.
- Performed quality assurance review of collected samples and field logs by a senior engineer.
- Performed laboratory tests on representative samples from the borings.
- Prepared final boring logs based on the field logs, quality assurance review, and laboratory test results.
- Prepared this Report.

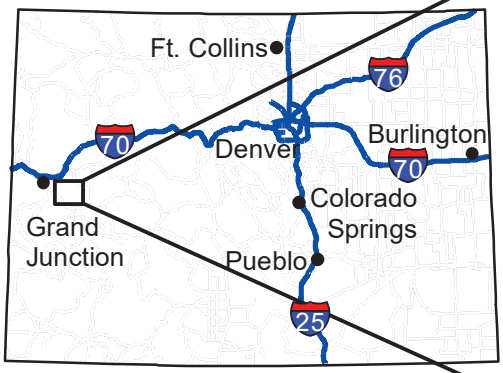
1.4 Authorization

This work was performed in general accordance with the terms and conditions of the Professional Services Contract RFP-4519-18-DH between the City and RJH, dated June 26, 2018 and a Contract Modification Request, dated September 12, 2018. Drilling was performed in general conformance with the Drilling and Site Investigation Plan approved by the SEO on July 16, 2018 and Addendum approved with contingencies by the SEO on August 28, 2018.

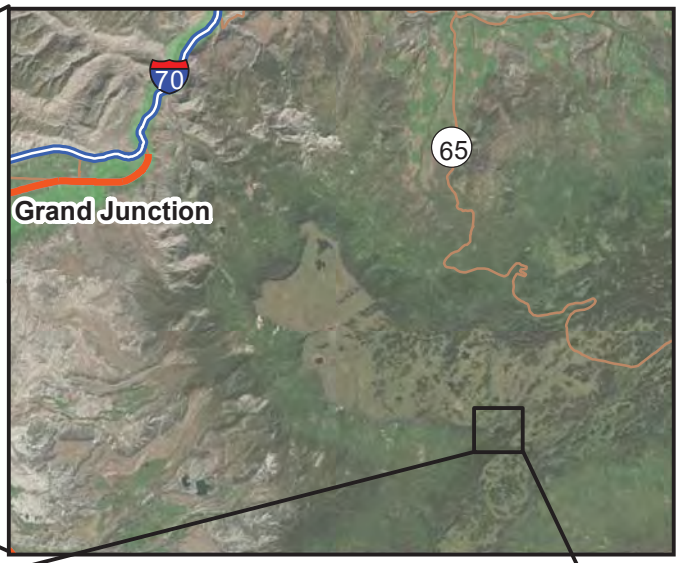
1.5 Project Personnel

The following personnel from RJH are responsible for the work contained in this Report:

Project Manager:	Garrett Jackson, P.E.
Staff Geological Engineer:	Jacquelyn Hagbery, E.I.
Technical Review:	Robert Huzjak, P.E.



PROJECT VICINITY MAP
(NOT TO SCALE)



PROJECT LOCATION MAP
(NOT TO SCALE)

SITE LOCATION MAP
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P:\18115 - Hogchute Dam Safety Evaluation\Engineering\GIS\Geotech\Geotech_Data_Report\Site Vicinity Map.mxd

CITY OF Grand Junction COLORADO	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	SITE VICINITY MAP
		PROJECT NO. 18115	January 2019 **Figure 1.1**

LEGEND

- ◉ MONITORING WELL
- ✥ BORING BACKFILLED WITH CEMENT–BENTONITE GROUT

NOTES:

1. AERIAL IMAGERY WAS OBTAINED FROM MICROSOFT BING, IMAGERY DATE UNKNOWN.
2. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
3. LOCATIONS OF MONITORING WELLS AND BORINGS WERE OBTAINED FROM HANDHELD GPS UNIT AND HAVE NOT BEEN SURVEYED.

REPRODUCE IN COLOR

0 50 100 200 300
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Grand Junction COLORADO

RJH CONSULTANTS, INC.

HOGCHUTE DAM
SAFETY EVALUATION
GEOTECHNICAL DATA REPORT

DAM PLAN

PROJECT NO. 18115

January 2019

Figure 1.2

P:\18115 – HOGCHUTE DAM SAFETY EVALUATION\CAD\FIGURES\MONITORING WELLS\MONITORING WELLS.DWG 12/21/2018 11:28 AM

SECTION 2 - SITE INVESTIGATION

2.1 General

The Site investigation was performed in two phases, from July 23 to July 28, 2018 and from September 17 to September 22, 2018. The Site investigation generally consisted of the following activities:

- Surveying with handheld GPS.
- Drilling, sampling, and logging borings.
- Preparing Daily Site Reports.
- Performing permeability tests in embankment fill and colluvium.
- Installing monitoring wells.
- Measuring groundwater levels.

2.2 Surveying

The City performed Project-specific topographic surveying in winter 2017. Topographic surveying of the dam and spillway was performed using conventional (i.e., field) survey equipment. RJH prepared a base topographic map for the Project based on the collected survey data. Based on the topographic data, the dam crest is at approximately El. 9902. The crest width of the embankment is between 14 and 18 feet, the upstream slope of the embankment is inclined at approximately 2.5H:1V to 3H:1V, and the downstream slope is inclined at approximately 2H:1V. Based on survey data and design documents, the maximum normal water level (NWL) is at approximately El. 9895. The horizontal coordinate system used for the Project is Mesa County Local Coordinate System Grand Mesa Area (GMA) with an offset because the surveyed area is beyond the limits of GMA. Therefore, the horizontal coordinates are spatially correct with respect to other points in the survey, but are not related to other global coordinate systems. The horizontal datum is NAD83 and vertical datum is NAVD88.

The borings were surveyed by RJH using a handheld GPS. The horizontal coordinates are in WGS84 and the datum is NAD83. The boring elevations were estimated based on the topographic data provided by the City. The City plans to survey the borings in spring 2019 when the Site can be accessed as weather and ground conditions allow.

2.3 Borings

Seven borings were drilled for the Project. The horizontal coordinates and ground surface elevations at the boring locations are provided in Table 2.1. The boring locations are shown on Figure 1.2. Boring logs are provided in Appendix B.

TABLE 2.1
SUMMARY OF BORINGS

Boring ID	Latitude[1] (deg)	Longitude[1] (deg)	Ground Surface Elevation[2] (ft)	Total Depth[3] of Boring (ft)	Boring Completion
Dam Crest					
B-101(P)	38.995296	-108.109759	9902.2	77.5	1.5-inch Monitoring Well
B-102A(P)	38.995677	-108.109800	9902.1	48.0	2-inch Monitoring Well
B-102B	38.995667	-108.109796	9902.1	5.0	Cement-Bentonite Grout
B-103(P)	38.996095	-108.109881	9901.8	30.0	2-inch Monitoring Well
Dam Downstream Toe					
B-104(P)	38.995285	-108.110292	9846.1	33.0	2-inch Monitoring Well
B-105A(P)	38.995632	-108.110160	9865.7	73.5	2-inch Monitoring Well
B-105B(P)	38.995681	-108.110127	9866.9	12.6	2-inch Monitoring Well

Notes:
1. Boring locations were surveyed by RJH with a handheld GPS. Horizontal coordinate system is WGS84 and datum is NAD83.
2. Elevation was estimated from the topographic survey data. Vertical datum is NAVD88.
3. Depth measured along boring axis. All borings were vertical.

RJH retained HRL Compliance Solutions, Inc. (HRL) of Grand Junction, Colorado to provide drilling equipment and services. Borings were drilled using a track-mounted CME 55LC drill rig with an automatic hammer.

Vertical borings in the dam crest were advanced from the ground surface using 7.75-inch outside-diameter (O.D.) (4.25-inch inside-diameter (I.D.)) hollow-stem augers. During auger advancement, sampling was generally performed at 2.5-foot intervals but sampling ranged from continuously to 8-foot intervals, depending on the presence of cobbles and boulders. Auger refusal was encountered in all dam crest borings and in our opinion was caused by cobbles and boulders in the subsurface. Based on the presence of cobbles and boulders in the crest borings, the drillers switched to a Symmetrix drive casing advancer, an air-hammer drilling method, for the dam downstream toe borings. The Symmetrix drive casing advancer was 5.375-inch O.D. (5.0-inch I.D.) and had continuous casing advancement. During casing advancement, sampling was generally performed at 2.5-foot

intervals but ranged from continuously to 5-foot intervals based on casing limitations at greater depths. The air compressor used for the Symmetrix drive casing advancer was set at the "low" pressure setting and air pressure measured at the air compressor ranged from 110 to 120 pounds per square inch (psi) during drilling.

The following sampler types were used during auger and Symmetrix drive casing advancer drilling:

- 1.375-inch I.D. (2.0-inch O.D.) standard split-spoon sampler (ASTM D1586). These sample locations are denoted with the prefix "S- "on the boring logs.
- 2.4-inch I.D. (2.5-inch O.D.) thin walled (Shelby) tube sampler (ASTM D1587). These sample locations are denoted with the prefix "U- "on the boring logs.
- 2.0-inch I.D. (2.5-inch O.D.) thick-walled, ring-lined (California) sampler (ASTM D3550). These sample locations are denoted with the prefix "CA- "on the boring logs.
- Bulk samples of cuttings were collected during auger advancement.

The ability to sample coarse particles was limited by the sampler sizes and sampling techniques; the collected samples likely underestimate the percentages of gravels, cobbles, or boulders within the embankment and colluvium.

A standard penetration test (SPT) was performed in general accordance with ASTM D1586 at the location of each split-spoon sample. At each SPT location, RJH obtained a "standard penetration resistance" or SPT N-value. The SPT N-value equals the number of blows that are required from a 140-pound hammer dropped 30 inches to drive a standard split-spoon sampler from 6 to 18 inches. At some locations, the SPT sampler encountered refusal (50 blows for less than 6 inches of penetration) prior to advancing 18 inches; therefore, SPT N-values and the associated samples could not be obtained at these locations. At some locations, more material was recovered than the penetration depth, likely because of either sampler seating blows or slough from the boring sides. Blow counts were also recorded at the location of California samples; these blow counts do not correlate directly to N-values, but provide a general indication of the consistency of the sampled material. The SPT N-values and blow counts presented in this Report were not adjusted to account for overburden pressures, hammer energy, etc. SPT and California sampler blow counts were likely influenced by the prevalence of larger gravel or cobbles.

Bedrock was not encountered in any of the borings.

At B-103(P), the initial boring location was terminated about 3 feet deep because of a boulder obstruction. This initial boring was backfilled with cuttings and the boring was re-drilled about 3 feet to the south. The surveyed coordinates presented in Table 2.1 correspond to the location of the re-drilled boring and the completed monitoring well.

Boring B-102B was terminated about 5 feet deep because of a boulder obstruction and was backfilled with cement-bentonite grout. Water from Carson Lake was used as the drilling fluid for mixing grout. The remaining borings were completed as open-standpipe monitoring wells as described in Section 2.7.

2.4 Daily Site Reports

RJH documented Site field activities in Daily Site Reports. Daily Site Reports are presented in Appendix D.

2.5 Logging and Sampling Procedures

RJH observed drilling procedures, recorded relevant drilling information, photographed and visually classified soil samples, and prepared a field log of each boring. In the field, soil samples were classified in general accordance with ASTM D2488 (visual-manual method), except for cuttings, where constituent percentages were estimated for the entire recovered sample, not just the fraction finer than 3 inches.

Recovered split-spoon samples were placed in sealed plastic bags to help preserve the natural moisture content of the material. Samples recovered from California samplers were generally capped and sealed with vinyl tape unless insufficient material was recovered and these samples were placed in sealed plastic bags to help preserve the natural moisture content. One successful Shelby tube sample was capped and sealed with vinyl tape. Bulk samples collected from auger cuttings were placed in either sealed plastic bags or canvas sample bags.

RJH prepared final boring logs based on field and laboratory classifications, quality assurance office review of samples, and indirect observations (i.e., drill chatter, drill resistance, etc.) as appropriate. Between recovered samples, the lithology presented on the boring logs is interpreted. Explanations of the soil descriptors used on the boring logs are presented in Appendix A. Boring logs are presented in Appendix B. Photographs of soil samples are presented in Appendix C.

2.6 Permeability Testing in Soil

RJH performed 13 tests to evaluate the hydraulic conductivity characteristics in the embankment fill and colluvium. In-situ permeability testing consisted of rising head and falling head tests over test intervals ranging between 0 and 21.0 feet in length. Testing was generally performed as follows:

Rising Head Test: Eleven rising head tests were performed in borings during drilling and in completed wells. Four tests were performed in borings during Symmetrix drilling; the casing was either raised from the bottom of the hole to expose the test interval or remained at the bottom of the hole. The test was conducted by measuring natural recovery of groundwater, because groundwater was removed from the hole during drilling by the use of an air compressor. No rising head tests were performed during auger drilling. The remaining seven tests were performed in wells; either a hand bailer or submersible pump was used to remove water from the well casing. The water level in the well was then measured over time as it recovered to near its original level. Hydraulic conductivity of the test interval was estimated from the field data using techniques published by Lambe and Whitman (1969) and equations by Hvorslev (1951) for all test configurations.

Falling Head Test: Two falling head tests were performed during auger drilling. Augers remained at the bottom of the hole and the vertical hydraulic conductivity was measured during testing. The augers were filled with water and then the water level within the augers was measured over time as it declined. Hole depths were measured again following the tests to confirm that hole collapse did not occur during testing. Hydraulic conductivity of the test interval was estimated from the field data using techniques published by Lambe and Whitman (1969) and equations by Hvorslev (1951) for both test configurations.

In-situ hydraulic conductivity test results are summarized in Table 2.2 and calculations are presented in Appendix E.

TABLE 2.2
SOIL HYDRAULIC CONDUCTIVITY TEST RESULTS

Boring ID	Test	Test Performed	Depth Interval (ft)[1]	Test Type	Hydraulic Conductivity (cm/s)[2]	USCS Soil Classification
Dam Embankment Fill						
B-102A(P)	K-1	During drilling	46.5 to 46.5	Falling Head[3]	$1.2x10^{-4}$	GP-GC
B-102A(P)	K-2	In well	36.5 to 48.0	Rising Head[4]	$2.7x10^{-6}$	GC, GP-GC, SC
B-102A(P)	K-3	In well	36.5 to 48.0	Rising Head[4]	$8.0x10^{-6}$	GC, GP-GC, SC
Colluvium						
B-101(P)	K-1	In well	53.0 to 74.0	Rising Head[4]	$4.0x10^{-6}$	Mostly CL, SP-SC
B-101(P)	K-2	In well	53.0 to 74.0	Rising Head[4]	$5.3x10^{-6}$	Mostly CL, SP-SC
B-103(P)	K-1	During drilling	30.0 to 30.0	Falling Head[3]	$1.6x10^{-3}$	SC
B-104(P)	K-1	During drilling	26.0 to 27.0	Rising Head[4]	$2.0x10^{-4}$	SC
B-104(P)	K-2	In well	8.9 to 14.5	Rising Head[4]	$7.9x10^{-5}$	CL
B-105A(P)	K-1	During drilling	21.0 to 22.0	Rising Head[4]	$1.4x10^{-4}$	CL
B-105A(P)	K-2	During drilling	52.0 to 52.0	Rising Head[3]	$1.1x10^{-3}$	SC
B-105A(P)	K-3	In well	53.0 to 73.5	Rising Head[4]	$1.0x10^{-4}$	CL, SC
B-105B(P)	K-1	During drilling	12.6 to 12.6	Rising Head[3]	$7.1x10^{-3}$	CL
B-105B(P)	K-2	In well	8.6 to 12.6	Rising Head[4]	$3.4x10^{-4}$	CL, SC

Notes:
1. Depth below the ground surface, measured along the orientation of the boring.
2. Geometric mean of hydraulic conductivity was calculated.
3. Tested vertical hydraulic conductivity.
4. Tested horizontal hydraulic conductivity

While drilling B-105A(P), water pressure generated by the Symmetrix drilling method caused water and air to be expelled at the ground surface between the casing and the boring wall. Rising head test B-105A(P), K-2 was performed to measure recovery of the groundwater and resolution of expelled water. The water and air expulsion ceased after approximately 33 minutes into the test, and the test was stopped after about 67 minutes once the groundwater level approached static conditions similar to rising head test B-105A(P), K-1. The results for test B-105A(P) K-2 presented in Table 2.2 are for the first 15 minutes of the test.

2.7 Monitoring Wells

2.7.1 Monitoring Well Installation

Open-standpipe monitoring wells were installed in all borings, except B-102B. The locations of the monitoring wells are shown on Figure 1.2. B-101(P) measures

groundwater levels in colluvium beneath the dam, B-102A(P) and B-103(P) measure groundwater levels in the embankment fill, and B-104(P), B-105A(P), and B-105B(P) measure groundwater levels in colluvium at the downstream toe of the dam. Information about construction of the monitoring wells is discussed below and shown on Figures 2.1 through 2.6.

All monitoring wells were installed following completion of the boring using conventional techniques, which generally consist of slowly introducing sand or gravel pack and boring sealing materials (bentonite chips or pellets and cement-bentonite grout) into the annular space between the boring wall and PVC pipe while simultaneously withdrawing either hollow-stem augers or casing from the ground.

Monitoring wells B-101(P), B-102A(P), and B-103(P) were constructed using solid and slotted PVC pipe and 10/20 silica sand pack. Well casings consisted of 2-inch Schedule 40 PVC pipe, except for B-101(P) which consisted of 1.5-inch Schedule 40 PVC pipe.

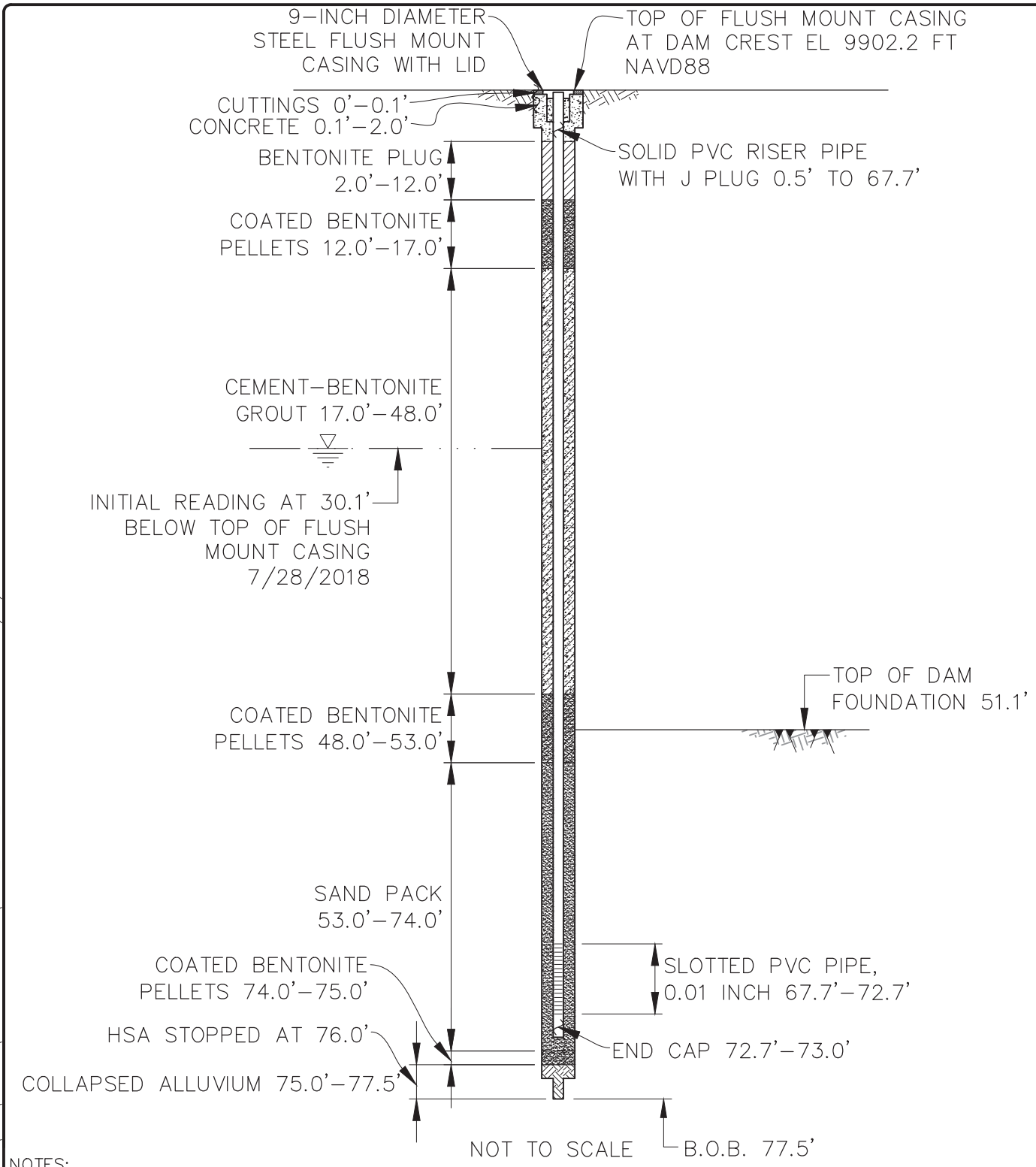
Monitoring wells B-104(P), B-105A(P), and B-105B(P) were constructed using solid PVC pipe, a pre-packed well screen, and minus ¼-inch gravel pack. The pre-packed well screen consisted of slotted PVC well screen surrounded by stainless steel mesh, which encapsulates 20/40 sized well sand between the PVC pipe exterior and the mesh interior. The pre-packed well screen was 2.0-inches I.D. and 2.8-inches O.D. Schedule 40 PVC pipe.

After installation, all monitoring wells were developed to remove groundwater and drilling water from the well and sand or gravel pack. The monitoring wells were developed by surging, bailing, and pumping water from the wells with a submersible pump until either no additional water could be removed or the water was clear.

2.7.2 Monitoring Well Readings

Monitoring wells were measured during the fieldwork. Groundwater level measurements were obtained by RJH while onsite. The City and the SEO also obtained groundwater level measurements. Measured groundwater levels obtained by RJH, the City, and the SEO are presented on Figure 2.7. Data are provided in Appendix F.

The groundwater level measured in B-101(P) on August 23, 2018 and September 6, 2018, do not appear to follow the trend of B-102A(P) or the general trend of decreasing reservoir level. These well measurements may have been improperly recorded.



NOTES:

1. MONITORING WELL INSTALLED 7/28/2018 WITH 7.75-INCH O.D. HOLLOW STEM AUGERS.
2. LOCATION: LAT 38.995296, LONG -108.109759 DEGREES WGS84. BASED ON HANDHELD GPS.
3. TWO 9/16-INCH BOLTS SECURE SURFACE MOUNT CASING.
4. PVC RISER PIPE 1.5 INCHES I.D. 1.9 INCHES O.D. SCH 40. SAND PACK SIZE 10/20.
5. ANNULAR BACKFILL FROM 76.0 FEET TO BOTTOM OF BORING IS ALLUVIUM THAT COLLAPSED INTO BORING AS HOLLOW STEM AUGERS WERE WITHDRAWN.
6. WELL DEVELOPED WITH HAND BAILER 7/28/2018 AND SUBMERSIBLE PUMP 9/18/2018.

CITY OF Grand Junction COLORADO	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	MONITORING WELL INSTALLATION B-101(P)
		PROJECT NO. 18115	January 2019 **Figure 2.1**

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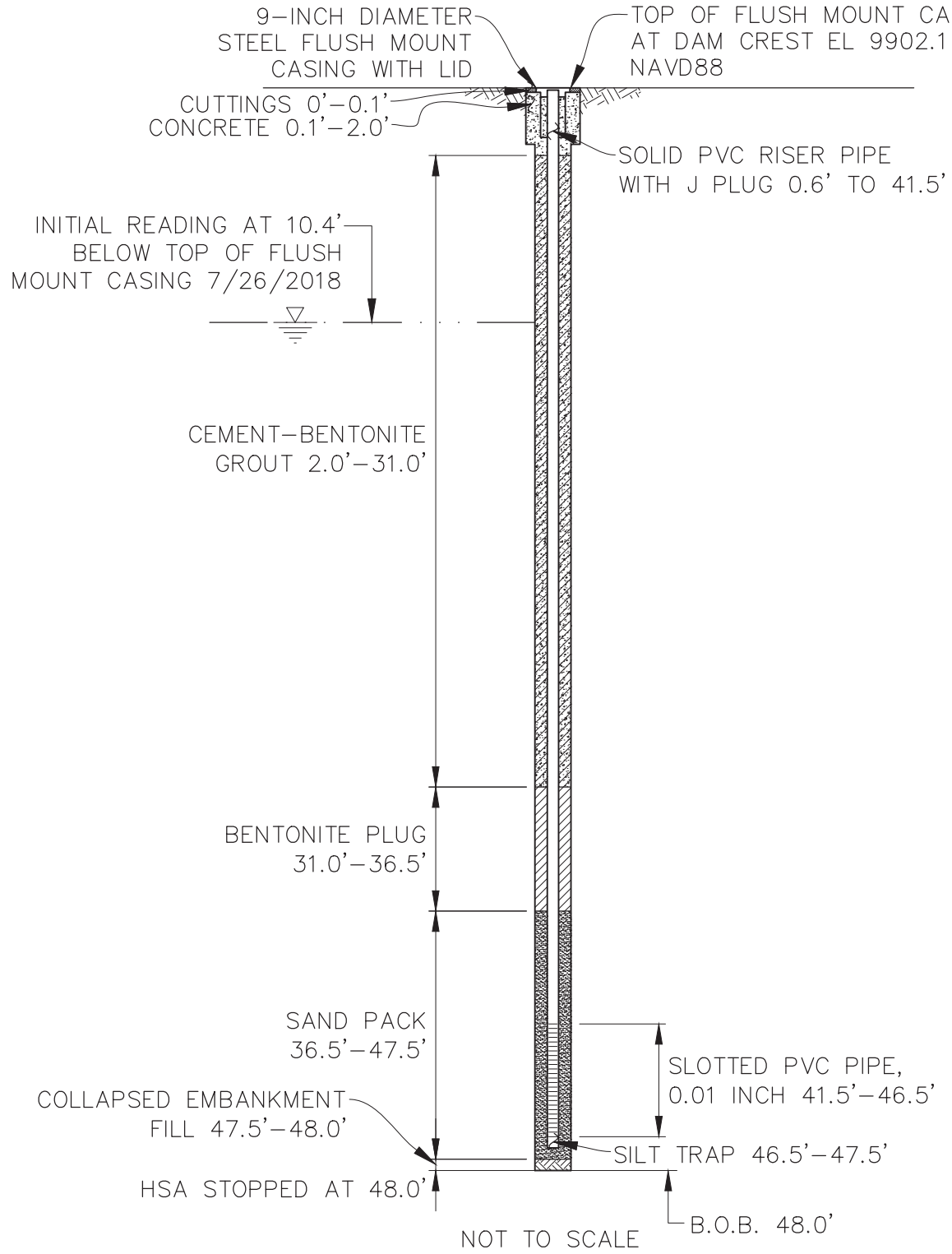

NOTES:

1. MONITORING WELL INSTALLED 7/26/2018 WITH 7.75–INCH O.D. HOLLOW STEM AUGERS.
2. LOCATION: LAT 38.995677, LONG –108.109800 DEGREES WGS84. BASED ON HANDHELD GPS.
3. TWO 9/16–INCH BOLTS SECURE SURFACE MOUNT CASING.
4. PVC RISER PIPE 2.0 INCHES I.D. 2–3/8 INCHES O.D. SCH 40. SAND PACK SIZE 10/20.
5. ANNULAR BACKFILL FROM 47.5 FEET TO BOTTOM OF BORING IS EMBANKMENT FILL THAT COLLAPSED INTO BORING AS HOLLOW STEM AUGERS WERE WITHDRAWN.
6. WELL DEVELOPED WITH SURGE BLOCK AND SUBMERSIBLE PUMP 7/26/2018.

CITY OF Grand Junction COLORADO	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	MONITORING WELL INSTALLATION B-102A(P)
		PROJECT NO. 18115	January 2019 **Figure 2.2**

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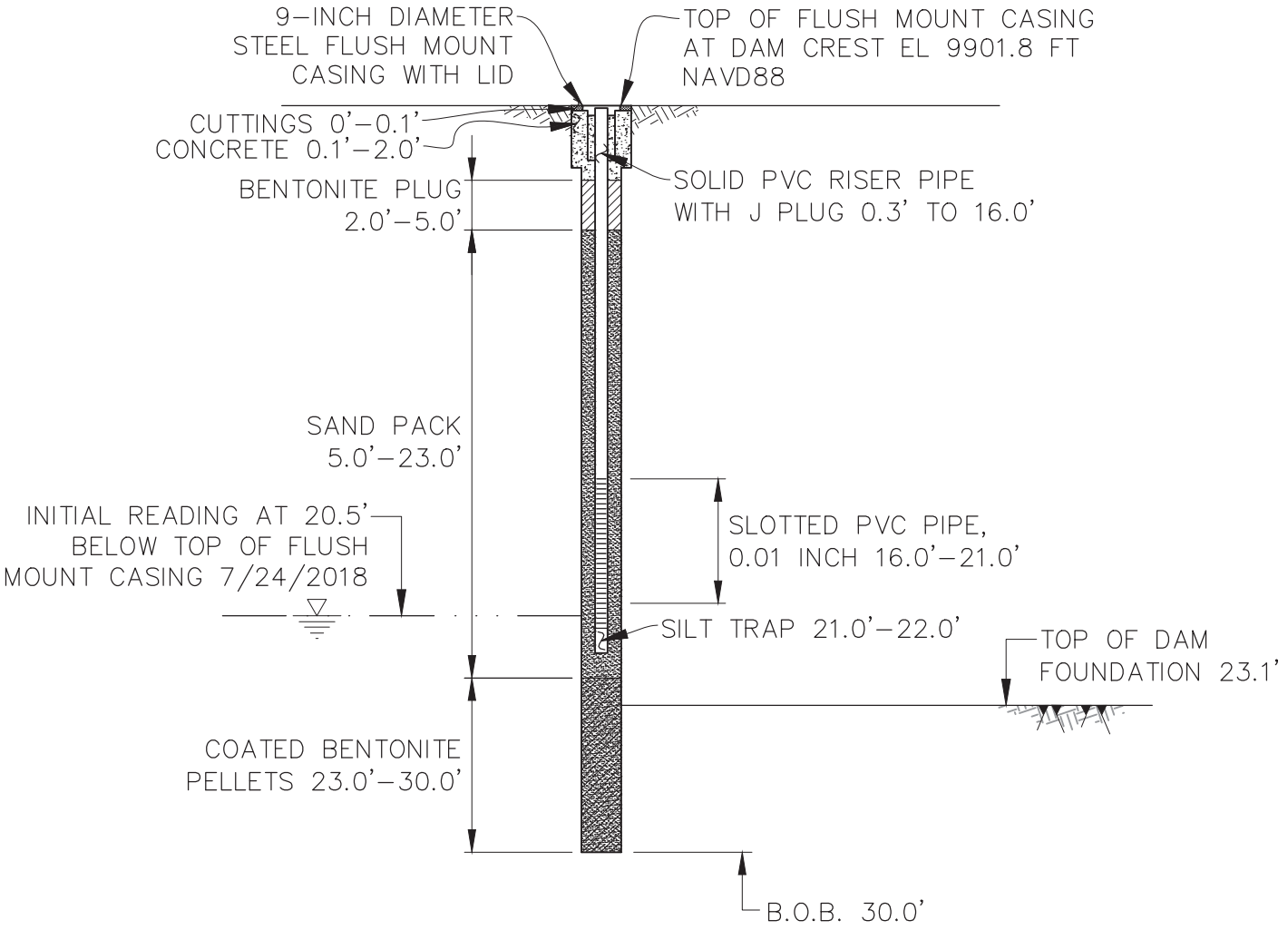

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NOTES:

1. MONITORING WELL INSTALLED 7/24/2018 WITH 7.75–INCH O.D. HOLLOW STEM AUGERS.
2. LOCATION: LAT 38.996095, LONG –108.109881 DEGREES WGS84. BASED ON HANDHELD GPS.
3. TWO 9/16–INCH BOLTS SECURE SURFACE MOUNT CASING.
4. PVC RISER PIPE 2.0 INCHES I.D. 2–3/8 INCHES O.D. SCH 40. SAND PACK SIZE 10/20.
5. WELL DEVELOPED WITH SURGE BLOCK AND SUBMERSIBLE PUMP 7/24/2018.

CITY OF Grand Junction COLORADO	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	MONITORING WELL INSTALLATION B-103(P)
		PROJECT NO. 18115	January 2019 **Figure 2.3**

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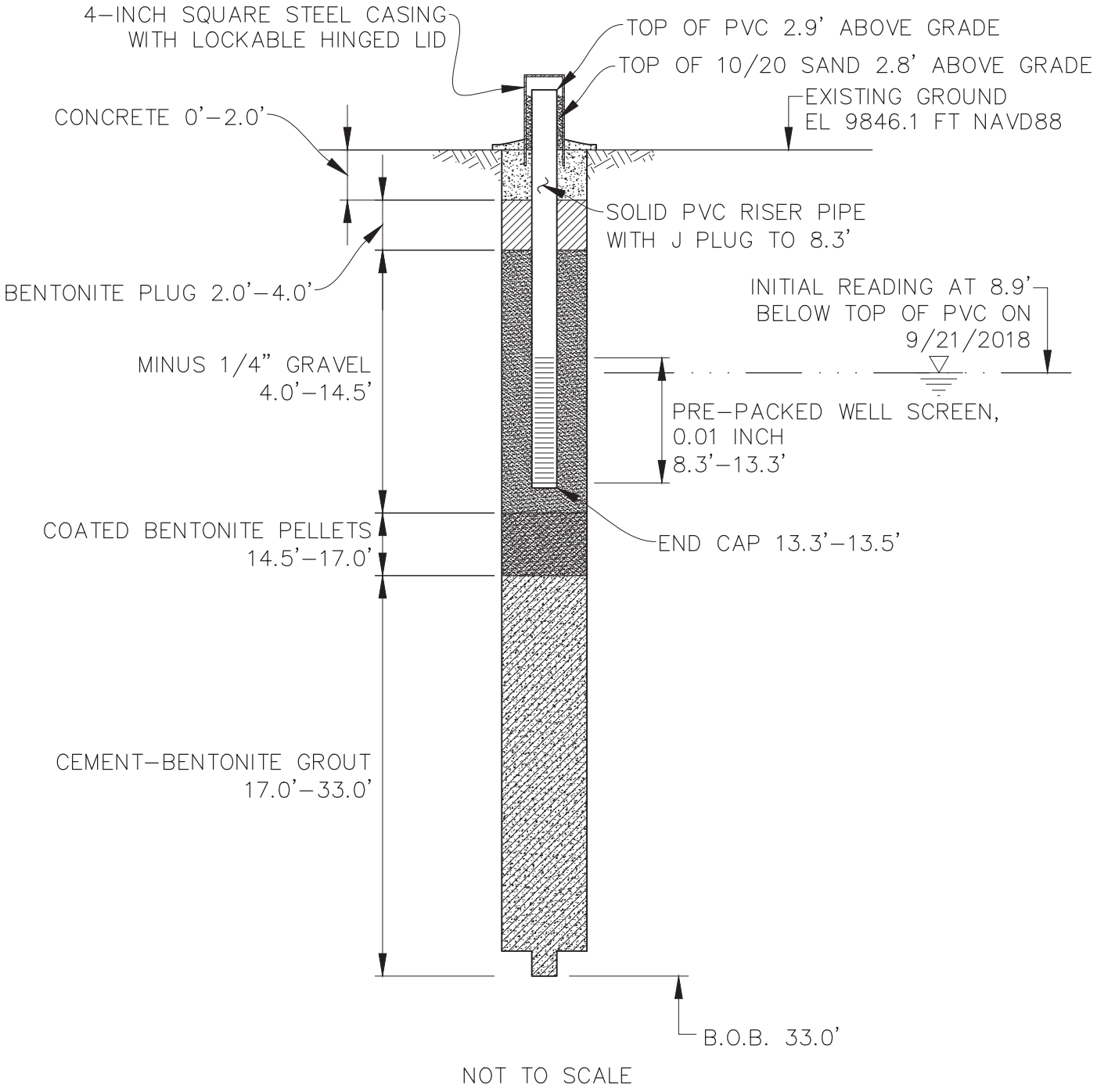

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NOTES:

1. MONITORING WELL INSTALLED 9/21/2018 WITH 5–3/8 INCHES O.D. SYMMETRIX DRIVE CASING ADVANCER.
2. LOCATION: LAT 38.995285, LONG –108.110292 DEGREES WGS84. BASED ON HANDHELD GPS.
3. PVC RISER PIPE 2.0 INCHES I.D. 2–3/8 INCHES O.D. SCHEDULE 40.
4. PRE–PACKED WELL SCREEN 2.0 INCHES I.D. 2.8 INCHES O.D. PVC SCHEDULE 40. SAND PACK SIZE 20/40.
5. WELL DEVELOPED WITH SURGE BLOCK AND SUBMERSIBLE PUMP 9/21/2018.

CITY OF Grand Junction COLORADO	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	MONITORING WELL INSTALLATION B-104(P)
		PROJECT NO. 18115	January 2019 **Figure 2.4**

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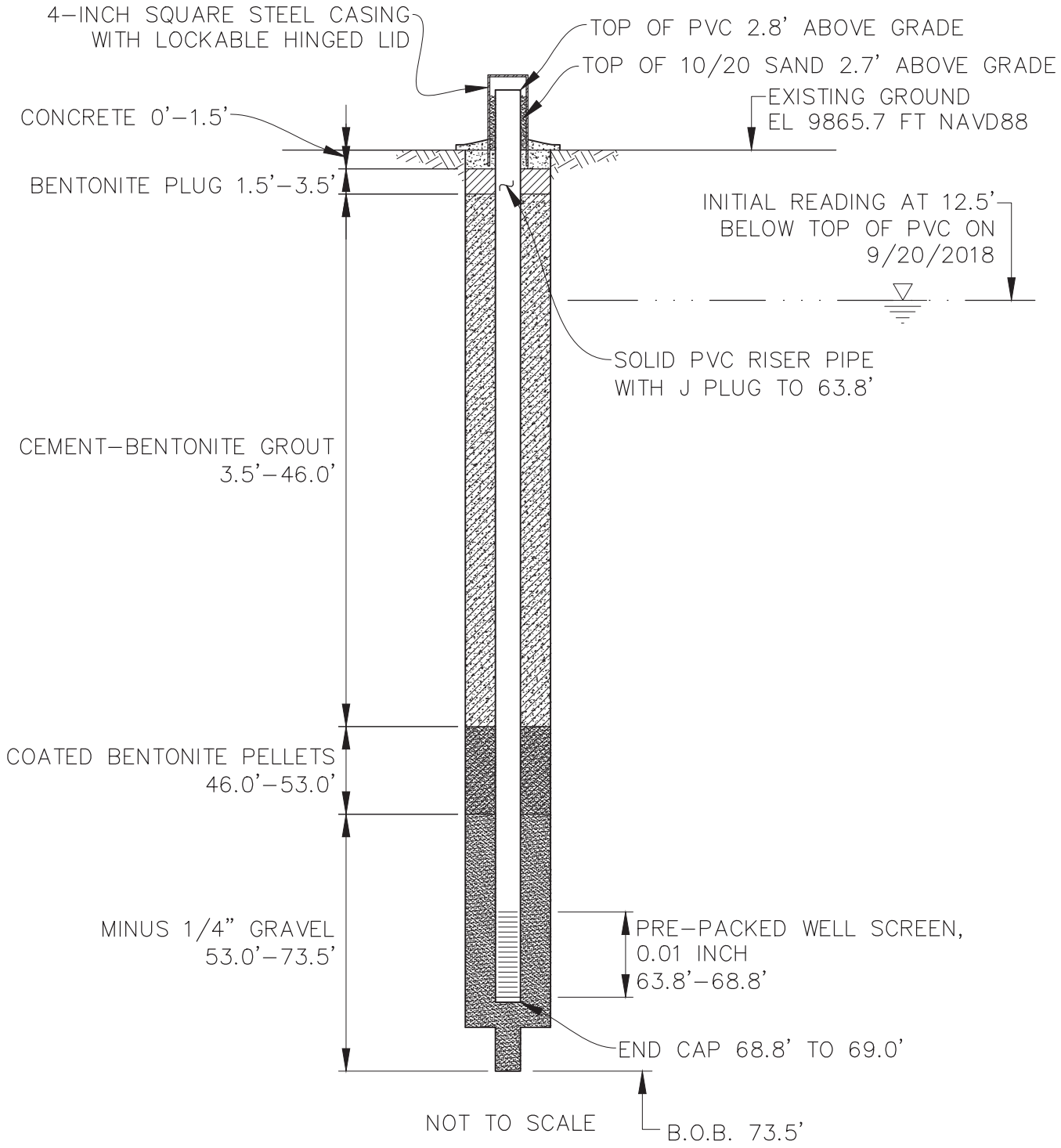

NOTES:

1. MONITORING WELL INSTALLED 9/20/2018 WITH 5–3/8 INCHES O.D. SYMMETRIX DRIVE CASING ADVANCER.
2. LOCATION: LAT 38.995632, LONG –108.110160 DEGREES WGS84. BASED ON HANDHELD GPS.
3. PVC RISER PIPE 2.0 INCHES I.D. 2–3/8 INCHES O.D. SCHEDULE 40.
4. PRE–PACKED WELL SCREEN 2.0 INCHES I.D. 2.8 INCHES O.D. PVC SCHEDULE 40. SAND PACK SIZE 20/40.
5. WELL DEVELOPED WITH SURGE BLOCK AND SUBMERSIBLE PUMP 9/20/2018.

CITY OF Grand Junction COLORADO	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	MONITORING WELL INSTALLATION B-105A(P)
		PROJECT NO. 18115	January 2019 **Figure 2.5**

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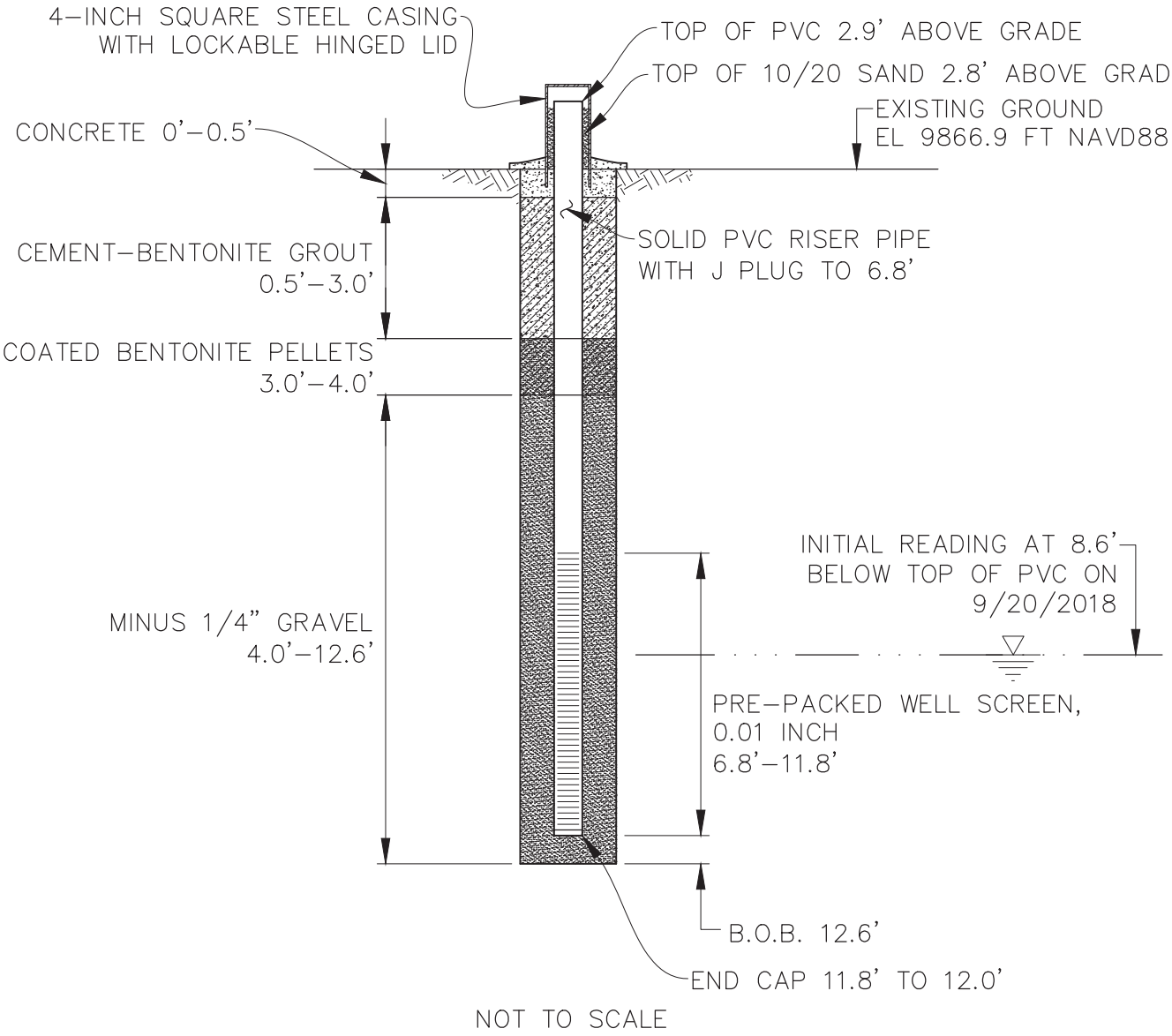

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NOTES:

1. MONITORING WELL INSTALLED 9/20/2018 WITH 5–3/8 INCHES O.D. SYMMETRIX DRIVE CASING ADVANCER.
2. LOCATION: LAT 38.995681, LONG –108.110127 DEGREES WGS84. BASED ON HANDHELD GPS.
3. PVC RISER PIPE 2.0 INCHES I.D. 2–3/8 INCHES O.D. SCHEDULE 40.
4. PRE–PACKED WELL SCREEN 2.0 INCHES I.D. 2.8 INCHES O.D. PVC SCHEDULE 40. SAND PACK SIZE 20/40.
5. WELL DEVELOPED WITH SURGE BLOCK AND SUBMERSIBLE PUMP 9/20/2018.

CITY OF Grand Junction COLORADO	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	MONITORING WELL INSTALLATION B-105B(P)
		PROJECT NO. 18115	January 2019 **Figure 2.6**

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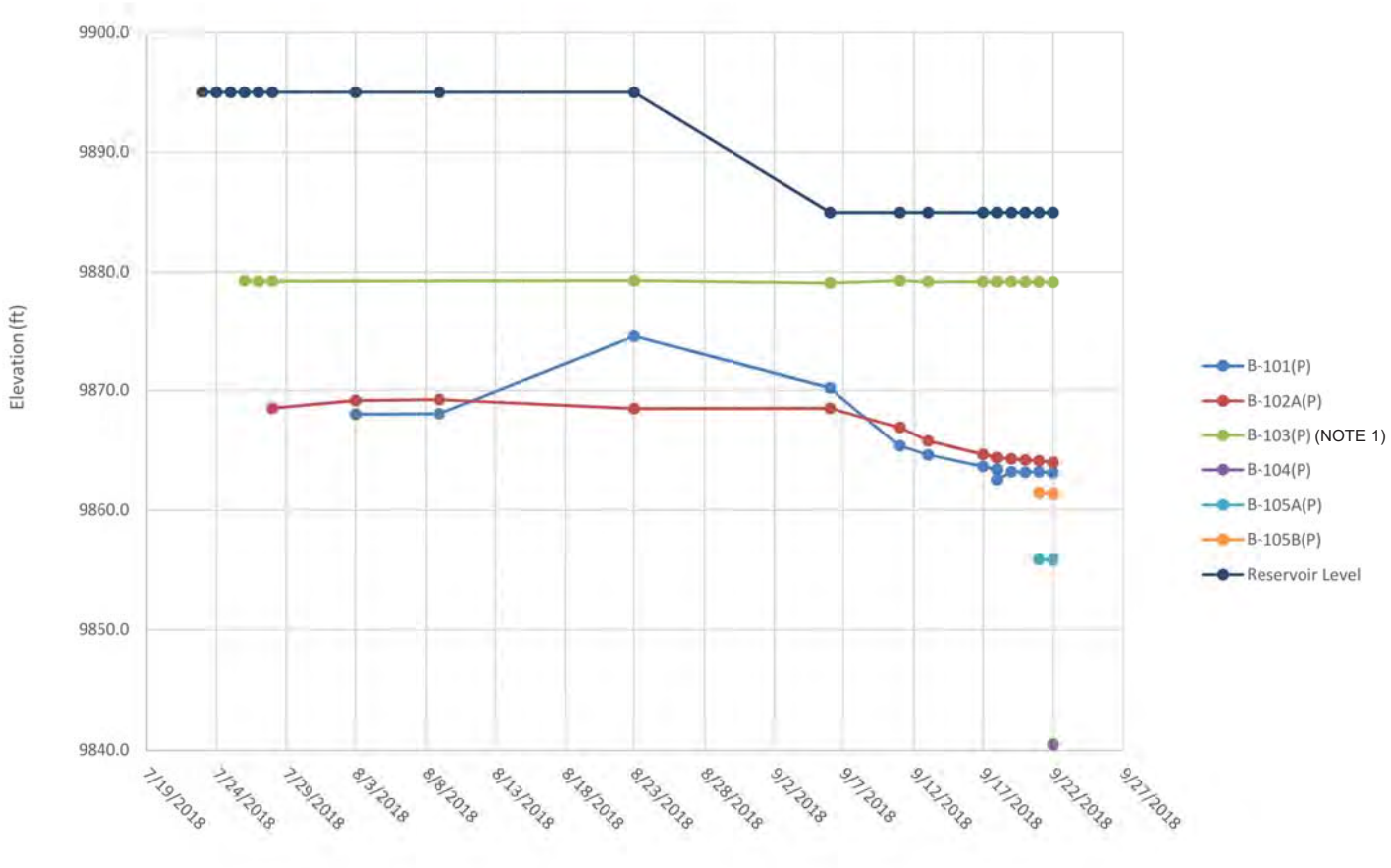

NOTE:

1. MEASUREMENTS FOR B–103(P) ARE THE BOTTOM OF THE DRY WELL AND DO NOT REPRESENT THE GROUNDWATER LEVEL

REPRODUCE IN COLOR

City of Grand Junction Colorado	RJH Consultants, Inc.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	FIELD INVESTIGATION MONITORING WELL DATA
		PROJECT NO. 18115	January 2019 **Figure 2.7**

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SECTION 3 - LABORATORY TESTING

Laboratory tests were performed on representative samples of soil collected from the borings. RJH retained Advanced Terra Testing of Lakewood, Colorado to perform the laboratory testing. The tests consisted of:

Index Tests:

- Three moisture content and density tests (ASTM D2216 and D7263).
- Five Atterberg limit tests (ASTM D4318).
- Four grain-size analyses (ASTM D6913).
- Two grain-size analyses with hydrometer (ASTM D6913 and D7928).
- Three percent minus #200 analyses (ASTM D1140).
- Three standard Proctor compaction tests (ASTM D698).
- One one-dimensional consolidation test (ASTM D2435).
- Three corrosion suite tests (ASTM C1580, D4972, D1411, and G187).
- Two pinhole dispersion tests (ASTM D4647 Method A).

Permeability Tests:

- Three back pressure permeability tests, flow pump method (ASTM D5084 Method D).

Strength Tests:

- One series of three consolidated undrained triaxial compression tests (ASTM D4767).
- One unconfined compressive strength test (ASTM D2166).

The unconfined compressive strength tests could not be performed on two samples because gravel prevented the samples from remaining intact during extrusion. Similar material recovery issues may have influenced other laboratory results.

Laboratory index test results are summarized in Table 3.1. Laboratory permeability and strength test results are summarized in Table 3.2. Strength test results are shown on Figures 3.1 and 3.2. Laboratory test sheets are provided in Appendix G.

TABLE 3.1
SUMMARY OF INDEX LABORATORY TEST RESULTS

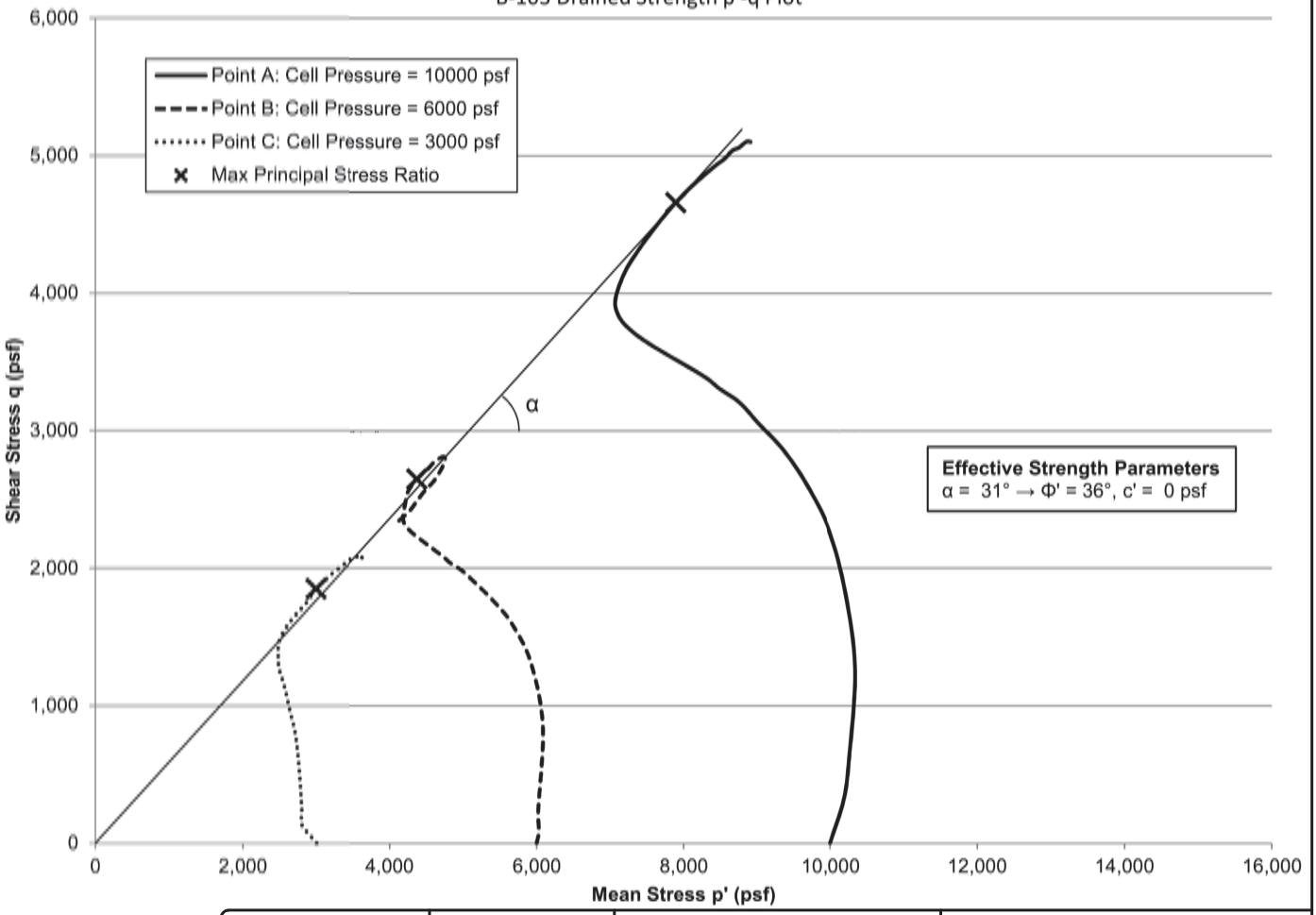
Boring	Sample ID	Depth (ft)	Moisture Content (%)	Dry Density (pcf)	Liquid Limit, LL	Plasticity Index, PI	Percent Gravel (3" to #4)	Percent Sand (#4 to #200)	Percent Fines (< #200)	Optimum Moisture Content (%)	Maximum Dry Unit Weight (pcf)	Coefficient of Compression, Cc	Coefficient of Re-Compression, Cr	Preconsolidation Stress (psf)	Sulfate Concentration (ppm)	Chloride Concentration (ppm)	pH	Minimum Resistivity (Ω*cm)	Sulfide Concentration (ppm)	Pinhole Dispersion Test Results
Dam Embankment Fill																				
B-101(P)	Bu-6	15.0 to 25.0								10.4[1]	131.1[1]									
B-101(P)	Bu-13	25.0 to 40.0			28	11	25.5	28.6	45.9											
B-101(P)	CA-11	30.0 to 31.0	14.3	109.6																ND1[2][3]
B-101(P)	CA-16	42.5 to 43.5					30.1	30.6	39.3											
B-101(P)	Bu-15	45.0 to 50.0													23	38.5	6.8	2,400	0.104	
B-101(P)	S-19	50.6 to 51.5													107	144	7.3	2,600	0.184	
B-102A(P)	Bu-10	18.0 to 41.5			28	11	24.3	29.1	46.6											
B-102A(P)	CA-14	29.0 to 30.0					57.0	21.1	21.9											
B-102A(P)	CA-20	44.0 to 45.0	25.4	95.6											5	108	7.4	1,540	0.01	
B-103(P)	Bu-11	10.0 to 17.5			27	11	23.9	33.1	43.0	10.3[1]	131.3[1]									
B-103(P)	CA-8	15.0 to 16.0	10.8	105.1			46.0	21.4	32.6											ND1[2][4]
Colluvium																				
B-101(P)	Bu-20	51.0 to 65.0			27	12	19.8	26.2	54.0	12.1[1]	126.9[1]									
B-103(P)	U-14	27.5 to 29.9	25.1[5]	101.6[5]			24.8	43.9	31.3			0.224	0.011	6,780						
B-104(P)	CA-9	32.0 to 33.0	19.7	111.8	32	13	33.4	8.5	58.1											

Notes:
1. Results in this table are presented with oversized particle corrections. Tests were performed using standard energy (ASTM D698) and the maximum particle size included in the test was ¾ inch. The percentage of oversized material exceeded the recommendations of ASTM D698.
2. ND1 corresponds to nondispersive clays with very slight to no colloidal erosion under 15 inches to 40 inches of head.
3. Specimen remolded to a dry unit weight of 110 pounds per cubic foot (pcf) and a water content of 17 percent.
4. Specimen remolded to a dry unit weight of 106 pcf and a water content of 14 percent.
5. Average of tests from three triaxial shear test specimens.

TABLE 3.2
SUMMARY OF PERMEABILITY AND STRENGH LABORATORY TEST RESULTS

Boring	Sample ID	Depth (ft)	Permeability (cm/s)	Effective Strength		Total Strength		Unconfined Compressive Strength (psf)
				ϕ' (deg.)	c' (psf)	ϕ_T (deg.)	c_T (psf)	
Colluvium								
B-103(P)	U-14	27.5 to 29.9	$1.7x10^{-5(1)}$ $3.1x10^{-3(2)}$ $3.4x10^{-5(3)}$	$36^{(4,5)}$	$0^{(4,5)}$	$22^{(4,5)}$	$640^{(4,5)}$	
B-104(P)	CA-9	32.0 to 33.0						958

Notes:
1. Permeability test performed at the consolidated undrained triaxial compression test pressure of 10,000 pounds per square foot (psf).
2. Permeability test performed at the consolidated undrained triaxial compression test pressure of 6,000 psf.
3. Permeability test performed at the consolidated undrained triaxial compression test pressure of 3,000 psf.
4. Consolidated undrained triaxial compression test performed in general accordance with ASTM D4767 with confining pressures of 3,000; 6,000; and 10,000 psf.
5. Based on maximum principal stress ratio.



City of Grand Junction Colorado	RJH Consultants, Inc.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	COLLUVIUM DRAINED STRENGTH
		PROJECT NO. 18115	January 2019 **Figure 3.1**

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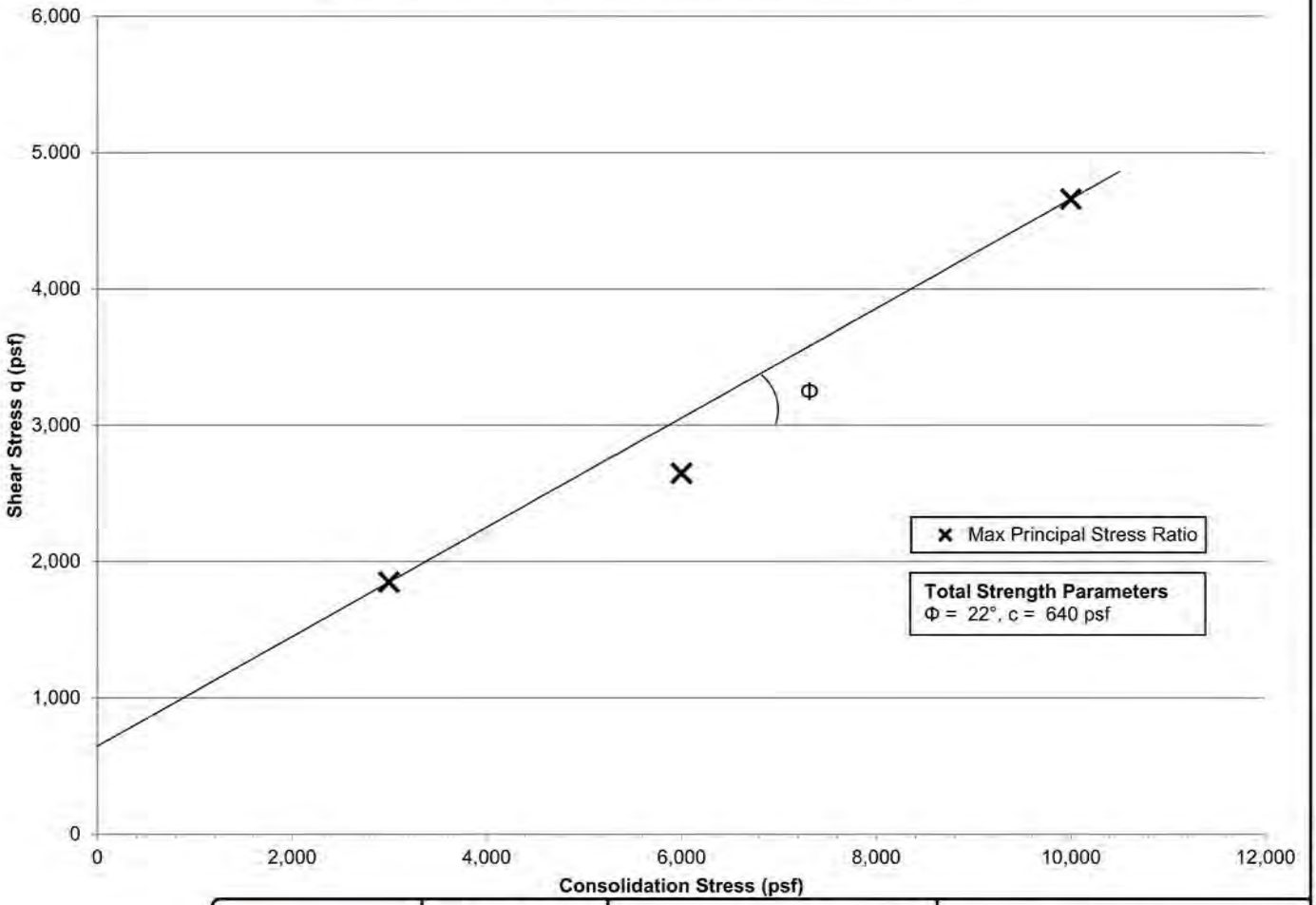


Grand Junction	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	COLLUVIUM UNDRAINED STRENGTH
		PROJECT NO. 18115	January 2019 **Figure 3.2**

P:\18115 - HOGCHUTE DAM SAFETY EVALUATION\CAD\FIGURES\MONITORING WELLS\FOUNDATION.DWG 1/2/2019 11:24 AM

SECTION 4 - SITE AND SUBSURFACE CONDITIONS

4.1 General Geology

According to published maps (Ellis and Gabaldo, 1989), the Site is located in the southwest portion of the Piceance Basin on the Grand Mesa. The Piceance Basin is a Late Cretaceous to early Tertiary-age (56 to 100 million years old) feature with a series of Laramide uplifts defining the boundaries of the structural basin. The Grand Mesa is capped by resistant basalt flows. Geologic units at the Site consist of Quaternary-age (less than 2.6 million years old) colluvium overlying Tertiary-age (Eocene, 33.9 to 56 million years old) Green River Formation bedrock. Other geologic units in the nearby area surrounding the Site consist of Quaternary-age terrace gravel and till, and Tertiary-age basalt and Wasatch Formation. The published geology at the Site and nearby surrounding area is shown on Figure 4.1.

Published maps do not show faults in the Site vicinity; however, the southern edge of the Grand Mesa is defined by an escarpment above the flat-lying valley below. The Site is located near the top of the escarpment.

4.2 Site Geology

The Site is generally covered by native vegetation that would be typical of a wet, high-altitude environment. Evidence of possible reservoir seepage downstream of the dam included a small area of inactive seeps near shrubs on the hillside to the right of the outlet works and water flowing under the downstream rock shell toe and discharging from a drain to the right of the outlet works. No evidence of active seepage was observed near the shrubs during our work. Seepage from the drain appeared to be flowing clear at the time of our work and at a rate of approximately 4 to 5 gallons per minute (gpm).

The geologic Site conditions observed by RJH generally agreed with the published geologic mapping. The Site generally consists of colluvial deposits with basalt outcrops forming cliffs to the west, north, and east. Bedrock was not encountered in any of the borings.

4.3 Subsurface Conditions

4.3.1 General Subsurface Profile

The subsurface units encountered in the borings were embankment fill and colluvium. Bedrock was not encountered in any of the borings. Borings at the dam's downstream toe did not encounter embankment fill.

The following sections describe the properties of the encountered materials. Subsurface sections are shown on Figures 4.2 through 4.5.

4.3.2 Embankment Fill

Embankment fill was encountered at the ground surface in B-101(P), B-102A(P), B-102B, and B-103(P). Embankment fill extended to depths of 51.1 feet and 23.1 feet in B-101(P) and B-103(P), respectively, and was underlain by colluvial deposits in both borings. Borings B-102A(P) and B-102B encountered refusal at 48.0 feet and 5.0 feet, respectively, and did not extend into the colluvium beneath the dam.

Approximately the first foot of embankment fill was crushed gravel road base. In order of prevalence, the remaining embankment fill consisted of clayey sand with gravel (SC), clayey gravel with sand (GC), clayey sand (SC), and poorly graded gravel with clay and sand (GP-GC). Embankment fill contained 15 to 80 percent fine to coarse grained gravel, 15 to 65 percent fine to coarse grained sand, 5 to 47 percent low to medium plasticity fines, and less than 5 percent cobbles. The maximum recovered particle size was 4 inches. The composition and maximum particle size observed in the recovered samples were influenced by the size of the samplers; difficult drilling and sampling conditions were encountered that are likely indicative of larger cobbles or boulders within the embankment fill.

Embankment fill was generally moist above the water table and moist to wet below the water table. Drive sampler refusal (50 blows for less than 6 inches) was encountered at six locations after advancing the sampler 0.1 to 0.3 foot. At 28 other sample locations, uncorrected SPT N-values ranged from 16 to 54 and averaged 35. In our opinion, the SPT results were likely influenced by larger gravel or cobbles within the embankment fill and are not reliable to correlate with material density; however, apparent density based on SPT values is reported on the boring logs.

Three in-situ permeability tests were performed in the embankment fill in B-102A(P). The calculated vertical hydraulic conductivity was 1.2×10^{-4} centimeters per second (cm/s) and the calculated horizontal hydraulic conductivity values ranged from 8.0×10^{-6} to 2.7×10^{-6} cm/s.

As discussed above, observations during drilling and sampling indicate the presence of significant fractions of large materials, including gravels, cobbles, and boulders in the embankment fill. The results of the field tests were likely influenced by the presence of these larger materials.

Laboratory index property tests were performed on eleven samples of embankment fill material from B-101(P), B-102A(P), and B-103(P), ranging in depth from 10.0 to 51.5 feet. Some of the results are summarized as follows:

- The natural moisture content ranged from 10.8 to 25.4 percent and averaged 16.8 percent.
- The natural dry density ranged from 95.6 to 109.6 pcf and averaged 103.4 pcf.
- The liquid limit was either 27 or 28 and the plasticity index was 11 for all samples.
- Two standard Proctor tests were performed, and the results were very similar between the two samples. The maximum dry density for sample B-103(P), Bu-11 was 131.3 pcf at optimum moisture of 10.3 percent. The maximum dry density for sample B-101(P), Bu-6 was 131.3 pcf at optimum moisture of 10.4 percent.
- Three samples from B-101(P) and B-102A(P) had a suite of corrosion tests performed. The three samples were taken at depths near the approximated depth of the outlet works conduit, ranging from 44.0 to 51.5 feet. See Table 3.1 for corrosion test results.
- Embankment fill materials were classified as nondispersive.

4.3.3 Colluvium (Qc)

Colluvium was encountered at the ground surface in B-104(P), B-105A(P), and B-105B(P), and extended to the final boring depths of 33.0, 73.5, and 12.6 feet, respectively. Colluvium was encountered beneath embankment fill and within B-101(P) and B-103(P) at approximately 51.1 and 23.1 feet, respectively, and extended to the final boring depths of 77.5 and 30.0 feet, respectively.

In order of prevalence, colluvium consisted of sandy lean clay with gravel (CL), clayey sand with gravel (SC), gravelly lean clay with sand (CL), lean clay with sand (CL), lean clay (CL), sandy lean clay (CL), poorly graded gravel with silt and sand (GP-GM), lean clay with gravel (CL), and poorly graded sand with clay and gravel (SP-SC). Colluvium contained 5 to 100 percent nonplastic to highly plastic fines, fines were mostly low to medium plasticity, 0 to 80 percent fine to coarse grained sand, and 0 to 75 percent fine to coarse grained gravel. The maximum recovered particle size was 2.0 inches. Chlorite deposits were present in colluvium from depths of 25.3 to 30.3 feet in B-104(P). The composition and maximum particle size observed in the recovered samples were influenced by the size of the samplers; difficult sampling conditions were encountered that are likely indicative of larger gravels, cobbles, or boulders within the colluvium. The colluvium is anticipated to be a heterogeneous material based on its formation from talus deposit, landslide, earthflow, and soil creep processes (Ellis and Gabaldo, 1989).

Colluvium was generally dry to moist above the water table and moist to wet below the water table. Drive samplers encountered refusal (50 blows for less than 6 inches) at nine locations after advancing the sampler 0.2 to 0.4 foot. At 20 other sample locations, uncorrected SPT N-values ranged from 5 to 76 and averaged 33. In our opinion, the SPT results were likely influenced by larger gravel, cobbles, or boulders within the colluvium and are not reliable to correlate with material density; however, apparent density based on SPT values is reported on the boring logs.

Ten in-situ permeability tests were performed in the colluvium; the calculated vertical hydraulic conductivity values ranged from 7.1×10^{-3} to 1.1×10^{-3} cm/s and the calculated horizontal hydraulic conductivity values ranged from 3.4×10^{-4} to 4.0×10^{-6} cm/s.

As discussed above, observations during drilling and sampling indicate the presence of significant fractions of large materials, including gravels, cobbles, and boulders in the colluvium. The results of the field tests were likely influenced by the presence of these larger materials.

Laboratory index, permeability, and strength tests were performed on three samples of colluvium from B-101(P), B-103(P), and B-104(P). Some of the results are summarized as follows:

- The natural moisture content ranged from 19.7 percent to 25.1 percent.
- The natural dry density ranged from 101.6 pcf to 111.8 pcf.
- The liquid limit was either 27 or 32 and the plasticity index was 12 or 13.

- One standard Proctor test was performed. The maximum dry density for sample B-101(P), Bu-20 was 126.9 pcf at optimum moisture of 12.1 percent.
- Consolidated-undrained triaxial tests and permeability tests were performed on one sample at compression test pressures of 3,000, 6,000, and 10,000 pounds per square foot (psf). The effective strength parameters were phi' of 36 degrees and c' of zero psf. The total strength parameters were phi of 22 degrees and cohesion of 640 psf. The triaxial data are based on the maximum principal stress ratio.
- The permeability results at 3,000, 6,000, and 10,000 psf compression test pressures were 3.4×10^{-5}, 3.1×10^{-3}, and 1.7×10^{-5} cm/s, respectively.
- See Table 3.1 for results of the one-dimensional compression test and see Table 3.2 for results on the unconfined compressive strength test.

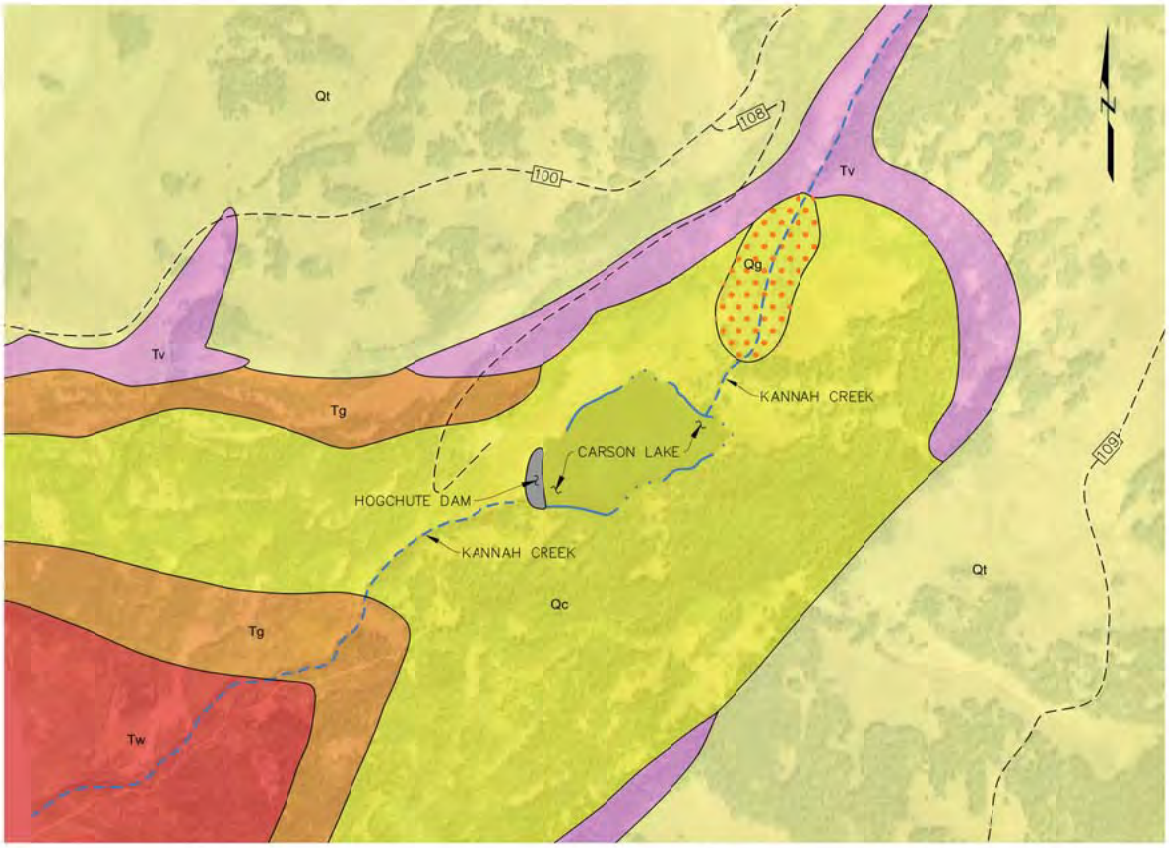
4.3.4 Groundwater

Three monitoring wells are located along the crest of the dam (B-101(P), B-102A(P), and B-103(P)). The static water level was recorded in B-101(P) and B-102A(P) at about El. 9868.0 and El. 9869.2, respectively on August 9, 2018, when the reservoir was full at El. 9895. The water level in both wells dropped about 5 feet with a 10-foot decrease in reservoir elevation (to about El. 9885 on September 22, 2018). It is our opinion that the readings for B-101(P) taken on August 23, 2018 and September 6, 2018 were likely recorded in error because they do not appear to follow the trend of B-102A(P) or the general trend of decreasing reservoir level, and do not represent the water level during that period. No water was measured in B-103(P) when the reservoir was full or when the reservoir was lowered 10 feet. The measurements for B-103(P) on Figure 2.7 are the bottom of the dry well; it is likely that groundwater is lower than the B-103(P) screened interval.

Three monitoring wells are located in colluvium downstream of the dam (B-104(P), B-105A(P), and B-105B(P)). The water level at the downstream toe of the dam in B-105A(P) and B-105(B) was at El. 9855.9 and El. 9861.4, respectively, on September 22, 2018 when the reservoir was at El. 9885 feet (about 10 feet below the spillway level). The water level in B-104(P), which is about 45 feet downstream from the embankment toe on the left side of the outlet works, was at El. 9840.5 on September 22, 2018 when the reservoir was at El. 9885.

The drilling operations did not appear to affect the observed flow at the drain near the outlet works or the seeps near shrubs on the hillside. The seepage at the outlet works

remained clear prior to, during, and following drilling activities and the seepage rate remained between approximately 4 and 5 gpm.



LEGEND:

- EMBANKMENT FILL
- Qg TERRACE GRAVEL
- Qc COLLUVIUM
- Qt UNDIFFERENTIATED TILL
- Tv BASALT
- Tg GREEN RIVER FORMATION
- Tw WASATCH FORMATION
- CREEK
- U.S. FOREST SERVICE ROAD
- BODY OF WATER

NOTES:

1. AERIAL IMAGERY WAS OBTAINED FROM MICROSOFT BING, IMAGERY DATE UNKNOWN.
2. MAPPED UNITS ARE APPROXIMATE AND BASED ON INFORMATION FROM A PUBLISHED U.S. GEOLOGICAL SURVEY MAP (ELLIS AND GABALDO, 1989). MAPPED UNITS ARE SHOWN ON THIS FIGURE FOR COMPLETENESS AND MAY NOT HAVE BEEN ENCOUNTERED DURING FIELD WORK PERFORMED BY RJH. NOT ALL UNITS SHOWN IMPACT THIS PROJECT AND THEREFORE ARE NOT DESCRIBED IN THE REPORT TEXT.
3. APPROXIMATE LOCATIONS OF U.S. FOREST SERVICE ROADS, HOGCHUTE DAM, AND CARSON LAKE DEVELOPED USING MICROSOFT BING AERIAL IMAGERY.

REPRODUCE IN COLOR

0 500 1000 2000 3000
SCALE IN FEET

City of Grand Junction Colorado

RJH Consultants, Inc.

HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	PUBLISHED SITE GEOLOGY
PROJECT NO. 18115	January 2019 Figure 4.1

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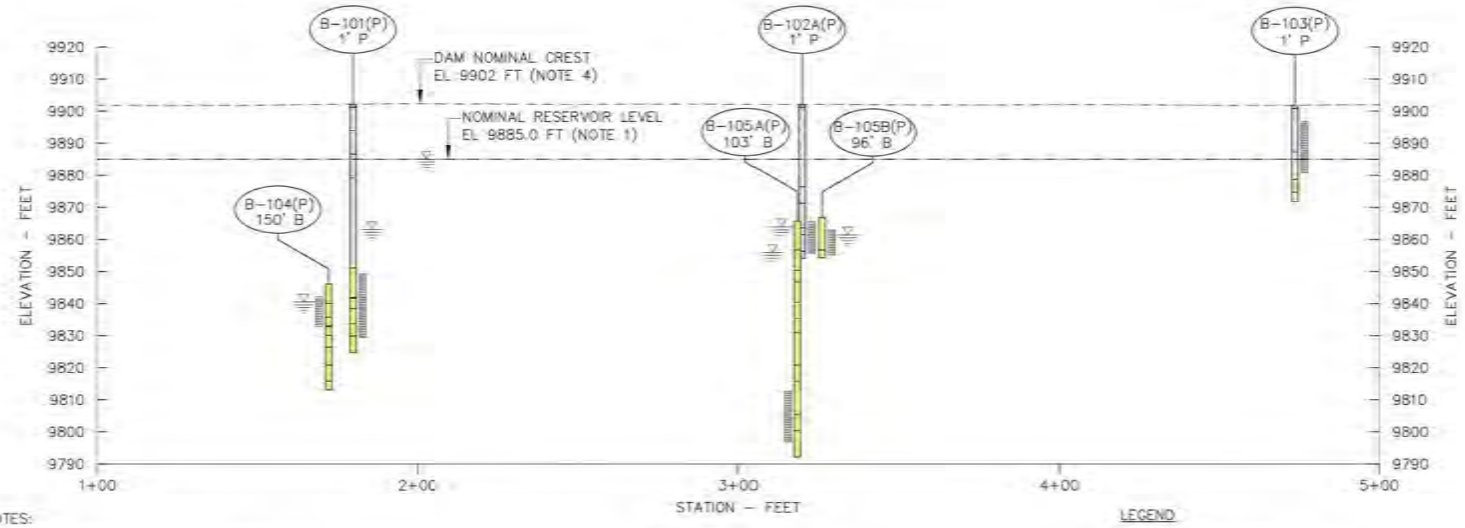


NOTES:

1. NOMINAL RESERVOIR LEVEL DURING FINAL GROUNDWATER LEVEL MEASUREMENT ON 9/22/2018.
2. B-103(P) WELL WAS DRY AT FINAL GROUNDWATER LEVEL MEASUREMENT.
3. ALL GEOLOGICAL CONTACTS ARE APPROXIMATE AND NOT ALL INFORMATION IS SHOWN. BORING LOGS SHOWN GENERALIZE SUBSURFACE CONDITIONS AND ARE NOT INTENDED TO BE A COMPLETE OR ACCURATE DEPICTION OF THE SUBSURFACE CONDITIONS. COMPLETE BORING LOGS ARE PROVIDED IN APPENDIX B.
4. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
5. GEOMETRY OF EMBANKMENT ZONES, OUTLET WORKS, TOE DRAIN, AND CUTOFF TRENCH NOT SHOWN.
6. BORING B-102B NOT SHOWN BECAUSE DRILLED STRATIGRAPHIC UNITS ARE THE SAME AS B-102A(P).

PROFILE

0 15 30 60 90

SCALE IN FEET

LEGEND

- ROAD BASE
- EMBANKMENT FILL
- Qc COLLUVIUM
- MONITORING WELL SENSING ZONE
- GROUNDWATER LEVEL MEASURED FROM MONITORING WELL ON 9/22/2018
- B-101(P) 1' P — MONITORING WELL NAME; DISTANCE PROJECTED (P) OR BEYOND (B) SECTION LOCATION

Grand Junction

RJH CONSULTANTS, INC.

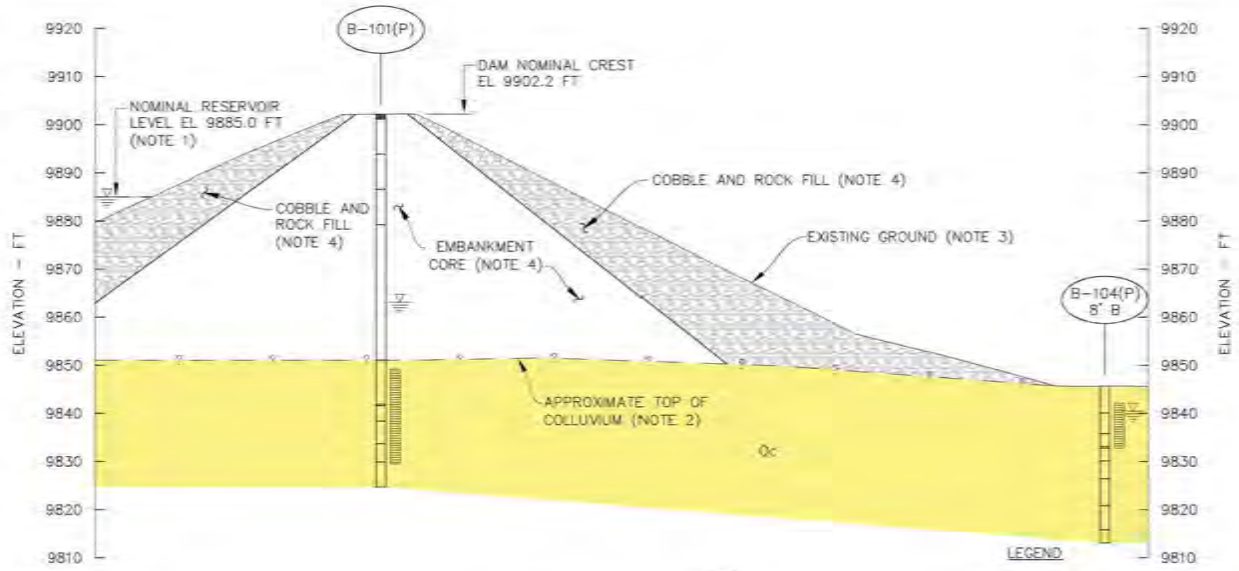
HOGCHUTE DAM
SAFETY EVALUATION
GEOTECHNICAL DATA REPORT

DAM PROFILE

PROJECT NO. 18115

January 2019

Figure 4.2



SECTION A 1.2

0 10 20 40 60

SCALE IN FEET

LEGEND

- ROAD BASE
- EMBANKMENT FILL
- Qc COLLUVIUM
- MONITORING WELL SENSING ZONE
- GROUNDWATER LEVEL MEASURED FROM MONITORING WELL ON 9/22/2018
- B-101(P) 1' P — MONITORING WELL NAME; DISTANCE PROJECTED (P) OR BEYOND (B) SECTION LOCATION

NOTES:

1. NOMINAL RESERVOIR LEVEL DURING FINAL GROUNDWATER LEVEL MEASUREMENT ON 9/22/2018.
2. ALL GEOLOGICAL CONTACTS ARE APPROXIMATE AND NOT ALL INFORMATION IS SHOWN. BORING LOGS SHOWN GENERALIZE SUBSURFACE CONDITIONS AND ARE NOT INTENDED TO BE A COMPLETE OR ACCURATE DEPICTION OF THE SUBSURFACE CONDITIONS. COMPLETE BORING LOGS ARE PROVIDED IN APPENDIX B.
3. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
4. GEOMETRY OF EMBANKMENT ZONES WAS APPROXIMATED USING DATA FROM DESIGN DRAWINGS. OUTLET WORKS, TOE DRAIN, AND CUTOFF TRENCH NOT SHOWN.

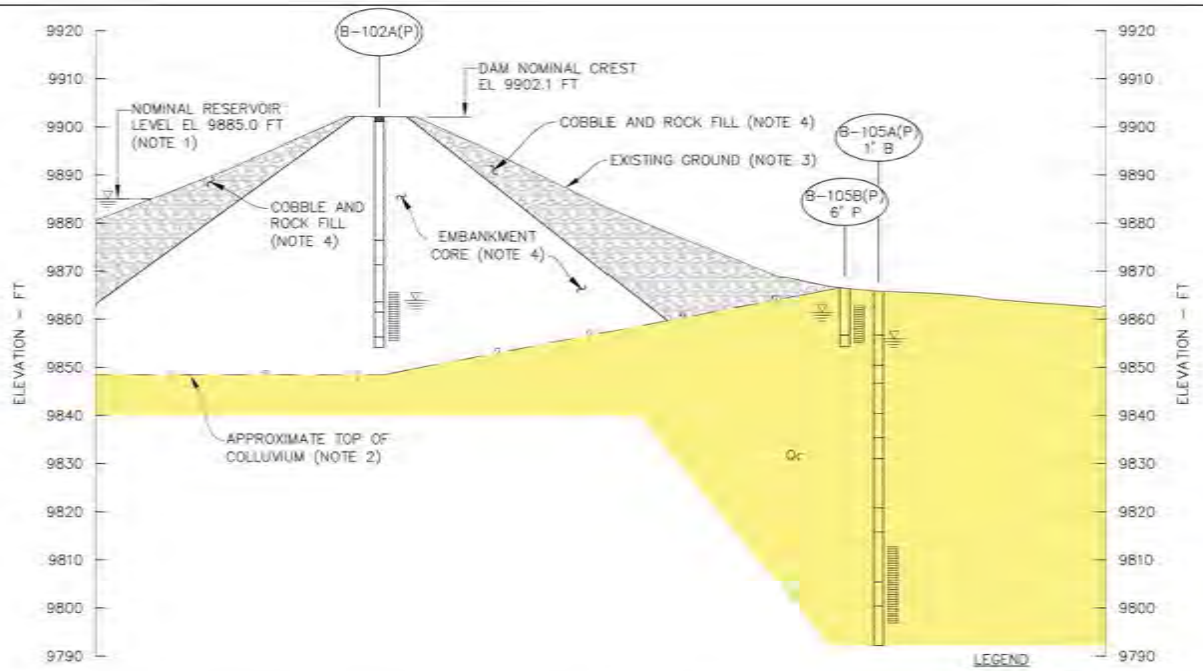
Grand Junction COLORADO

RJH CONSULTANTS, INC.

HOGCHUTE DAM
SAFETY EVALUATION
GEOTECHNICAL DATA REPORT

DAM SECTION A
SOUTH

PROJECT NO. 18115 | January 2019 | Figure 4.3



NOTES:

1. NOMINAL RESERVOIR LEVEL DURING FINAL GROUNDWATER LEVEL MEASUREMENT ON 9/22/2018.
2. ALL GEOLOGICAL CONTACTS ARE APPROXIMATE AND NOT ALL INFORMATION IS SHOWN. BORING LOGS SHOWN GENERALIZE SUBSURFACE CONDITIONS AND ARE NOT INTENDED TO BE A COMPLETE OR ACCURATE DEPICTION OF THE SUBSURFACE CONDITIONS. COMPLETE BORING LOGS ARE PROVIDED IN APPENDIX B.
3. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
4. GEOMETRY OF EMBANKMENT ZONES WAS APPROXIMATED USING DATA FROM DESIGN DRAWINGS. OUTLET WORKS, TOE DRAIN, AND CUTOFF TRENCH NOT SHOWN.
5. BORING B-102B NOT SHOWN BECAUSE DRILLED STRATIGRAPHIC UNITS ARE THE SAME AS B-102A(P). BORING B-102B TERMINATED SHALLOW ON A ROCK.

SECTION B 1.2

0 10 20 40 60
SCALE IN FEET

LEGEND
- ROAD BASE
- EMBANKMENT FILL
- Qc COLLUVIUM
- MONITORING WELL SENSING ZONE
- GROUNDWATER LEVEL MEASURED FROM MONITORING WELL ON 9/22/2018
- B-101(P) 1' P — MONITORING WELL NAME; DISTANCE PROJECTED (P) OR BEYOND (B) SECTION LOCATION

Grand Junction

RJH CONSULTANTS, INC.

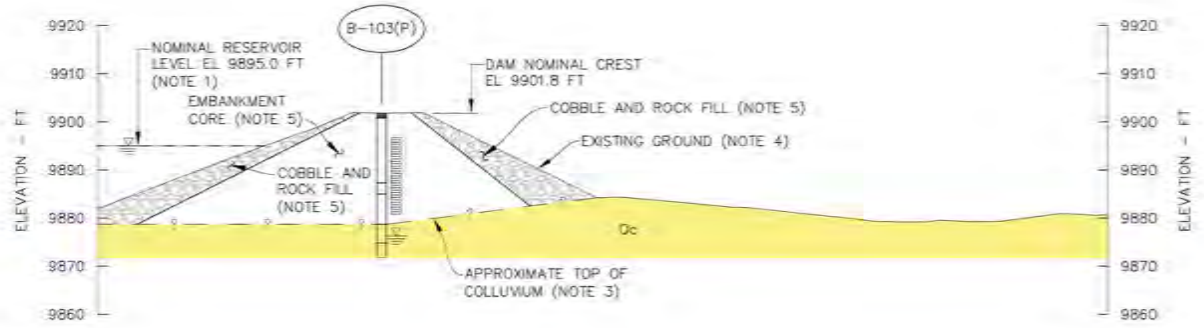
HOGCHUTE DAM
SAFETY EVALUATION
GEOTECHNICAL DATA REPORT

DAM SECTION B
MIDDLE

PROJECT NO. 18115

January 2019

Figure 4.4



SECTION C 1.2

0 10 20 40 60

SCALE IN FEET

NOTES:

1. NOMINAL RESERVOIR LEVEL DURING DRILLING ON 7/23/2018.
2. B-103(P) WELL WAS DRY DURING FINAL GROUNDWATER LEVEL MEASUREMENT FROM WELL ON 9/22/2018. GROUNDWATER LEVEL SHOWN WAS MEASURED DURING DRILLING ON 7/23/2018.
3. ALL GEOLOGICAL CONTACTS ARE APPROXIMATE AND NOT ALL INFORMATION IS SHOWN. BORING LOGS SHOWN GENERALIZE SUBSURFACE CONDITIONS AND ARE NOT INTENDED TO BE A COMPLETE OR ACCURATE DEPICTION OF THE SUBSURFACE CONDITIONS. COMPLETE BORING LOGS ARE PROVIDED IN APPENDIX B.
4. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
5. GEOMETRY OF EMBANKMENT ZONES WAS APPROXIMATED USING DATA FROM DESIGN DRAWINGS. OUTLET WORKS, TOE DRAIN, AND CUTOFF TRENCH NOT SHOWN.

LEGEND

- ROAD BASE
- EMBANKMENT FILL
- Qc COLLUVIUM
- MONITORING WELL SENSING ZONE
- GROUNDWATER LEVEL MEASURED FROM MONITORING WELL ON 7/23/2018
- B-101(P) MONITORING WELL NAME

Grand Junction	RJH CONSULTANTS, INC.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	DAM SECTION C NORTH
		PROJECT NO. 18115	January 2019 Figure 4.5

SECTION 5 - LIMITATIONS

This Report has been prepared for the exclusive use of RJH, the City of Grand Junction, and the SEO to support evaluation of potential dam safety issues at Hogchute Dam. RJH is not responsible for technical interpretations of this data by others. RJH has endeavored to conduct our professional services for this Project in a manner consistent with a level of care and skill ordinarily exercised by members of the engineering profession currently practicing in Colorado under similar conditions as this Project. RJH makes no other warranty, expressed or implied.

The methods used in this study indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot be relied on to accurately reflect variations in subsurface conditions that may exist between sampling locations.

SECTION 6 - REFERENCES

Ellis, M.S. and Gabaldo, V. (1989). *Geologic Map and Cross Sections of Parts of the Grand Junction and Delta 30' x 60' Quadrangles, West-Central Colorado.* U.S. Geological Survey Coal Investigations Map C-124.

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APPENDIX A

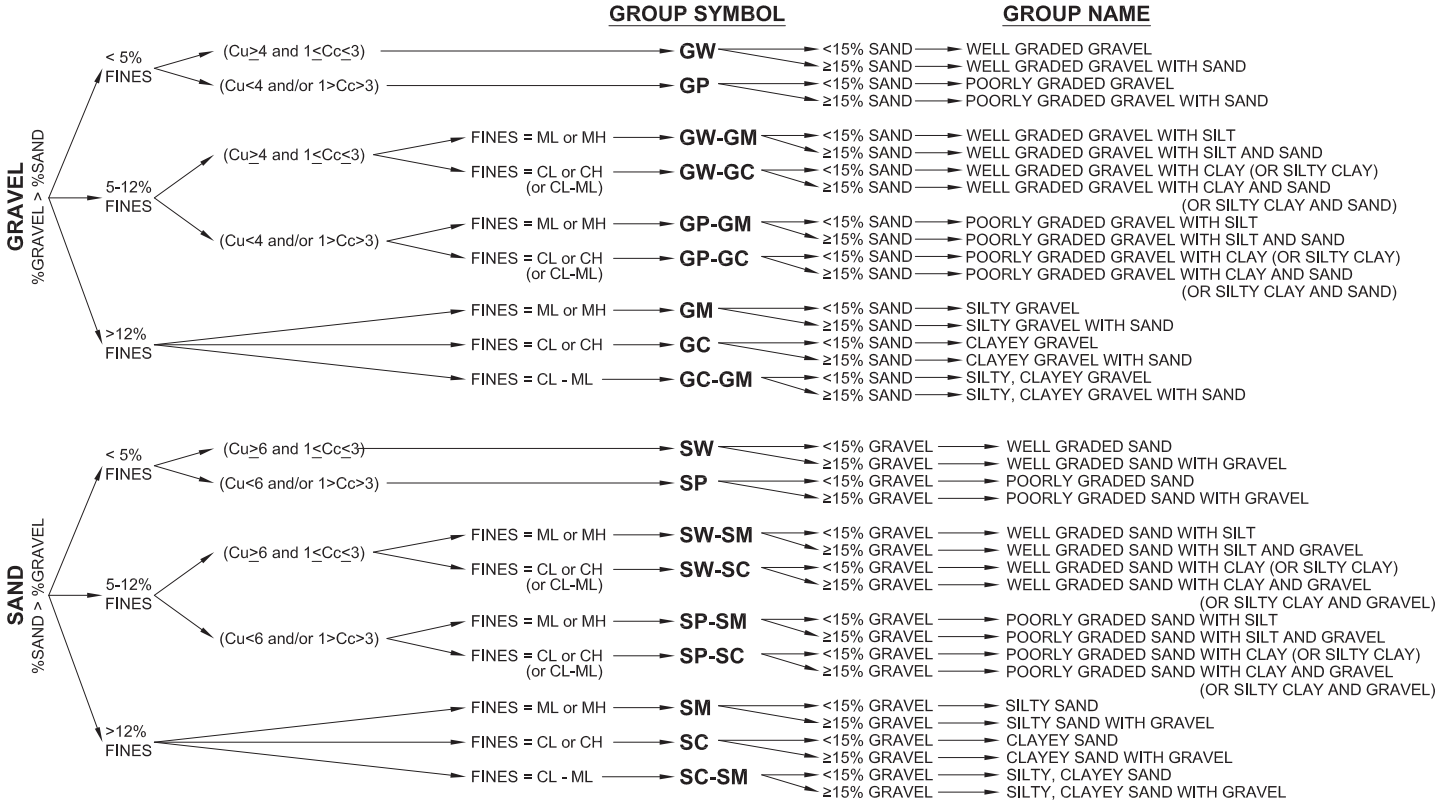
SOIL DESCRIPTORS

ABBREVIATIONS

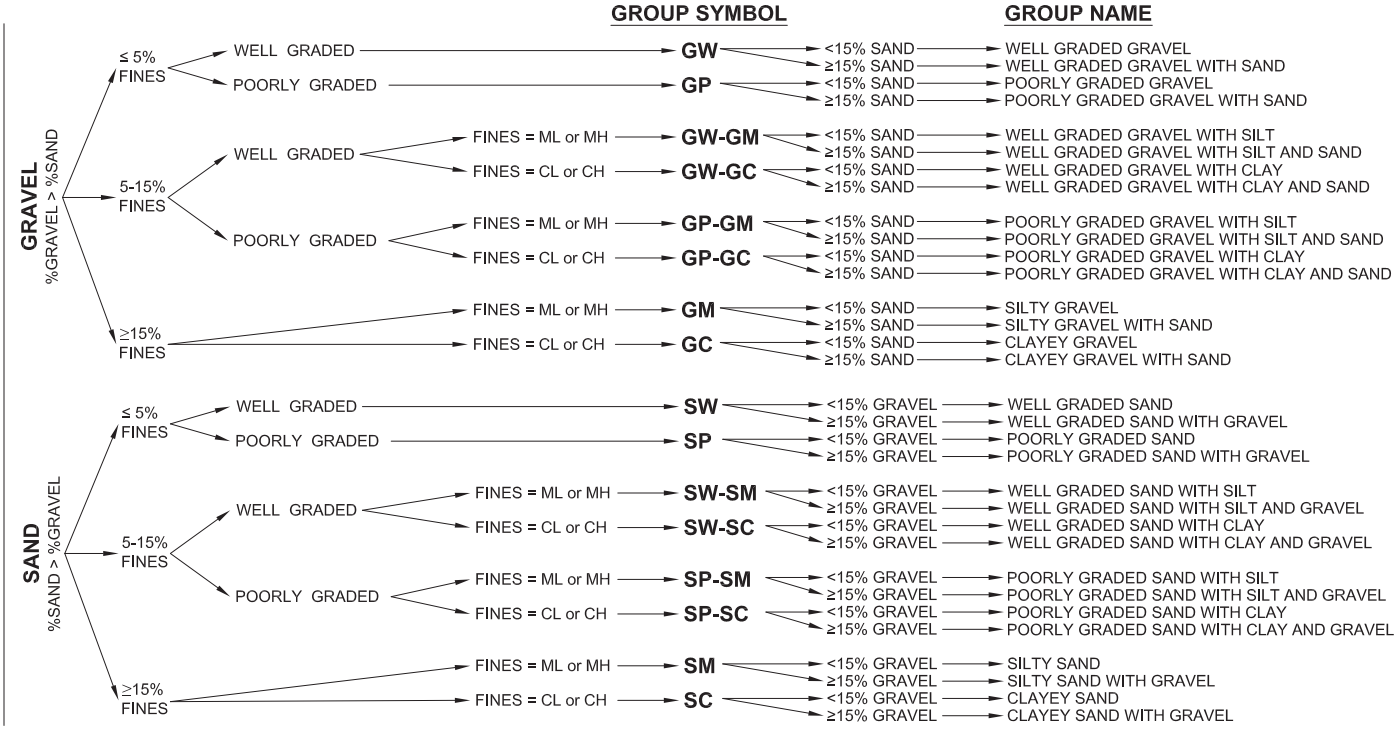
Bu	Bulk sample
CA	2.0-inch I.D. ring-lined split barrel California sample
DM	2.5-inch I.D. ring-lined split barrel Dames and Moore (modified California) sample
RQD	Rock Quality Designation
S	1.375-inch I.D. standard split-spoon sample (unlined)
U	Shelby Tube sample

COARSE GRAINED SOILS
(< 50% FINES)

A) FLOWCHART APPLIED TO LABORATORY TESTED SOIL SAMPLES.
ADAPTED FROM ASTM D 2487 *CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES (USCS)*.



B) FLOWCHART APPLIED TO FIELD CLASSIFIED SOIL SAMPLES.
ADAPTED FROM ASTM D 2488 *DESCRIPTION AND IDENTIFICATION OF SOILS (VISUAL-MANUAL PROCEDURE)*.



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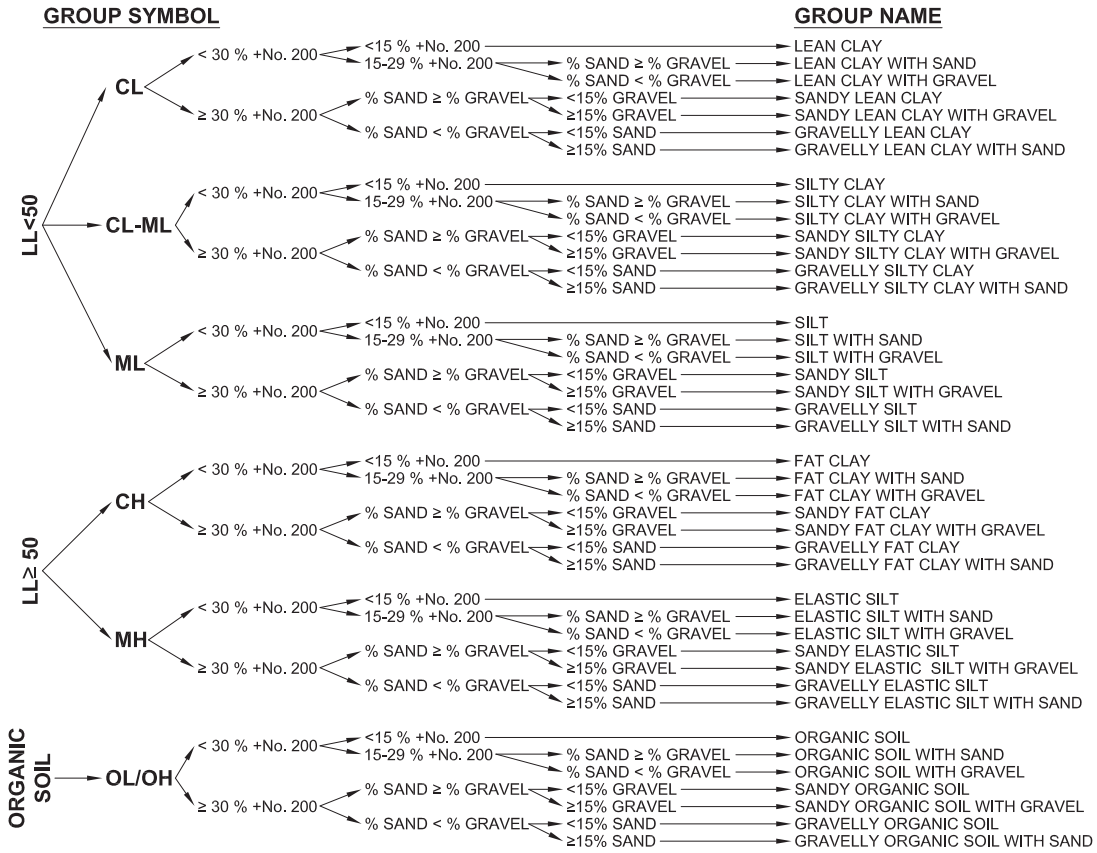
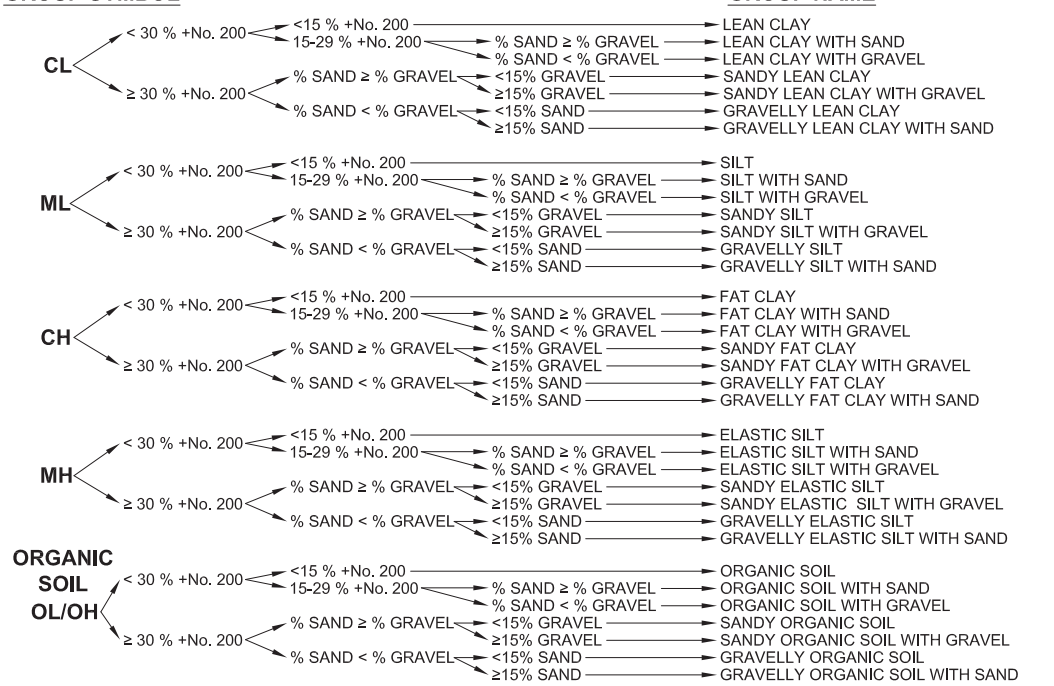
FINE GRAINED SOILS
(≥ 50% FINES)

A) FLOWCHART APPLIED TO LABORATORY TESTED SOIL SAMPLES.
ADAPTED FROM ASTM D 2487 *CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES (USCS)*.



B) FLOWCHART APPLIED TO FIELD CLASSIFIED SOIL SAMPLES.
ADAPTED FROM ASTM D 2488 *DESCRIPTION AND IDENTIFICATION OF SOILS (VISUAL-MANUAL PROCEDURE)*.



NOTE:

1. **THE PLASTICITY CHART ON THE FOLLOWING PAGE WAS USED TO IDENTIFY THE GROUP SYMBOL FOR FLOWCHART A. A COMBINATION OF THE VISUAL MANUAL CRITERIA ON THE FOLLOWING PAGE WERE USED TO IDENTIFY THE GROUP SYMBOL FOR FLOWCHART B.**

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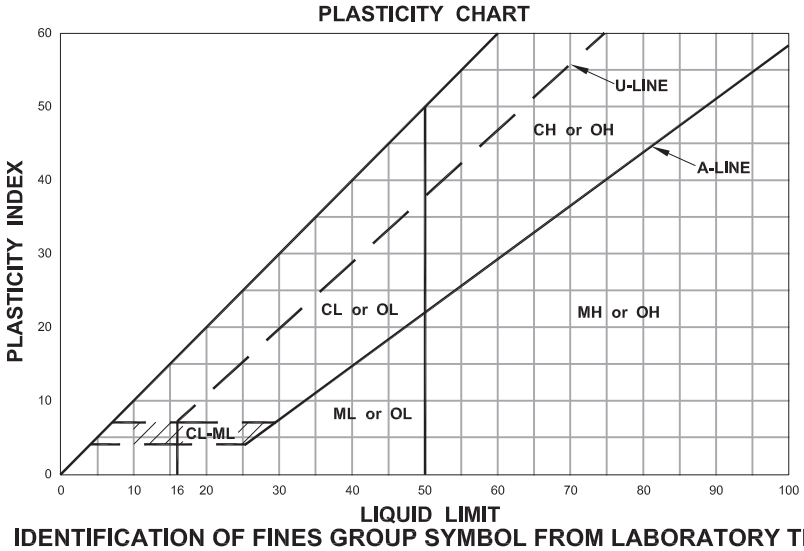
SOIL PLASTICITY CHARACTERISTICS



A) IDENTIFICATION OF FINES GROUP SYMBOL FROM LABORATORY TESTS.
REPRODUCED FROM ASTM D 2487 *CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES (USCS)*.

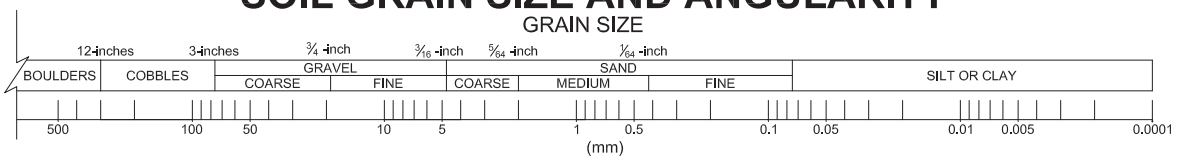
B) IDENTIFICATION OF FINES GROUP SYMBOL FROM VISUAL-MANUAL CRITERIA.
REPRODUCED FROM ASTM D 2488 *DESCRIPTION AND IDENTIFICATION OF SOILS (VISUAL-MANUAL PROCEDURE).*

DRY STRENGTH	
DESCRIPTION	CRITERIA
NONE	CRUMBLES TO POWDER WHILE HANDLING.
LOW	CRUMBLES TO POWDER WITH SOME FINGER PRESSURE.
MEDIUM	BREAKS INTO PIECES OR CRUMBLES WITH CONSIDERABLE FINGER PRESSURE.
HIGH	CANNOT BE BROKEN WITH FINGER PRESSURE. BREAKS INTO PIECES BETWEEN THUMB AND HARD SURFACE.
VERY HIGH	CANNOT BE BROKEN BETWEEN THUMB AND HARD SURFACE.
DILATANCY (RESISTANCE TO SHAKING)	
DESCRIPTION	CRITERIA
NONE	NO VISIBLE CHANGE IN SPECIMEN.
SLOW	WATER APPEARS SLOWLY ON THE SURFACE OF THE SPECIMEN DURING SHAKING AND DOES NOT DISAPPEAR OR DISAPPEARS SLOWLY UPON SQUEEZING.
RAPID	WATER APPEARS QUICKLY ON THE SURFACE OF THE SPECIMEN DURING SHAKING AND DISAPPEARS QUICKLY UPON SQUEEZING.

TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	
DESCRIPTION	CRITERIA
LOW	ONLY SLIGHT PRESSURE IS REQUIRED TO ROLL THE THREAD. THREAD AND LUMP ARE WEAK AND SOFT.
MEDIUM	MEDIUM PRESSURE IS REQUIRED TO ROLL THE THREAD. THREAD AND LUMP HAVE MEDIUM STIFFNESS.
HIGH	CONSIDERABLE EFFORT IS REQUIRED TO ROLL THE THREAD. THREAD AND LUMP HAVE HIGH STIFFNESS.
PLASTICITY	
DESCRIPTION	CRITERIA FOR A ⅛-INCH (3 mm) THREAD.
NON-PLASTIC	THREAD CANNOT BE ROLLED.
LOW	THREAD CAN BARELY BE ROLLED AND THE LUMP CANNOT BE FORMED WHEN DRIER THAN THE PLASTIC LIMIT.
MEDIUM	THREAD IS EASY TO ROLL AND NOT MUCH TIME IS REQUIRED TO REACH THE PLASTIC LIMIT. THE THREAD CANNOT BE RE-ROLLED SEVERAL TIMES AFTER REACHING THE PLASTIC LIMIT. THE LUMP CRUMBLES WHEN DRIER THAN THE PLASTIC LIMIT.
HIGH	IT TAKES CONSIDERABLE TIME ROLLING AND KNEADING TO REACH THE PLASTIC LIMIT. THE THREAD CAN BE RE-ROLLED SEVERAL TIMES AFTER REACHING THE PLASTIC LIMIT. THE LUMP CAN BE FORMED WITHOUT CRUMBLING WHEN DRIER THAN THE PLASTIC LIMIT.

SYMBOL	DRY STRENGTH	DILATANCY	TOUGHNESS AND PLASTICITY	PLASTICITY
ML	NONE - LOW	SLOW - RAPID	LOW	LOW TO NON-PLASTIC
CL	MEDIUM - HIGH	NONE - SLOW	MEDIUM	LOW TO MEDIUM
MH	LOW - MEDIUM	NONE - SLOW	LOW TO MEDIUM	LOW TO MEDIUM
CH	HIGH - VERY HIGH	NONE	HIGH	HIGH

SOIL GRAIN SIZE AND ANGULARITY



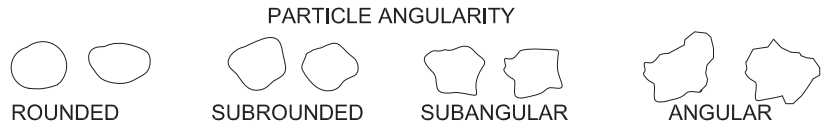


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TABLE 1.1
CRITERIA FOR DESCRIBING SOIL STRUCTURE(1)

Description	Criteria
Stratified	Alternating layers of varying material or color with layers greater than or equal to 1/4 inch thick (6 mm)
Laminated	Alternating layers of varying material or color with layers less than 1/4 inch thick (6 mm)
Fissured	Breaks along definite plates of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay
Homogeneous	Same color and appearance throughout

Note:
1. Modified from ASTM D 2488 *Description and Identification of Soils (Visual-Manual Procedure)* and differ from the U.S. Bureau of Reclamation *Engineering Geology Field Manual* (2001).

TABLE 1.2
RELATIVE DENSITY OF SANDS ACCORDING TO RESULTS OF STANDARD PENETRATION TEST(1)

Number of Blows N	Relative Density
0-4	Very Loose
5-10	Loose
11-30	Medium
31-50	Dense
Over 50	Very Dense

Note:
1. Modified from Terzaghi, Peck, and Mesri (1996).

TABLE 1.3
GUIDE FOR STIFFNESS OF FINE-GRAINED SOILS(1)

Description	Criteria	Estimated Unconfined Compressive Strength (TSF)
Very Soft	Extrudes between fingers when squeezed	<0.25
Soft	Molded by light finger pressure	0.25-0.50
Medium	Molded by strong finger pressure	0.50-1.00
Stiff	Readily indented by thumb or penetrated with great effort	1.00-2.00
Very Stiff	Readily indented by thumbnail	2.00-4.00
Hard	Indented with difficulty by thumbnail	>4.00

Note:
1. Reproduced from NAVFAC (1986).

TABLE 1.4
CRITERIA FOR DESCRIBING SOIL MOISTURE CONDITION[1]

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below the water table

Note:
1. Reproduced from ASTM 2488 *Description and Identification of Soils (Visual-Manual Procedure)*.

TABLE 1.5
CRITERIA FOR DESCRIBING SOIL CEMENTATION[1][2]

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

Notes:
1. Reproduced from ASTM 2488 *Description and Identification of Soils (Visual-Manual Procedure)*.
2. The absence of cementation was not recorded on boring logs.

TABLE 1.6
CRITERIA FOR DESCRIBING SOIL REACTION WITH HCL[1]

Description	Criteria
None[2]	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

Notes:
1. Reproduced from ASTM 2488 Description and Identification of Soils (Visual-Manual Procedure).
2. The absence of a reaction was not recorded on boring logs.

ASTM D 2487 (2011). *Standard Classification of Soils for Engineering Purposes (USCS)*. June.

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Terzaghi, Karl, Peck, Ralph B., and Mesri, Gholamreza. (Terzaghi, Peck, and Mesri). (1996). *Soil Mechanics in Engineering Practice*.

U.S. Bureau of Reclamation (USBR) (2001). *Engineering Geology Field Manual*.

APPENDIX B

BORING LOGS

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995296, Long: -108.109759 deg
Ground El: 9902.2 ft Total Depth: 77.5 ft
Groundwater El: 9878.0 ft On Date: 07-26-2018

Start Date: 07-25-2018 End Date: 07-28-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-101(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
								0.0 to 1.0 ft: Road Base; [--]
9901.2	1					Grinding and minor rig rocking from 1.0 to 14.0 feet.		Bu-1, S-2: Clayey Gravel with Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 20-35% fines, medium plasticity; 15-30% sand, fine to coarse grained, subangular to subrounded; less than 5% cobbles; maximum particle size = 4 inches; dense; moist; dark brown; (GC); [Fill]
	2					Sample Bu-1 collected from 2.0 to 10.0 feet.		
	3							
	4							
	5							*5.0 to 6.5 ft: 20-35% sand; 15-30% fines;*
	6	S - 2	7/13/21	1.5	1.6			
	7							
9893.9	8							
	9							S-3, S-4, U-5: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 20-35% fines, medium plasticity; 15-25% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.75 inches; medium dense; moist; dark brown; (SC); [Fill]
	10							
	11	S - 3	5/10/12	1.5	1.6			
	12							
	13	S - 4	7/11/10	1.5	1.9			
	14					Smooth augering from 14.0 to 16.0 feet.		
	15	U - 5		0.6		U-5 disturbed, gravel/cobble damaged sampler.		
9886.6	16					Sample Bu-6 collected from 15.0 to 25.0 feet. Grinding and minor rig rocking from 16.0 to 36.5 feet.		Bu-6, S-7, CA-8: Clayey Gravel with Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 20-30% sand, fine to coarse grained, subangular to subrounded; 15-30% fines, medium plasticity; maximum particle size = 2.25 inches; dense; moist; dark brown; (GC); [Fill]
	17							
	18	S - 7	11/15/20	1.5	0.6			
	19							
	20							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the dam foundation.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995296, Long: -108.109759 deg
Ground El: 9902.2 ft Total Depth: 77.5 ft
Groundwater El: 9878.0 ft On Date: 07-26-2018

Start Date: 07-25-2018 End Date: 07-28-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-101(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	21	CA - 8	35/40	1.0	1.0	CA-8 disturbed by gravel/ cobble.		
	22							
9879.2	23							S-9, S-10, CA-11, S-12, Bu-13, S-14, Bu-15, CA-16, U-17, S-18, S-19: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 20-45% fines, medium plasticity; 20-35% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; medium dense to dense; moist to wet; dark brown; (SC); [Fill]
	24					Groundwater encountered at 24.2 feet during drilling on 7/26/2018.		
	25	S - 9	8/13/18	1.5	1.6	Sample Bu-13 collected from 25.0 to 40.0 feet.		
	26							
	27							
	28	S - 10	9/10/23	1.5	1.5			
	29							
	30	CA - 11	10/15	1.0	1.0			*30.0 to 31.0 ft: PP = 4 tsf;*
	31							
	32							
	33							
	34					Cuttings and samples are moist, no longer wet, from 34.0 to 55.3 feet. Potential perched water table at 24.2 feet.		
	35							*35.0 to 36.5 ft: 15-30% fines;*
	36	S - 12	30/21/17	1.5	1.5			
	37					Grinding with occasional periods of smooth augering from 36.5 to 64.5 feet.		
	38							
	39					Groundwater encountered at 38.3 feet on 7/27/2018 a.m. after drilling to 75.0 feet the previous day.		
	40							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the dam foundation.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995296, Long: -108.109759 deg
Ground El: 9902.2 ft Total Depth: 77.5 ft
Groundwater El: 9878.0 ft On Date: 07-26-2018

Start Date: 07-25-2018 End Date: 07-28-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-101(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	41	S - 14	6/10/25	1.5	1.6			
	42							
	43	CA - 16	13/28	1.0	1.0			*42.5 to 43.5 ft: PP = 4.5 tsf;*
	44							
	45	U - 17		0.3	0.3	U-17 disturbed, gravel/cobble damaged sampler.		
	46	S - 18	9/17/20	1.5	1.7	Sample Bu-15 collected from 45.0 to 50.0 feet.		
	47							
	48							
	49							
	50							
	51	S - 19	11/23/20	1.5	1.6	Bottom of embankment fill at approximately 51.1 feet.		
9851.1								S-19, Bu-20, U-21, S-22: Sandy Lean Clay with Gravel Mostly fines, low to medium plasticity; 20-35% sand, fine to coarse grained, subangular to subrounded; 15-25% gravel, fine to coarse grained; maximum particle size = 2 inches; soft to medium stiff; moist; brown-gray; gravel composed of mostly basalt; (CL); [Colluvium]
	52					Sample Bu-20 collected from 51.0 to 65.0 feet.		
	53							
	54							
	55	U - 21		0.3	0.3	U-21 disturbed, gravel/cobble damaged sampler.		
	56					Groundwater encountered at 55.3 feet during drilling on 7/26/2018.		
	57							
	58							
	59							
	60							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the dam foundation.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995296, Long: -108.109759 deg
Ground El: 9902.2 ft Total Depth: 77.5 ft
Groundwater El: 9878.0 ft On Date: 07-26-2018

Start Date: 07-25-2018 End Date: 07-28-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-101(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
9841.9								
9841.6	61	S - 22	8/23/31	1.5	1.5			S-22: Poorly Graded Sand with Clay and Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 15-30% gravel, fine grained, subangular to subrounded; 5-15% fines, low to medium plasticity; maximum particle size = 0.5 inches; very dense; wet; dark brown-black; (SP-SC); [Colluvium]
	62							S-22: Sandy Lean Clay with Gravel Mostly fines, medium plasticity; 20-35% gravel, fine to coarse grained, subangular to subrounded; 15-25% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 1.75 inches; soft to medium stiff; moist; brown-gray; (CL); [Colluvium]
	63							
9838.4	64							Bu-23: Lean Clay with Sand Mostly fines, medium plasticity; 5-15% sand, fine to coarse grained, subangular to subrounded; less than 5% gravel, fine grained; maximum particle size = 0.2 inches; very soft; wet; brown-gray; (CL); [Colluvium]
	65					Grinding and minor rig rocking from 64.5 to 76.5 feet.		
	66					Sample Bu-23 collected from 66.0 to 67.0 feet.		
	67							
	68							
9833.7	69							S-24: Lean Clay Mostly fines, medium plasticity; 5-15% sand, fine to coarse grained, subangular to subrounded; soft to medium stiff; moist to wet; dark brown-gray; (CL); [Colluvium]
	70	S - 24	4/14/18	1.5	1.5			
	71							
	72							
9829.9	73	S - 25	10/41/35	1.5	1.6			S-25, S-26: Sandy Lean Clay Mostly fines, medium plasticity; 10-25% sand, fine to coarse grained, subangular to subrounded; 5-15% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; medium stiff; moist; dark brown-gray; (CL); [Colluvium]
	74							
	75							
	76	S - 26	11/32/50 for 5 inches	1.4	1.5	Auger encountered refusal at 76.5 feet.		
	77							
9824.7	78					Bottom of boring at 77.5 feet.		End of boring log at 77.50 ft
	79							
	80							

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the dam foundation.

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LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995677, Long: -108.109800 deg
Ground El: 9902.1 ft Total Depth: 48.0 ft
Groundwater El: 9860.6 ft On Date: 07-24-2018

Start Date: 07-24-2018 End Date: 07-26-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-102A(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
9901.1	1					Continuous grinding and occasional rig rocking from 0.5 to 6.0 feet.		0.0 to 1.0 ft: Road Base; [--]
	2	S - 2	4/7/9	1.5	1.1	Sample Bu-1 collected from 2.0 to 18.0 feet.		Bu-1, S-2, S-3, S-4, S-5, S-6, S-7, S-8, CA-9, Bu-10, S-11, S-12: Clayey Gravel with Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 20-35% sand, fine to coarse grained, subangular to subrounded; 20-30% fines, medium plasticity; maximum particle size = 3 inches; medium dense; moist; dark brown; (GC); [Fill]
	3							
	4	S - 3	16/7/30	1.5	1.5			*4.0 to 5.5 ft: dense;*
	5							
	6					Smooth augering from 6.0 to 7.5 feet.		
	7	S - 4	11/15/14	1.5	1.1	Continuous grinding from 7.5 to 11.0 feet.		
	8							
	9	S - 5	7/9/11	1.5	1.5			
	10							
	11					Smooth augering from 11.0 to 14.0 feet.		
	12	S - 6	9/8/10	1.5	1.7			
	13							
	14					Grinding from 14.0 to 16.0 feet.		
	15	S - 7	7/10/15	1.5	1.5			
	16					Smooth augering from 16.0 to 17.5 feet.		
	17	S - 8	8/21/33	1.5	0.8	Continuous grinding and occasional rig rocking from 17.5 to 21.0 feet.		*16.5 to 18.0 ft: very dense;*
	18					Sample Bu-10 collected from 18.0 to 41.5 feet.		*18.0 to 23.0 ft: 20-49% fines, low to medium plasticity;*
	19	CA - 9	34/50 for 2 inches	0.7	0.5	CA-9 disturbed by gravel/ cobble.		
	20							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the embankment.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995677, Long: -108.109800 deg
Ground El: 9902.1 ft Total Depth: 48.0 ft
Groundwater El: 9860.6 ft On Date: 07-24-2018

Start Date: 07-24-2018 End Date: 07-26-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-102A(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	21					Smooth augering from 21.0 to 22.0 feet.		
	22	S - 11	12/19/30	1.5	1.5	Continuous grinding and minor rig rocking from 22.0 to 26.0 feet.		*21.5 to 23.0 ft: dense;*
	23							
	24	S - 12	26/50 for 3 inches	0.8	0.8			*24.0 to 24.8 ft: very dense; brown-gray;*
	25							
9876.4	26					Smooth augering from 26.0 to 27.0 feet.		S-13, CA-14: Clayey Sand Mostly sand, fine to coarse grained, subangular to subrounded; 20-30% fines, medium plasticity; 15-25% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1 inch; dense; moist; dark brown; (SC); [Fill]
	27	S - 13	10/19/21	1.5	1.1	Minor grinding from 27.0 to 30.0 feet.		
	28							
	29	CA - 14	59/60	1.0	1.0			*29.0 to 30.0 ft: PP = 2.75 tsf;*
	30					Smooth augering from 30.0 to 33.0 feet.		
9871.3	31							S-15, S-16, S-17: Clayey Gravel with Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 20-35% sand, fine to coarse grained, subangular to subrounded; 20-30% fines, medium plasticity; maximum particle size = 1.25 inches; dense; moist; dark brown; (GC); [Fill]
	32	S - 15	13/16/15	1.5	1.7			
	33					Continuous grinding and minor rig rocking from 33.0 to 48.0 feet.		
	34							
	35	S - 16	13/21/28	1.5	1.0			
	36							
	37	S - 17	7/14/21	1.5	1.6			
	38							
9863.6	39							S-18: Poorly Graded Gravel with Clay and Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 15-25% sand, fine to coarse grained, subangular to subrounded; 5-15% fines, medium plasticity; maximum particle size = 1.5 inches; very dense; moist;
	39	S - 18	52/50 for 4 inches	0.8	0.6	Shelby tube attempt had no recovery, gravel/cobble damaged sampler. Drove split spoon at same depth.		
	40							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the embankment.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995677, Long: -108.109800 deg
Ground El: 9902.1 ft Total Depth: 48.0 ft
Groundwater El: 9860.6 ft On Date: 07-24-2018

Start Date: 07-24-2018 End Date: 07-26-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-102A(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
9861.5	41							brown-gray; (GP-GC); [Fill]
	42	S - 19	12/10/11	1.5	0.3	Groundwater encountered at 41.5 feet during drilling on 7/24/2018.		S-19, CA-20: Clayey Sand Mostly sand, fine to coarse grained, subangular to subrounded; 25-40% fines, medium plasticity; 20-35% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 3 inches; medium dense; wet; dark brown; (SC); [Fill]
	43							
	44	CA - 20	15/22	1.0	1.2			*44.0 to 45.0 ft: PP = 3.5 tsf;*
	45							
9856.3	46							S-21: Poorly Graded Gravel with Clay and Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 15-30% sand, fine to coarse grained, angular to subrounded; 5-15% fines, medium plasticity; maximum particle size = 1 inch; very dense; wet; black; gravels composed of mostly basalt; (GP-GC); [Fill]
	47	S - 21	50 for 3 inches	0.3	0.3			
9854.1	48					Augers encountered refusal at 48.0 feet. Bottom of boring at 48.0 feet.		End of boring log at 48.00 ft
	49							
	50							
	51							
	52							
	53							
	54							
	55							
	56							
	57							
	58							
	59							
	60							

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the embankment.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995667, Long: -108.109796 deg
Ground El: 9902.1 ft Total Depth: 5.0 ft
Groundwater El: Not Encountered On Date: 07-26-2018

Start Date: 07-25-2018 End Date: 07-26-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-102B**
Sheet 1 of 1

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
9901.1	0							0.0 to 1.0 ft: Road Base; [--]
	1					Continuous grinding and rig rocking.		Bu-1: Clayey Gravel with Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 20-35% fines, medium plasticity; 15-30% sand, fine to coarse grained, subangular to subrounded; less than 5% cobbles; maximum particle size = 4 inches; moist; dark brown; (GC); [Fill]
	2					Sample Bu-1 collected from 2.0 to 5.0 feet.		
	3							
	4							
9897.1	5					Auger encountered refusal at 5.0 feet. Bottom of boring at 5.0 feet.		End of boring log at 5.00 ft
	6							
	7							
	8							
	9							
	10							
	11							
	12							
	13							
	14							
	15							
	16							
	17							
	18							
	19							
	20							

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was backfilled with cement-bentonite grout.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.996095, Long: -108.109881 deg
Ground El: 9901.8 ft Total Depth: 30.0 ft
Groundwater El: 9876.3 ft On Date: 07-23-2018

Start Date: 07-23-2018 End Date: 07-24-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-103(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
								0.0 to 1.0 ft: Road Base; [--]
9900.8	1					Sample Bu-1 collected from 1.0 to 7.5 feet. Grinding, minor rig rocking.		Bu-1, S-2, S-3, S-4, S-5, S-6, S-7: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 30-45% fines, medium plasticity; 20-35% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; dense to very dense; moist; dark brown; (SC); [Fill]
	2	S - 2	8/19/34	1.5	1.0			
	3					Augers encountered refusal at 3.0 feet. Backfilled hole with cuttings, moved 3 feet south and continued augering.		
	4	S - 3	29/24/50 for 1 inch	1.1	0.5	At 3.5 feet, changed to conical auger bit to help reduce grinding.		
	5					Minor grinding from 5.0 to 23.1 feet.		
	6	S - 4	5/15/31	1.5	1.4			
	7							
	8	S - 5	22/10/22	1.5	1.5			*7.5 to 9.0 ft: occasional strong reaction with HCl;*
	9							
	10					Sample Bu-11 collected from 10.0 to 17.5 feet.		
	11	S - 6	19/14/30	1.5	1.5			
	12							
	13	S - 7	11/13/29	1.5	1.5			
	14							
9887.3	15							CA-8: Clayey Gravel with Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 30-40% fines, medium plasticity; 15-25% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; moist; dark brown; PP = 1.5 tsf; (GC); [Fill]
	16	CA - 8	15/17	1.0	1.0			
9885.0	17							CA-9, S-10, Bu-11, S-12: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 30-45% fines, medium plasticity; 20-35% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.75 inches; dense to very dense; moist; dark brown; (SC); [Fill]
	18	CA - 9	22/22	1.0	1.0			*17.5 to 18.5 ft: 15-35% fines; PP = 1 tsf;*
	19							
	20							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the embankment.

RJH CONSULTANTS, INC

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.996095, Long: -108.109881 deg
Ground El: 9901.8 ft Total Depth: 30.0 ft
Groundwater El: 9876.3 ft On Date: 07-23-2018

Start Date: 07-23-2018 End Date: 07-24-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 4-1/4" ID, 7-3/4" OD Hollow Stem Auger (HSA)

Borehole ID: **B-103(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	21							
	22	S - 10	12/18/50 for 3 inches	1.3	1.3			
9878.7	23	S - 12	8/4/9	1.5	1.5	Bottom of embankment fill at approximately 23.1 feet. Smooth augering from 23.1 to 30.0 feet.		S-12, S-13: Gravelly Lean Clay with Sand Mostly fines, medium plasticity; 15-30% gravel, fine to coarse grained, subangular to subrounded; 15-25% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 1 inch; soft; moist to wet; brown; (CL); [Colluvium]
	24							
	25	S - 13	2/4/4	1.5	1.3	Groundwater encountered at 25.5 feet during drilling on 7/23/2018.		
	26							
9874.8	27							U-14: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 25-35% fines, medium plasticity; 20-30% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1 inch; moist; brown; (SC); [Colluvium]
	28	U - 14		2.1	2.1			
	29							
9871.8	30					Bottom of boring at 30.0 feet.		End of boring log at 30.00 ft
	31							
	32							
	33							
	34							
	35							
	36							
	37							
	38							
	39							
	40							

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well in the embankment.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995285, Long: -108.110292 deg
Ground El: 9846.1 ft Total Depth: 33.0 ft
Groundwater El: 9835.7 ft On Date: 09-20-2018

Start Date: 09-19-2018 End Date: 09-21-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 5" ID, 5-3/8" OD Symmetrix Drive Casing Advancer

Borehole ID: **B-104(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	1–5					Continuous slow, smooth drilling; dust from cobbles/ boulders from 0 to 6.0 feet.		S-1: Poorly Graded Gravel with Silt and Sand Mostly gravel, fine to coarse grained, subangular to subrounded; 20-35% sand, fine to coarse grained, subangular to subrounded; 5-15% fines, nonplastic; maximum particle size = 1.25 inches; dense; dry; black; gravel composed of mostly basalt; (GP-GM); [Colluvium]
	4–5	S - 1	7/15/18	1.5	0.3			
9840.1	6					At 6.0 feet, driller said material feels like clay.		S-2: Sandy Lean Clay with Gravel Mostly fines, low plasticity; 20-35% sand, fine to coarse grained, subangular to subrounded; 15-30% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; medium stiff; moist; brown; (CL); [Colluvium]
	7–8	S - 2	4/7/9	1.5	1.5	Continuous slow, smooth drilling from 6.0 to 33.0 feet.		
9835.8	10					Groundwater encountered at 10.4 feet during drilling on 9/20/2018.		U-3: Lean Clay with Sand Mostly fines, medium to high plasticity; 5-15% sand, fine to coarse grained, subangular to subrounded; 5-10% gravel, fine grained, subangular to subrounded; maximum particle size = 0.75 inches; stiff; moist; dark gray; (CL); [Colluvium]
	12	U - 3		1.0	1.0	U-3 disturbed, cobble/boulder damaged sampler.		
9833.1	13	S - 4	50 for 3 inches	0.3	0.3			S-4: Gravelly Lean Clay with Sand Mostly fines, medium plasticity; 20-35% gravel, fine to coarse grained, subangular to subrounded; 15-25% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; medium stiff; moist; dark gray; (CL); [Colluvium]
9832.8	14	S - 5	17/28/24	1.5	1.9			
	15–16					Driller said material feels like gravel from 13.3 to 16.0 feet.		S-5: Sandy Lean Clay with Gravel Mostly fines, low plasticity; 20-35% sand, fine to coarse grained, subangular to subrounded; 15-30% gravel, fine to coarse grained, subangular to rounded; maximum particle size = 1.25 inches; stiff; moist; brown; (CL); [Colluvium]
9830.1	17	S - 6	50 for 4 inches	0.3	0.3			S-6: Gravelly Lean Clay with Sand Mostly fines, medium plasticity; 20-35% gravel, fine to coarse grained, subangular to subrounded; 15-25% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; soft; wet; brown to gray; (CL); [Colluvium]
	18–19							
9826.4	20							S-7: Lean Clay with Gravel

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well at the dam toe.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995285, Long: -108.110292 deg
Ground El: 9846.1 ft Total Depth: 33.0 ft
Groundwater El: 9835.7 ft On Date: 09-20-2018

Start Date: 09-19-2018 End Date: 09-21-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 5" ID, 5-3/8" OD Symmetrix Drive Casing Advancer

Borehole ID: **B-104(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	21							Mostly fines, low plasticity; 10-15% gravel, fine to coarse grained, subangular to subrounded; 5-10% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 1 inch; very stiff; moist; brown; occasional weak reaction to HCl; (CL); [Colluvium]
	22							
	23	S - 7	12/14/17	1.5	1.5			
	24							
9820.8	25							
	26							S-8: Sandy Lean Clay with Gravel Mostly fines, low plasticity; 20-35% sand, fine to coarse grained, subangular to subrounded; 15-30% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1 inch; very stiff; moist; brown; chlorite deposits throughout; (SC); [Colluvium]
	27					Unable to perform Shelby tube because of gravel/cobbles.		
	28	S - 8	20/17/15	1.5	1.5			
	29							
9815.8	30							
	31							CA-9: Gravelly Lean Clay Mostly fines, low to medium plasticity; 30-40% gravel, fine to coarse grained, subangular to subrounded; 5-10% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 1.25 inches; stiff to very stiff; moist; brown; PP = 1.75 tsf; (CL); [Colluvium]
	32	CA - 9	9/13	1.0	1.3	Unable to perform Shelby tube because of gravel/cobbles.		
9813.1	33					Bottom of boring at 33.0 feet.		End of boring log at 33.00 ft
	34							
	35							
	36							
	37							
	38							
	39							
	40							

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well at the dam toe.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995632, Long: -108.110160 deg
Ground El: 9865.7 ft Total Depth: 73.5 ft
Groundwater El: 9860.9 ft On Date: 09-17-2018

Start Date: 09-17-2018 End Date: 09-20-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 5" ID, 5-3/8" OD Symmetrix Drive Casing Advancer

Borehole ID: **B-105A(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	1					Sample Bu-1 collected from 0 to 12.0 feet. Continuous slow, smooth drilling. Dust from large boulder while drilling, approximately 1 foot in diameter per driller.		Bu-1, S-2, S-3, S-4,: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 20-35% gravel, fine to coarse grained, subangular to subrounded; 15-30% fines, medium plasticity; maximum particle size = 1.5 inches; medium dense; moist; brown; occasional strong reaction with HCl; (SC); [Colluvium]
	2							
	3	S - 2	11/8/7	1.5	0.5			
	4							
	5	S - 3	3/2/4	1.5	1.5	Groundwater encountered at 4.8 feet during drilling on 9/17/2018.		*4.5 to 6.0 ft: 15-25% gravel; 25-40% fines; loose; wet;*
	6							
	7	S - 4	50 for 5 inches	0.9	0.7			*7.0 to 7.9 ft: 20-35% fines; very dense; moist to wet;*
	8					Dust from large boulder while drilling, approximately 1 foot in diameter per driller.		
9856.7	9					Groundwater encountered at 9.2 feet on 9/18/2018 a.m. after drilling to 27.0 feet the previous day.		S-5, S-6: Sandy Lean Clay with Gravel Mostly fines, low plasticity; 15-30% sand, fine to coarse grained, subangular to subrounded; 15-25% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 0.75 inches; very soft to soft; moist to wet; brown; (CL); [Colluvium]
	10							
	11	S - 5	2/3/3	1.5	1.8			
	12							
	13	S - 6	2/3/2	1.5	1.3			
	14					Sampling interval changed to about every 5 feet.		
9850.4	15							S-7: Lean Clay with Sand Mostly fines, low to medium plasticity; 5-15% sand, fine to medium grained, subangular to subrounded; less than 10% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.25 inches; soft; moist to wet; brown; (CL); [Colluvium]
	16							
	17							
	18	S - 7	4/4/6	1.5	1.7			
9846.7	19					Driller said material changed to clay with more gravel or a stiffer clay.		S-8: Lean Clay Mostly fines, low to medium plasticity; less than 10% sand, fine to medium grained, subangular to subrounded; medium
	20							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well at the dam toe.

RJH CONSULTANTS, INC

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995632, Long: -108.110160 deg
Ground El: 9865.7 ft Total Depth: 73.5 ft
Groundwater El: 9860.9 ft On Date: 09-17-2018

Start Date: 09-17-2018 End Date: 09-20-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 5" ID, 5-3/8" OD Symmetrix Drive Casing Advancer

Borehole ID: **B-105A(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	21							stiff; moist to wet; brown; (CL); [Colluvium]
	22							
	23	S - 8	8/13/15	1.5	1.8			
	24							
9840.4	25					Very slow drilling and increased basalt gravel in cuttings from 25.5 to 34.0 feet.		S-9: Sandy Lean Clay with Gravel Mostly fines, low plasticity; 15-30% sand, fine to coarse grained, subangular to subrounded; 15-25% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.25 inches; medium stiff to stiff; moist; (CL); [Colluvium]
	26							
	27	S - 9	12/15/42	1.5	1.5			
	28							
	29							
9835.4	30							S-10: Sandy Lean Clay Mostly fines, low plasticity; 15-30% sand, fine to coarse grained, subangular to subrounded; 5-15% gravel, fine grained, subangular to subrounded; maximum particle size = 0.75 inches; medium stiff to stiff; moist; brown; (CL); [Colluvium]
	31							
	32	S - 10	50 for 4 inches	0.3	0.4			
	33							
	34							
9831.0	35							S-11, CA-12: Sandy Lean Clay with Gravel Mostly fines, low plasticity; 15-30% sand, fine to coarse grained, subangular to subrounded; 15-25% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.5 inches; medium stiff; moist; brown; (CL); [Colluvium]
	36							
	37							
	38	S - 11	25/34/31	1.5	1.8			
	39							
	40							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well at the dam toe.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995632, Long: -108.110160 deg
Ground El: 9865.7 ft Total Depth: 73.5 ft
Groundwater El: 9860.9 ft On Date: 09-17-2018

Start Date: 09-17-2018 End Date: 09-20-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Borehole ID: **B-105A(P)**
Sheet 3 of 4

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 5" ID, 5-3/8" OD Symmetrix Drive Casing Advancer

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	41							
	42	CA - 12	50/50 for 3 inches	0.8	0.8			*42.0 to 42.8 ft: PP = 2.75 tsf;*
	43							
	44							
9820.8	45							CA-13: Gravelly Lean Clay with Sand Mostly fines, low to medium plasticity; 20-35% gravel, fine to coarse grained, subangular to rounded; 15-25% sand, fine to coarse grained, subangular to subrounded; maximum particle size = 2 inches; medium stiff to stiff; moist; brown; (CL); [Colluvium]
	46							
	47	CA - 13	28/50 for 2 inches	0.7	0.7	CA-13 disturbed, sampler bouncing on cobble/boulder.		
	48							
	49					Driller said encountered pea gravel at 49.0 feet.		
9815.8	50							S-14, S-15: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 25-40% fines, low to medium plasticity; 20-35% gravel, fine to coarse grained, subangular to rounded; maximum particle size = 1.5 inches; very dense; moist to wet; brown; (SC); [Colluvium]
	51							
	52	S - 14	10/50 for 5 inches	0.9	0.4	Pressurized water coming out between casing and boring annulus at surface on 9/18/2018 after drilling to 52.0 feet. Resolved and continued drilling approximately 1 hour later.		
	53							
	54							
	55							
	56							
	57							*57.0 to 58.5 ft: 20-35% fines; 15-25% gravel, subangular to subrounded;*
	58	S - 15	31/27/39	1.5	0.5			
	59							
	60							

Continued on next sheet

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well at the dam toe.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995632, Long: -108.110160 deg
Ground El: 9865.7 ft Total Depth: 73.5 ft
Groundwater El: 9860.9 ft On Date: 09-17-2018

Start Date: 09-17-2018 End Date: 09-20-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 5" ID, 5-3/8" OD Symmetrix Drive Casing Advancer

Borehole ID: **B-105A(P)**

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
9805.4	61							CA-16: Sandy Lean Clay with Gravel Mostly fines, low plasticity; 15-30% sand, fine to coarse grained, subangular to subrounded; 15-25% gravel, fine to coarse grained, subangular to subrounded; maximum particle size = 1.25 inches; medium stiff; moist to wet; brown; (CL); [Colluvium]
	62					CA-16 sample rings collected from 62.5 to 63.5 feet.		*62.0 to 63.5 ft: PP = 4.5 tsf;*
	63	CA - 16	20/13/19	1.5	1.5			
	64							
	65							
9800.4	66							S-17, S-18: Clayey Sand with Gravel Mostly sand, fine to coarse grained, subangular to subrounded; 20-35% gravel, fine to coarse grained, subangular to subrounded; 15-25% fines, low to medium plasticity; maximum particle size = 1.5 inches; very dense; moist to wet; brown; (SC); [Colluvium]
	67	S - 17	50 for 2 inches	0.2	0.2			
	68							
	69							
	70							
	71							
	72							*72.0 to 73.5 ft: 25-35% fines;*
	73	S - 18	12/22/33	1.5	1.5			
9792.2	74					Bottom of boring at 73.5 feet.		End of boring log at 73.50 ft
	75							
	76							
	77							
	78							
	79							
	80							

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well at the dam toe.

RJH CONSULTANTS, INC.

LOG OF SOIL BORING

Project name: Hogchute Dam Safety Evaluation
Project No: 18115
Boring Location: Lat: 38.995681, Long: -108.110127 deg
Ground El: 9866.9 ft Total Depth: 12.6 ft
Groundwater El: 9858.7 ft On Date: 09-19-2018

Start Date: 09-19-2018 End Date: 09-20-2018
Driller: HRL Compliance - Jose Logged By: JNH
Bedrock Depth: Not encountered Checked By: ERS

Drilling Rig: CME 55LC Track Mounted Rig
Equipment: 5" ID, 5-3/8" OD Symmetrix Drive Casing Advancer

Borehole ID: **B-105B(P)**
Sheet 1 of 1

Elevation	Depth (ft)	Type - No	Blows per 6 inch	Penetration (ft)	Recovery (ft)	Remarks	Graphic Lithology	Description and Classification of Materials
	0					Continuous slow, smooth drilling from 0 to 12.6 feet.		0.0 to 12.6 ft: No Sampling. Refer to B-105A(P) for lithology.; [Colluvium]
	1							
	2							
	3							
	4							
	5							
	6							
	7							
	8					Groundwater encountered at 8.2 feet during drilling on 9/19/2018.		
	9							
	10							
	11							
	12							
9854.3	13					Bottom of boring at 12.6 feet.		End of boring log at 12.60 ft
	14							
	15							
	16							
	17							
	18							
	19							
	20							

Notes Contacts are approximate and lithology between recovered samples is interpreted. Material descriptions are based on recovered samples, cuttings, and surface observations. Density descriptions are based on blow counts. Large particles may have influenced blow counts and sample recovery. Boring was completed as a monitoring well at the dam toe.

RJH CONSULTANTS, INC.

Appendix C

Sample Photographs

B-101(P)

Photograph 1: B-101(P); S-2 from 5.0 to 6.5 feet. Clayey Gravel with Sand [Fill].

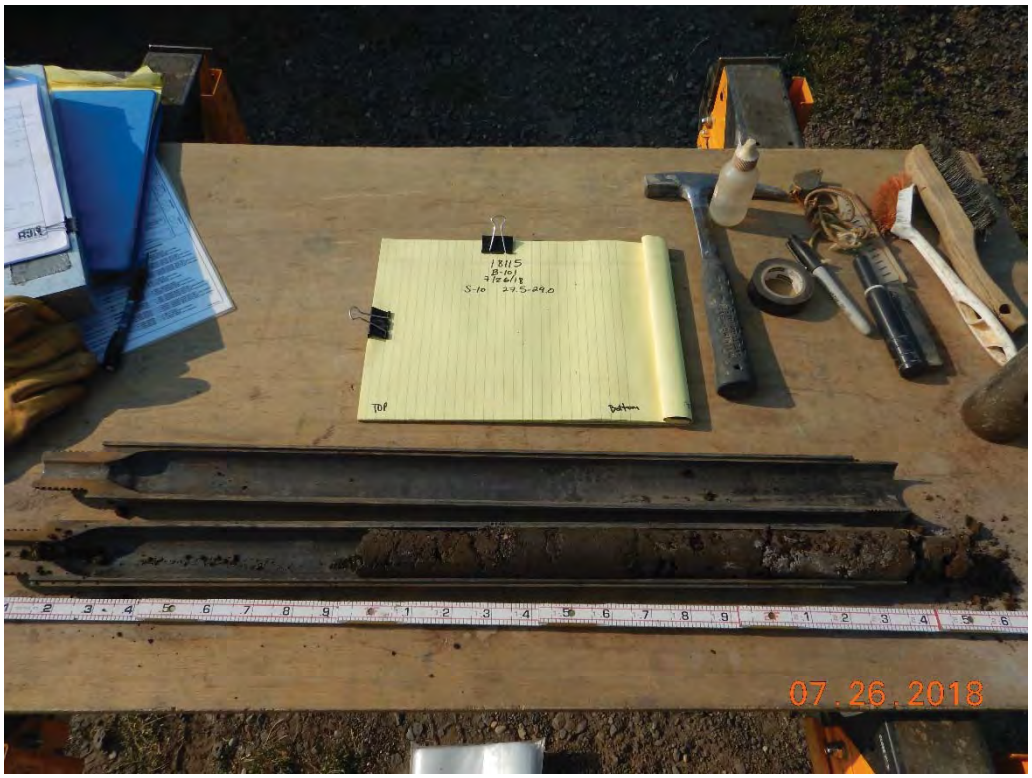
Photograph 2: B-101(P); S-3 from 10.0 to 11.5 feet. Clayey Sand with Gravel [Fill].

Photograph 3: B-101(P); S-3 from 12.5 to 14.0 feet. Clayey Sand with Gravel [Fill].

Photograph 4: B-101(P); U-5 from 15.0 to 15.6 feet. Disturbed. Clayey Sand with Gravel [Fill].

Photograph 5: B-101(P); S-7 from 17.5 to 19.0 feet. Clayey Sand with Gravel [Fill].

Photograph 6: B-101(P); S-9 from 25.0 to 26.5 feet. Clayey Sand with Gravel [Fill].



Photograph 7: B-101(P); S-10 from 27.5 to 29.0 feet. Clayey Sand with Gravel [Fill].

Photograph 8: B-101(P); S-12 from 35.0 to 36.5 feet. Clayey Sand with Gravel [Fill].

Photograph 9: B-101(P); S-14 from 40.0 to 41.5 feet. Clayey Sand with Gravel [Fill].

Photograph 10: B-101(P); S-18 from 45.0 to 46.5 feet. Clayey Sand with Gravel [Fill].

Photograph 11: B-101(P); S-19 from 50.0 to 51.5 feet. Clayey Sand with Gravel [Fill]. Sandy Lean Clay with Gravel [Alluvium/Colluvium].

Photograph 12: B-101(P); S-22 from 60.0 to 61.5 feet. Sandy Lean Clay with Gravel and Poorly Graded Sand with Clay and Gravel [Alluvium/Colluvium].

Photograph 13: B-101(P); S-24 from 70.0 to 71.5 feet. Lean Clay [Alluvium/Colluvium].

Photograph 14: B-101(P); S-25 from 73.0 to 74.5 feet. Sandy Lean Clay [Alluvium/Colluvium].

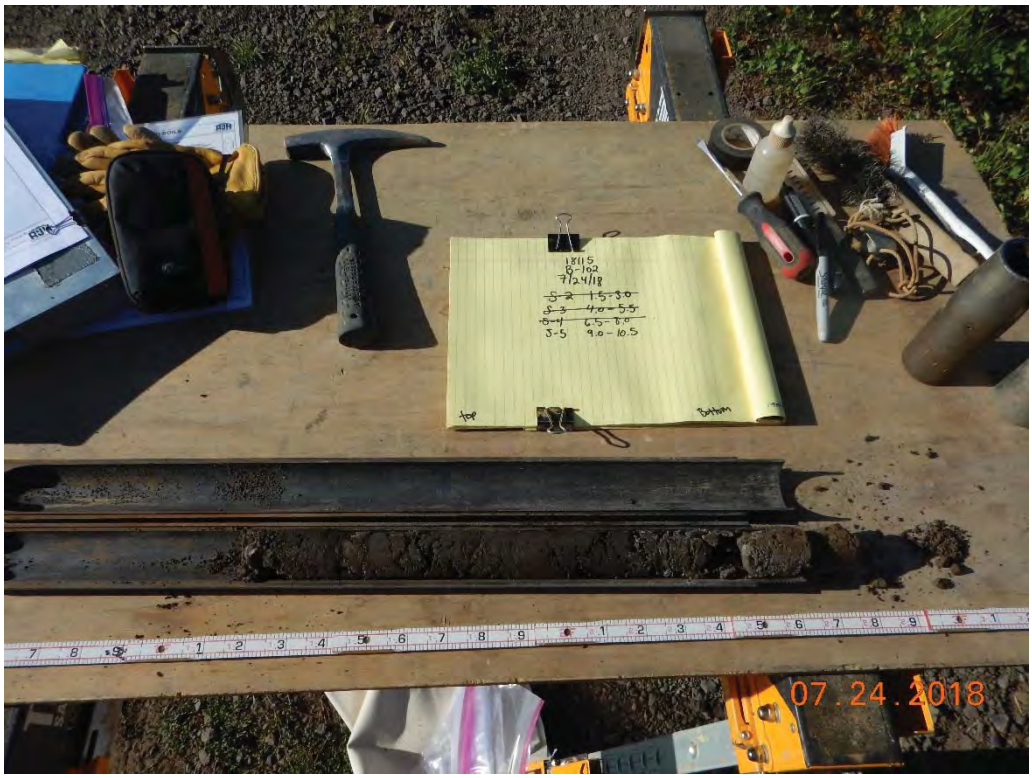
Photograph 15: B-101(P); S-26 from 76.0 to 77.5 feet. Sandy Lean Clay [Alluvium/Colluvium].

B-102A(P)

Photograph 16: B-102A(P); S-2 from 1.5 to 3.0 feet. Clayey Gravel with Sand [Fill].

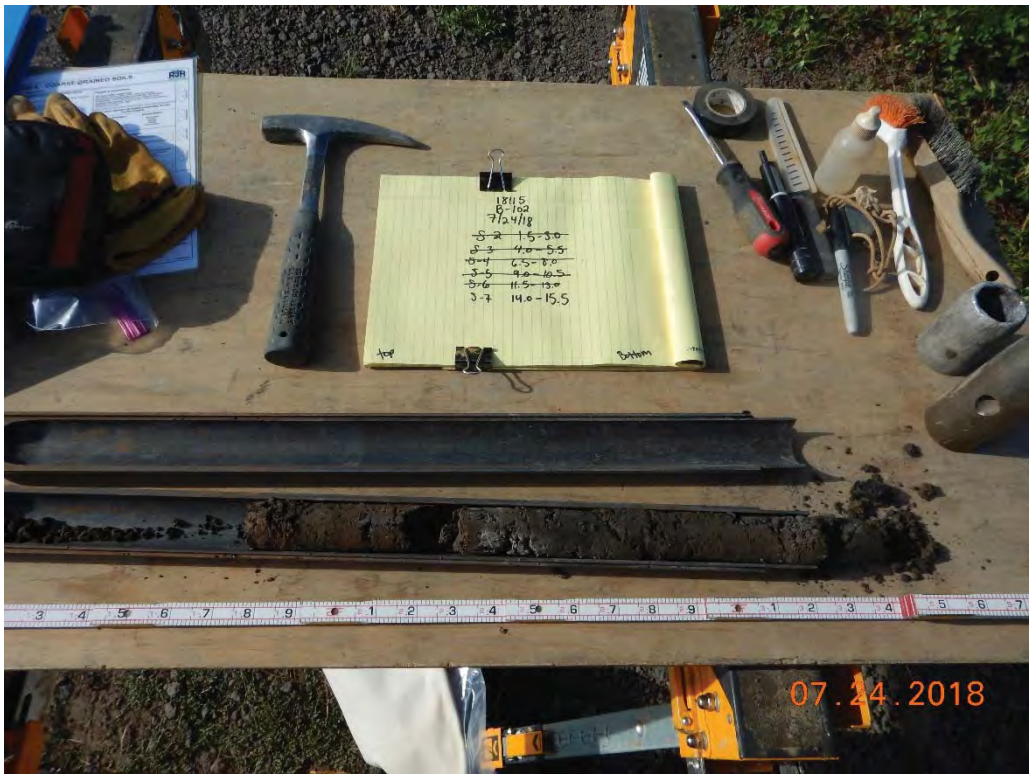
Photograph 17: B-102A(P); S-3 from 4.0 to 5.5 feet. Clayey Gravel with Sand [Fill].

Photograph 18: B-102A(P); S-4 from 6.5 to 8.0 feet. Clayey Gravel with Sand [Fill].



Photograph 19: B-102A(P); S-5 from 9.0 to 10.5 feet. Clayey Gravel with Sand [Fill].

Photograph 20: B-102A(P); S-6 from 11.5 to 13.0 feet. Clayey Gravel with Sand [Fill].



Photograph 21: B-102A(P); S-7 from 14.0 to 15.5 feet. Clayey Gravel with Sand [Fill].

Photograph 22: B-102A(P); S-8 from 16.5 to 18.0 feet. Clayey Gravel with Sand [Fill].

Photograph 23: B-102A(P); S-11 from 21.5 to 23.0 feet. Clayey Gravel with Sand [Fill].

Photograph 24: B-102A(P); S-12 from 24.0 to 24.8 feet. Clayey Gravel with Sand [Fill].

Photograph 25: B-102A(P); S-13 from 26.5 to 28.0 feet. Clayey Sand with Gravel [Fill].

Photograph 26: B-102A(P); S-15 from 31.5 to 33.0 feet. Clayey Gravel with Sand [Fill].

Photograph 27: B-102A(P); S-16 from 34.0 to 35.5 feet. Clayey Gravel with Sand [Fill].

Photograph 28: B-102A(P); S-17 from 36.5 to 38.0 feet. Clayey Gravel with Sand [Fill].

Photograph 29: B-102A(P); S-18 from 39.0 to 39.8 feet. Poorly Graded Gravel with Clay and Sand [Fill].

Photograph 30: B-102A(P); S-19 from 41.5 to 43.0 feet. Clayey Sand with Gravel [Fill].

Photograph 31: B-102A(P); S-21 from 46.5 to 46.8 feet. Poorly Graded Gravel with Clay and Sand [Fill].

B-103(P)

Photograph 32: B-103(P); S-2 from 1.0 to 2.5 feet. Clayey Sand with Gravel [Fill].

Photograph 33: B-103(P); S-3 from 3.5 to 4.6 feet. Clayey Gravel with Sand [Fill].

Photograph 34: B-103(P); S-4 from 5.0 to 6.5 feet. Clayey Sand with Gravel [Fill].

Photograph 35: B-103(P); S-5 from 7.5 to 9.0 feet. Clayey Sand with Gravel [Fill].

Photograph 36: B-103(P); S-6 from 10.0 to 11.5 feet. Clayey Sand with Gravel [Fill].

Photograph 37: B-103(P); S-7 from 12.5 to 14.0 feet. Clayey Sand with Gravel [Fill].

Photograph 38: B-103(P); S-10 from 21.0 to 22.3 feet. Clayey Sand with Gravel [Fill].

Photograph 39: B-103(P); S-12 from 22.5 to 24.0 feet. Clayey Sand with Gravel [Fill]. Gravelly Lean Clay with Sand [Alluvium/Colluvium].

Photograph 40: B-103(P); S-13 from 25.0 to 26.5 feet. Gravelly Lean Clay with Sand [Alluvium/Colluvium].

B-104(P)

Photograph 41: B-104(P); S-1 from 4.0 to 5.5 feet. Basalt [Alluvium/Colluvium].

Photograph 42: B-104(P); S-2 from 4.0 to 5.5 feet. Sandy Lean Clay with Gravel [Alluvium/Colluvium].

Photograph 43: B-104(P); S-4 from 13.0 to 13.3 feet. Gravelly Lean Clay with Sand [Alluvium/Colluvium].

Photograph 44: B-104(P); S-5 from 13.4 to 14.9 feet. Sandy Lean Clay with Gravel [Alluvium/Colluvium].

Photograph 45: B-104(P); S-6 from 17.0 to 17.3 feet. Gravelly Lean Clay with Sand [Alluvium/Colluvium].

Photograph 46: B-104(P); S-7 from 22.0 to 23.5 feet. Lean Clay with Gravel [Alluvium/Colluvium].

Photograph 47: B-104(P); S-8 from 27.0 to 28.5 feet. Sandy Silt with Gravel [Alluvium/Colluvium].

B-105A(P)

Photograph 48: B-105A(P); S-2 from 2.0 to 3.5 feet. Clayey Sand with Gravel [Alluvium/Colluvium].

Photograph 49: B-105A(P); S-3 from 4.5 to 6.0 feet. Clayey Sand with Gravel [Alluvium/Colluvium].

Photograph 50: B-105A(P); S-4 from 7.0 to 7.9 feet. Clayey Sand with Gravel [Alluvium/Colluvium].

Photograph 51: B-105A(P); S-5 from 10.0 to 11.5 feet. Sandy Lean Clay with Gravel [Alluvium/Colluvium].

Photograph 52: B-105A(P); S-6 from 12.0 to 13.5 feet. Sandy Lean Clay with Gravel [Alluvium/Colluvium].

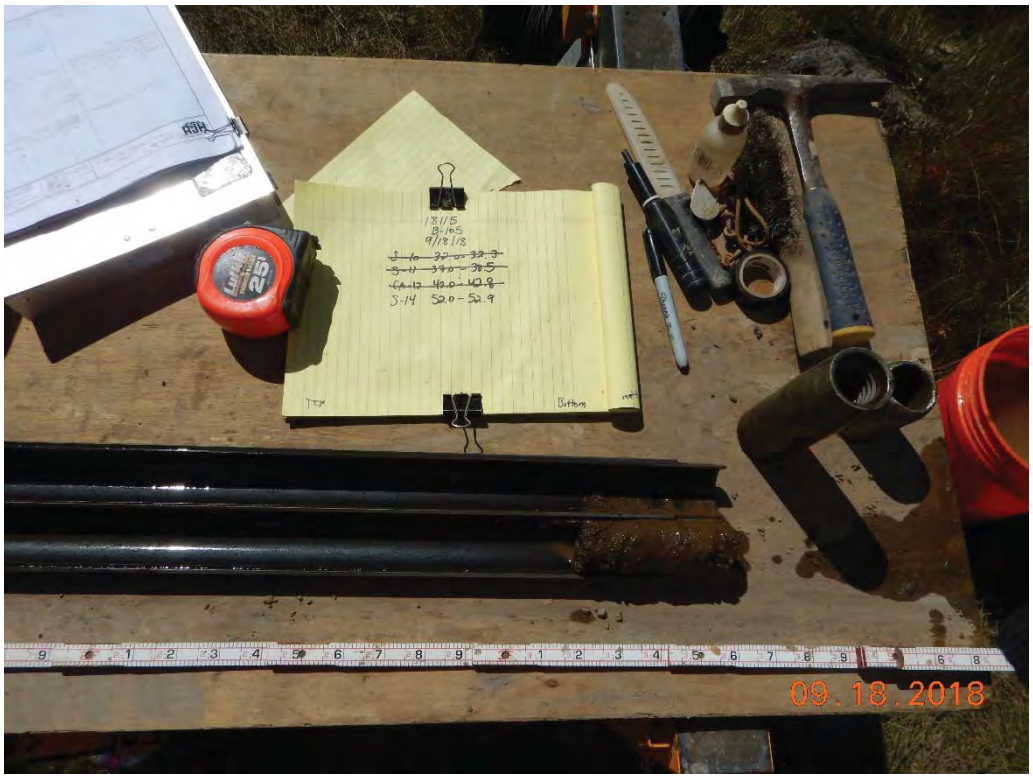
Photograph 53: B-105A(P); S-7 from 17.0 to 18.5 feet. Lean Clay with Sand [Alluvium/Colluvium].

Photograph 54: B-105A(P); S-8 from 22.0 to 23.5 feet. Lean Clay [Alluvium/Colluvium].

Photograph 55: B-105A(P); S-9 from 27.0 to 28.5 feet. Sandy Lean Clay with Gravel [Alluvium/Colluvium].

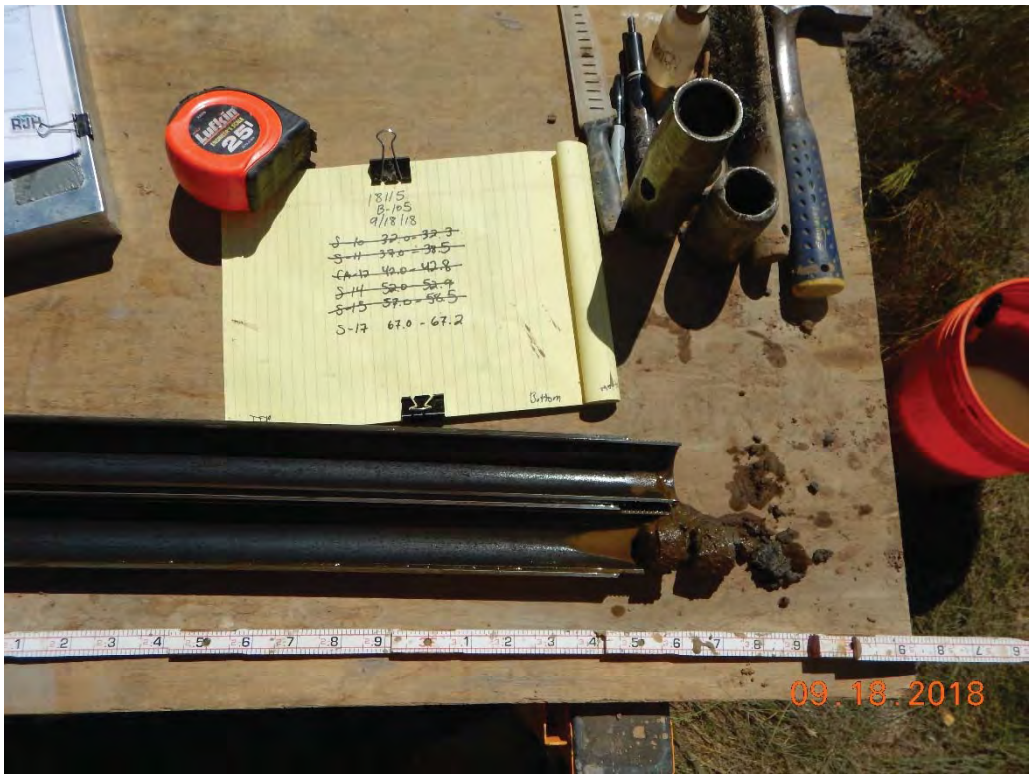
Photograph 56: B-105A(P); S-10 from 32.0 to 32.3 feet. Sandy Lean Clay [Alluvium/Colluvium].

Photograph 57: B-105A(P); S-11 from 37.0 to 38.5 feet. Sandy Lean Clay with Gravel [Alluvium/Colluvium].



Photograph 58: B-105A(P); S-14 from 52.0 to 52.9 feet. Clayey Sand with Gravel [Alluvium/Colluvium].

Photograph 59: B-105A(P); S-15 from 57.0 to 58.5 feet. Clayey Sand with Gravel [Alluvium/Colluvium].



Photograph 60: B-105A(P); S-17 from 67.0 to 67.2 feet. Clayey Sand with Gravel [Alluvium/Colluvium].

Photograph 61: B-105A(P); S-18 from 72.0 to 73.5 feet. Clayey Sand with Gravel [Alluvium/Colluvium].

APPENDIX D

DAILY SITE REPORTS

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 001-JNH**
Task 1002 Geotechnical Investigation **Date: Monday, July 23, 2018**

Prepared By: JNH
Weather: low 80's, mostly sunny, light-moderate breeze
Boring(s): B-103(P)

People on Site (arrival/departure time)

- RJH:
  - JNH (09:45/19:00)
  - GOJ (09:45/17:10)
- HRL:
  - Jose Suarez & Devin Lucero (10:00/18:55)
  - Mark Mumby (10:00/12:05)
- Colorado SEO: Jackie Blumberg (on-site upon RJH arrival/15:50)
- U.S. Forest Service: name unknown, 1 person (11:45)

Equipment on Site

Mobile
- RJH T3
- GJ Chevy Silverado
- HRL Chevy 2500HD
- HRL Dodge 3500
- SEO Ford F-150
- USFS full-size pick-up truck

At Site Parking Lot Overnight
- HRL semi-truck and trailer
- HRL support trailer & materials

At Drill Site Overnight
- CME 55LC track mounted drill rig

Material Used

- 2 buckets coated pellets (5 gallon)
- 2 bags of medium bentonite chips (50# bag)
- 15 bags of 10/20 sand (50# bag)
- 1 J plug
- 10 ft slotted 2" PVC (40) pipe
- (2) 10 ft riser 2" PVC (40) pipe
- 1 slip cap

Drilling Progress Summary

- HRL mobilized the drill rig and drilling support equipment to the Site.
- Boring B-103 was drilled via HSA to a depth of 30.0 feet. Drillers changed the drill bit to a conical drill bit at 3.5 feet to aid with progressing through gravel and cobbles in the embankment fill.
- A falling head permeability test was completed in the foundation material, below groundwater, at the bottom of the boring.
- The drillers began the monitoring well installation and completed the backfill materials to about 2.0 feet. Tomorrow, concrete will be used to complete the backfill material and a surface mount casing placed in concrete.
- The drillers positioned the drill rig over the boring so as to cover the partially completed monitoring well. A J plug is set in the PVC to prevent debris from entering the monitoring well.

Seepage Observations

- Seepage near outlet works at toe appeared to have little to no flow prior to and post drilling activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to and post drilling activities.

Plan for Next Work Day

- JNH and drillers to meet at Site at 07:00.
- Mark to deliver concrete and packer. Complete monitoring well installation at B-103.
- Begin the next boring, B-102, and drill via HSA until advanced 2-3 feet into bedrock. Once in bedrock, switch to HQ wireline coring.
- If time and site conditions allow, install two nested piezometers at B-102; one into bedrock (denoted as B-102A(P)) and one into the embankment fill (denoted as B-102B(P)).

Site Coordination Activities

- A USFS personnel (Cliff) opened the locked gate so the field crew could transport equipment onto the crest. The gate lock is “dummy locked” for overnight so the field crew can access equipment tomorrow. Either the USFS or the City of Grand Junction will provide the field crew a key to the gate; until this occurs, the field crew will continue to “dummy lock” the gate for overnight.
- Several hikers and fishermen walked past the drill rig on the dam crest during the day with no incidents. Cliff will contact Jon Hare (USFS) about placing signage to close the

dam crest to the public during drilling operations. GJ also left a phone message for Jon about closing the dam crest.

Photographs

Boring B-103:

Figure 1. Dam crest and B-103 prior to drilling activities, looking south.

Figure 2. B-103 equipment set-up, looking southeast.

Figure 3. B-103 at the end of day, looking southeast. Drill rig parked over monitoring well.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 002-JNH**
Task 1002 Geotechnical Investigation **Date: Tuesday, July 24, 2018**

Prepared By: JNH
Weather: upper 70's, mostly sunny, clouds and breeze increase throughout day
Boring(s): B-102A(P) & B-103(P)

People on Site (arrival/departure time)

- RJH: JNH (06:50/19:45)
- HRL:
  - Jose Suarez & Devin Lucero (07:00/19:40)
  - Mark Mumby (12:20/13:25)
- City of Grand Junction: Lee Cooper (09:50/10:20)
- Colorado SEO: Jason Ward (12:40/13:10)

Equipment on Site

Mobile
- RJH T3
- HRL Chevy 2500HD
- HRL Dodge 3500
- City of Grand Junction full-size pick-up truck
- SEO Chevy Trailblazer

At Site Parking Lot Overnight
- HRL semi-truck and trailer
- HRL support trailer & materials

At Drill Site Overnight
- CME 55LC track mounted drill rig

Material Used

B-102A(P)
- 2 bags of 10/20 sand (50# bag)
- 1 J plug
- 10 ft slotted 2" PVC (40) pipe
- (4) 10 ft riser 2" PVC (40) pipe
- 1 slip cap

B-103(P)
- 1 bag concrete
- (1) 9-inch flush mount casing

Drilling Progress Summary

- Boring B-102 was drilled via HSA until augers hit refusal at 48.0 feet in the embankment fill. At 46.5 feet, drilling progressed 1.5 feet in 45 to 60 minutes to reach auger refusal depth of 48.0 feet. Numerous gravel and cobbles were encountered throughout drilling and prevented further advancement of the boring.
- A falling head permeability test was completed in the embankment fill, below groundwater and prior to auger refusal.
- Monitoring well, denoted as B-102A(P), was installed at about 48 feet within the embankment fill.
- The drillers began the monitoring well installation at B-102A(P) and completed the backfill materials to about 44.0 feet.
- The drillers attached the auger casing rod to the augers in boring B-102A(P) to cover and protect the integrity of the monitoring well for overnight.
- The drillers completed monitoring well B-103(P) by backfilling the remaining 2.0 feet with concrete and installing the surface mount casing.
- JNH developed B-103(P) using the surge block and submersible pump.

Seepage Observations

- Seepage behind the outlet works at toe appeared to have little to no flow prior to, during, and post drilling activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. This amount of seepage was also observed yesterday with no changes throughout the day today.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to, during, and post drilling activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH and drillers to meet at Site at 07:30.
- Mark to deliver additional cement and sand.
- Complete the monitoring well installation at B-102A(P).
- Attempt to drill B-102B via HSA about 5-8 feet south of B-102A location.
    - If difficulties advancing boring, JNH will contact GJ. GJ may advise field crew to backfill the boring with cement bentonite grout and proceed to drilling the toe borings.
    - If drilling advancement successful, drill via HSA until advanced 2-3 feet into bedrock. Once in bedrock, switch to HQ wireline coring. If time and site conditions allow, install monitoring well within bedrock.

Site Coordination Activities

- Representatives from the City of Grand Junction and Colorado SEO were temporarily on-site to meet the field crew, observe and discuss drilling progress, and make other on-site observations.
- GJ provided a lock to secure the USFS gate for the remainder of the week. JNH will remove the lock upon completion of site activities.
- For public safety, JNH will place flags on the crest between the parking lot and the drilling equipment and project vehicles.

Non-RJH Activities

- About 8-10 hikers and fishermen on-site throughout the day, but not near the crest or drilling equipment.

Photographs

Boring B-102A:

Figure 1. Dam crest and B-102A prior to drilling activities, looking south. Green "X" in photo indicates boring location.

Figure 2. B-102A equipment set-up, looking southeast.

Figure 3. B-102A at the end of day, looking southeast. Auger casing rod is attached to in-place augers.

Boring B-103:

Figure 1. Monitoring well at B-103 covered with bucket while awaiting concrete backfill and well completion. Looking north.

Figure 2. Completed monitoring well installation at B-103, looking north.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 003-JNH**
Task 1002 Geotechnical Investigation **Date: Wednesday, July 25, 2018**

Prepared By: JNH
Weather: mid 70's, partly sunny and moderate breeze
Boring(s): B-102A(P), B-102B, B-101

People on Site (arrival/departure time)

- RJH: JNH (07:20/18:30)
- HRL:
  - Jose Suarez & Devin Lucero (07:20/18:30)
  - Mark Mumby (08:50/11:55)
- City of Grand Junction: Slade & Jerry(?) (10:30/10:50)

Equipment on Site

Mobile
- RJH T3
- HRL Chevy 2500HD
- HRL Dodge 3500
- City of Grand Junction full-size pick-up truck

At Site Parking Lot Overnight
- HRL semi-truck and trailer
- HRL support trailer & materials

At Drill Site Overnight (at B-101)
- CME 55LC track mounted drill rig

Material Used

B-102A(P)
- 4 bags of 10/20 sand (50# bag)
- 4 buckets of coated bentonite pellets
- 5 bags of Portland cement (47# bag)
- ½ bag high yield bentonite powder (50# bag)

B-102B(P)
- 2 bags of Portland cement (47# bag)
- ¼ bag high yield bentonite powder (50# bag)

B-101
- None

Drilling Progress Summary

- The drillers continued to work on the B-102A(P) monitoring well installation. The sand pack was having difficulty settling on the bottom of the boring due to the presence of turbid water inside the boring. Mark was able to help the drillers troubleshoot the problem by pumping the fluid out of the boring and adding clean water to the boring; this allowed for the sand to settle and pack properly at the bottom of the boring.
- Cement bentonite grout was placed in B-102A(P) via tremie pipe and hose. The grout needs to set overnight and the monitoring well was covered with a bucket at the end of the day.
- Boring B-102B was attempted about 8 feet south of B-102A(P) and drilled via HSA until auger refusal at about 5 feet. It took 45 minutes to advance the boring 5 feet and continuous auger grinding and rig rocking were observed. Numerous gravel and cobbles were encountered during drilling and prevented further advancement of the boring.
- B-102B was backfilled with cement bentonite grout. The grout needs to set overnight and the boring was covered with a basalt boulder at the end of the day.
- Boring B-101 was drilled via HSA. Gravel and cobbles are present in the embankment fill; however, the augers are advancing at a rate of about 5 feet in 30 minutes, including taking samples. Today, attempts to sample with the California sampler and the Shelby tube have failed due to the presence of gravel and cobbles. The depth of the boring was at 25.0 feet at the end of the day.
- The drillers attached the auger casing rod to the augers in B-101 to cover and protect the integrity of the boring for overnight.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have little flow prior to, during, and post drilling activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. JNH approximated the seepage rate using a 5-gallon bucket at about 4.3 gpm. No changes in the seepage rate or clarity of water were observed prior to, during, and post drilling activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to, during, and post drilling activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH and drillers to meet at Site at 07:30.
- Mark to deliver additional cement.

- Complete the monitoring well installation at B-102A(P) by placing concrete and the surface mount casing.
- Complete the backfilling of B-102B and cover with cuttings.
- Continue to drill B-101 via HSA.
    - If drilling advancement successful, drill via HSA until advanced 2-3 feet into bedrock. Once in bedrock, switch to HQ wireline coring. If time and site conditions allow, install nested piezometers (one in bedrock and one in embankment).
    - If difficulties advancing boring, either install a monitoring well below the phreatic surface or, if groundwater not encountered prior to auger refusal, backfill boring with cement bentonite grout.

Site Coordination Activities

- Representatives from the City of Grand Junction were temporarily on-site to deliver materials to place flagging on the crest between the parking lot and the drilling equipment and project vehicles. JNH placed two orange cones and strung caution tape between the cones, across the crest. The representatives said to leave the material near the gate building on the crest at the end of drilling activities.

Non-RJH Activities

- About 20 hikers and fishermen on-site throughout the day. In the morning, a group of about 8 hikers walked around the drilling equipment along the crest but remained a safe distance from equipment.
- City of Grand Junction on-site with full-size pick-up truck and backhoe. Slade said the City is removing a beaver dam on the right side of the reservoir; a backhoe is being used to remove the debris. The beaver dam has diverted water flow into the reservoir, thus resulting in the spillway overtopping. Once the beaver dam is removed, the reservoir level should return to below or at the spillway crest.

Photographs

Borings B-102A and B-102B:

Figure 1. B-102A at start of day, looking southeast.

Figure 2. B-102A equipment set-up during grouting, looking southeast.

Figure 3. B-102B prior to drilling activity, boring area circled in red and indicated with green "X." Augers in foreground are in B-102A during coated pellet curing time. Looking south.

Figure 4. B-102A and B-102B at the end of day, looking north. B-102A is in background covered with a white bucket, circled in red. B-102B is in foreground covered with a basalt boulder, indicated by red arrow.

Boring B-101:

Figure 5. B-101 prior to drilling activities, indicated by green "X." Looking south.

Figure 6. B-101 equipment set-up during drilling, looking south.

Figure 7. B-101 at the end of day, looking south.

Other Photos:

Figure 8. Cones and caution tape across crest at north end (nearest parking lot), looking south.

Figure 9. Crest at the end of day, looking south. Cones and caution tape across crest are indicated with red arrows.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 004-JNH**
Task 1002 Geotechnical Investigation **Date: Thursday, July 26, 2018**

Prepared By: JNH
Weather: mid 70's, mostly sunny with moderate breeze in afternoon
Boring(s): B-102A(P), B-102B, B-101

People on Site (arrival/departure time)

- RJH:
  - JNH (07:30/18:55)
  - GOJ (11:20/12:05)
- HRL:
  - Jose Suarez & Devin Lucero (07:30/18:55)
  - Chris (07:30/08:00)

Equipment on Site

Mobile
- RJH T3
- GJ Chevy Silverado
- HRL Chevy 2500HD
- HRL Dodge 3500

At Site Parking Lot Overnight
- HRL semi-truck and trailer
- HRL support trailer & materials

At Drill Site Overnight (at B-101)
- CME 55LC track mounted drill rig

Material Used

B-102A(P)
- 2 bags of concrete (50# bag)
- 9-inch surface mount casing

B-102B(P)
- None

B-101
- None

Drilling Progress Summary

- Chris (HRL) dropped off additional bags of Portland cement and buckets of coated pellets.
- The drillers completed the monitoring well installation at B-102A(P) by placing concrete and installing the surface mount casing.
- JNH developed B-102A(P) using the surge block and submersible pump. A rising head permeability test was also completed.
- The backfill of B-102B was completed by covering the cement bentonite grout with 0.3 feet of cuttings to create a level surface with the crest.
- Drilling B-101 was continued via HSA. Prior to start of drilling for the day, groundwater was measured in the boring to about 24.2 feet below ground surface (bgs). The cuttings appeared wet to about 34 feet bgs; however, the samples within the interval from 25 to 31 feet were moist. Deeper than 34 feet the cuttings and samples were moist.
- The contact between the embankment and natural ground occurred at about 50.6 feet bgs. The contact appeared as a distinct change in color and increasing clay content (from a generally clayey sand with gravel in the embankment to a sandy lean clay with gravel in the foundation).
- Groundwater was encountered at about 55.3 feet bgs.
- Today, attempts to sample with the California sampler were successful but attempts with the Shelby tube have failed due to the presence of gravel and cobbles.
- Numerous cobbles were encountered throughout drilling resulting in slow boring advancement. At the beginning of the day, the augers were advancing at a rate of about 5 feet in 25 to 30 minutes. Near the middle to end of the day, the augers were advancing about 5 feet in 45 minutes with intermittent auger grinding.
- The depth of the boring was at 75.0 feet at the end of the day and bedrock was not encountered.
- The drillers attached the auger casing rod to the augers in B-101 to cover and protect the integrity of the boring for overnight.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have little flow prior to, during, and post drilling activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. JNH approximated the seepage rate using a 5-gallon bucket at about 4.3 gpm. No changes in the seepage rate or clarity of water were observed prior to, during, and post drilling activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to, during, and post drilling activities.

- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH and drillers to meet at Site at 07:30.
- Mark to deliver additional 5-10 feet of augers and core boxes.
- Continue to drill B-101 via HSA until auger refusal, socketed into bedrock, or if augers reach maximum depth as identified by HRL.
    - If bedrock is encountered, drill via HSA until augers advanced 2-3 feet into bedrock only if the end auger depth is less than the maximum identified by HRL. Once in bedrock, switch to HQ wireline coring. If time and site conditions allow, install piezometer into bedrock only, since groundwater was encountered in the embankment foundation.
    - If difficulties advancing boring, install a monitoring well within the foundation material.

Site Coordination Activities

- None

Non-RJH Activities

- About 25-30 hikers and fishermen on-site throughout the day. A group of 3 hikers walked around the drilling equipment along the crest but remained a safe distance from equipment.

Photographs

Borings B-102A and B-102B:

Figure 1. B-102A and B-102B at start of day, looking south. B-102A in the foreground is covered by a white bucket, circled in red. B-102B in the background is covered by a basalt boulder, indicated by red arrow.

Figure 2. B-102A and B-102B at completion, looking south. B-102A in the foreground is circled in red. B-102B in the background is indicated by red arrow.

Figure 3. B-102B at completion, circled in red. Looking south.

Boring B-101:

Figure 4. B-101 at start of day, looking south.

Figure 5. B-101 equipment set-up during drilling, looking southeast.

Figure 6. B-101 at the end of day, looking southeast.

Other Photos:

Figure 7. Crest at the end of day, looking south. Cones and caution tape across crest are indicated with red arrows.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 005-JNH**
Task 1002 Geotechnical Investigation **Date: Friday, July 27, 2018**

Prepared By: JNH
Weather: high mid 70's, mostly sunny with increasing clouds and thunderstorm in the evening. Drilling was stopped and Site evening clean-up was occurring when first signs of thunder began.
Boring(s): B-101

People on Site (arrival/departure time)

- RJH: JNH (07:20/19:20)
- HRL:
  - Jose Suarez & Devin Lucero (07:20/19:20)
  - Mark Mumby (08:30/19:15)

Equipment on Site

Mobile
- RJH T3
- HRL Chevy 2500HD
- HRL Dodge 3500

At Site Parking Lot Overnight
- HRL semi-truck and trailer
- HRL support trailer & materials

At Drill Site Overnight (at B-101)
- CME 55LC track mounted drill rig

Material Used

B-101
- 10 bags of 10/20 sand (50# bag)
- 4 buckets of coated bentonite pellets
- 3 bags of Portland cement (92.6# bag) (so about 6 bags of 47# cement)
- 1 bag high yield bentonite powder (50# bag)
- (1) 5 ft slotted 1.5" PVC (40) pipe
- (7) 10 ft riser 1.5" PVC (40) pipe
- (16) 5 ft tremie 1" PVC (40) pipe
- 1 PVC elbow
- 1 end cap

Drilling Progress Summary

- Drilling B-101 was continued via HSA. Prior to start of drilling for the day, groundwater was measured in the boring to about 38.3 feet below ground surface (bgs). The augers hit refusal at 76.0 feet. At 75.0 feet, drilling progressed 1.0 foot in 30 to 40 minutes with continuous auger grinding and rig rocking to reach auger refusal depth of 76.0 feet. A split spoon sample was taken from 76.0 to 77.5 feet and recovered a sandy lean clay with basalt gravel. Bedrock was not encountered in B-101.
- The drillers installed monitoring well B-101(P) at about 73.0 feet within the foundation. Backfill materials were completed to about 12.0 feet.
- During monitoring well installation, a nested piezometer was considered at B-101, with one piezometer within the foundation and one within the embankment; however, due to well installation challenges, time constraints, driller's Department of Transportation (DOT) hour restrictions, and other non-project constraints, only one piezometer was installed within the foundation.
- During the initial phase of the monitoring well installation, the sand pack was having difficulty settling on the bottom of the boring, similar to conditions encountered during the installation of B-102A(P). Mark was on-site to help the drillers troubleshoot the well installation. After discussions with RJH management, additional water was poured down the hole to counteract an upward gradient from a confined groundwater source. The additional water head allowed for the sand to settle and pack properly at the bottom of the boring.
- Cement bentonite grout was placed in B-101(P) via tremie pipe to about 12.0 feet. The grout needs to set overnight and the monitoring well was covered for overnight.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have little flow prior to, during, and post drilling activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. JNH approximated the seepage rate using a 5-gallon bucket at about 4.3 gpm. No changes in the seepage rate or clarity of water were observed prior to, during, and post drilling activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to, during, and post drilling activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH and drillers to meet at Site at 07:30.
- Complete the monitoring well installation at B-101(P) by placing bentonite chips, concrete, and the surface mount casing. Place cuttings around monitoring well to cover concrete.
- Demobilize equipment from site. Preferably, this will occur prior to persons from the Grand Mesa Ultra running race being on-site. However, if runners are present, equipment travel will attempt to be coordinated so as to move equipment while runners are not on the crest.
- JNH to store City of Grand Junction cones and caution tape at the gate structure building and to relock the gate with the USFS lock.

Site Coordination Activities

- None

Non-RJH Activities

- About 15-20 hikers and fishermen on-site throughout the day. Two hikers walked around the drilling equipment along the crest but remained a safe distance from equipment.
- A person from the Grand Mesa Ultra running race was on-site placing flags along the crest. The man said that there is a trail running race taking place tomorrow, Saturday 7/28/18, and the route travels across the crest of Hogchute Dam. The man indicated that the runners will be at the crest starting around 14:00.

Photographs

Boring B-101:

Figure 1. B-101 at start of day, looking southeast.

Figure 2. B-101 equipment set-up during grouting, looking southeast.

Figure 3. B-101 at end of day, looking southeast.

Other Photos:

Figure 4. Cones and caution tape across crest at north end (nearest parking lot) at end of day, looking south.

Figure 5. Flag placed on crest for trail running race that occurs Saturday 7/28/18, looking north. Flag circled in red. Monitoring well in photo is B-102A(P).

Figure 6. Flag placed on roadway between parking lot and crest for trail running race.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 006-JNH**
Task 1002 Geotechnical Investigation **Date: Saturday, July 28, 2018**

Prepared By: JNH
Weather: high low-mid 60's, clouds increasing to overcast with intermittent rain and few lightning and thunder. Drilling equipment was demobilized off the crest prior to lightning.
Boring(s): B-101

People on Site (arrival/departure time)

- RJH: JNH (07:20/10:30)
- HRL:
  - Jose Suarez & Devin Lucero (07:30/10:05)
  - Mark Mumby (08:25/10:05)

Equipment on Site

Mobile
- RJH T3
- HRL Chevy 2500HD
- HRL Dodge 3500

Demobilized
- HRL semi-truck and trailer
- HRL support trailer & materials
- CME 55LC track mounted drill rig

Material Used

B-101
- 2 buckets of coated bentonite pellets
- 5 bags of medium bentonite chips (50# bag)
- 2 bags of concrete (50# bag)
- 9-inch surface mount casing

Drilling Progress Summary

- No drilling occurred today.
- Prior to continuing the monitoring well installation at B-101 for the day, groundwater was measured in the well to about 30.55 feet below ground surface (bgs).
- The drillers completed the monitoring well installation at B-101(P).
- The drillers demobilized the drill rig and equipment off of the crest. Mark was on-site to assist the drillers with demobilization. The drillers completed site clean-up while JNH

developed well B-101(P) with the 1-inch hand bailer and performed a rising head permeability test.

- No 1.5-inch slip cap or J-plug was on-site. The well at B-101(P) was covered with duct tape to protect the well from debris. Mark said that he would return to the site next week (of 7/30/18) and install a cap for the well; JNH requested that Mark install a J-plug. Mark to notify JNH when he returns to the site to cover the well at B-101(P).
- JNH placed the City of Grand Junction cones and tape surrounding B-101(P) in an attempt to prevent the trail runners or other public on the crest from stepping into the concrete that was placed today. Plenty of space remains for public to safely access the crest on either side of the monitoring well.
- JNH secured the USFS gate with the USFS lock and removed RJH's lock.
- The drillers and all equipment and material were demobilized from the Site prior to any trail runners being present.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have little flow prior to and post daily activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. JNH approximated the seepage rate using a 5-gallon bucket at about 4.3 gpm. No changes in the seepage rate or clarity of water were observed prior to and post daily activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to and post daily activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- Drilling for this phase of the geotechnical investigation is complete.
- Mark to notify JNH when he returns to the site next week to place the J-plug in B-101(P).

Site Coordination Activities

- None

Non-RJH Activities

- About 15-20 fishermen and trail running race spectators were on-site throughout the morning.

Photographs

Boring B-101:

Figure 1. B-101 at start of day, looking southeast.

Figure 2. Duct tape covering well opening at B-101.

Figure 3. B-101 at end of day, looking north. Cones and caution tape placed either side of monitoring well surface mount casing.

Figure 4. Crest and B-101 at end of day, looking south. B-101 location circled in red.

Other Photos:

Figure 5. Crest at the beginning of day, looking south.

Figure 6. Crest at the end of day, looking south.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 007-JNH**
Task 1002 Geotechnical Investigation **Date: Monday, September 17, 2018**

Prepared By: JNH
Weather: high low 70's, mostly sunny, light to moderate breeze
Boring(s): B-105

People on Site (arrival/departure time)

- RJH:
  - JNH (08:45/17:45)
  - GOJ (12:50/17:10)
- HRL:
  - Jose Suarez & Devin Lucero (09:15/17:35)
  - Mark Mumby (09:15/11:45)
  - Chris (09:15/09:35 and 16:25/16:35)
- SEO: Jackie Blumberg (12:15/16:30)
- Girardis Towing (09:15/09:25)

Equipment on Site

Mobile
- RJH T3
- GOJ Chevy Silverado
- HRL Chevy 2500HD
- HRL Dodge 3500
- HRL Dodge diesel pickup truck and trailer
- SEO Ford F-150
- Girardis semi-truck and trailer

At Site Parking Lot Overnight
- HRL support trailer & materials

At Drill Site Overnight (B-105)
- CME 55LC track mounted drill rig
- Sullair 375HH trailer mounted air compressor
- New Holland C185 track mounted skid steer

Material

At Site Parking Lot
- About 4 cubic yards (CY) of C-33 fine aggregate sand
- About 4 CY of Chat/minus ¼ inch gravel

Used (B-105)

- About 2 CY each of fine aggregate sand and Chat/minus ¼ inch gravel staged near B-105

Carson Lake Water Level

- About 10 feet below the spillway crest

Drilling Progress Summary

- JNH recorded groundwater levels in B-101(P), B-102A(P), and B-103(P) prior to the start of drilling activities. B-102A(P) was pumped dry with the submersible pump and a rising head permeability test was performed.
- HRL mobilized the drill rig and drilling support equipment to the Site. HRL stated that Whitewater delivered the C-33 fine aggregate and Chat/minus ¼ inch gravel to the Site parking lot Friday 9/14/18.
- The keyed lock to the USFS gate was cut to allow equipment access. GOJ to coordinate lock replacement with the City and USFS.
- HRL mobilized about 2 CY each of the fine aggregate sand and ¼ inch gravel emergency supplies to near B-105.
- Boring B-105 was drilled via drive casing advancer methods, specifically Symmetrix, to a depth of 27.0 feet. A representative from the SEO was onsite during drilling. Smooth drilling advanced through clay, gravel, cobbles, and a few boulders. The air compressor was kept at the “low pressure” setting throughout drilling.
- Groundwater was encountered at about 4.8 feet and the depth of water in the boring varied throughout drilling. The groundwater was under positive pressure and as a result, a rising head permeability test was completed at 21.0 to 22.0 feet in lean clay material.
- A split spoon sample was taken from about 27.0 to 28.5 at the end of the day and bedrock was not encountered.
- The drillers attached the drive casing head in B-105 to cover and protect the integrity of the boring for overnight.
- RJH placed a keyed lock to secure the USFS gate for overnight.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have typical low flow prior to and post daily activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. JNH approximated the seepage rate using a 5-gallon bucket at about 5 gpm. No changes in the seepage rate or clarity of water were observed prior to and post daily activities.

- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to and post daily activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH to use the 1.5-inch submersible pump on B-101(P) and perform a rising head permeability test.
- JNH to meet drillers at Site at 07:30.
- Continue to drill B-105 via drive casing advancer until bedrock is encountered or a depth of about 75 feet is reached; HRL identified about 75 feet as the maximum allowable depth.
    - If bedrock is encountered, drill via drive casing advancer until advanced 2-3 feet into bedrock, only if the end casing depth is less than the maximum identified by HRL. Once in bedrock, switch to HQ wireline coring and perform packer tests.
- If time and site conditions allow, install piezometer in colluvium/alluvium.

Site Coordination Activities

- None

Non-RJH Activities

- About 15-25 fishermen and campers were on-site throughout the day; no public were near equipment at dam toe.

Photographs

Boring B-105:

Figure 1. B-105 (circled in red) prior to drilling, looking southeast.

Figure 2. B-105 equipment set-up prior to drilling, taken from crest looking west.

Figure 3. B-105 equipment set-up during drilling, looking south.

Figure 4. B-105 at the end of day, looking southeast.

Other Photos:

Figure 5. Carson lake water level prior to drilling activities, looking southwest. Water level is about 10 feet below normal maximum. Normal maximum marked with red arrow.

Figure 6. Emergency stockpiles of sand and gravel stationed at the Site parking lot, looking east.

Figure 7. Symmetrix drive casing advancer bit. Camera case is 6-inches long for scale.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 008-JNH**
Task 1002 Geotechnical Investigation **Date: Tuesday, September 18, 2018**

Prepared By: JNH
Weather: high low-mid 70's, mostly sunny, light to moderate breeze
Boring(s): B-105A

People on Site (arrival/departure time)

- RJH: JNH (06:40/17:40)
- HRL: Jose Suarez & Devin Lucero (07:25/17:25)
- SEO: Jason Ward (09:05/15:00)

Equipment on Site

Mobile
- RJH T3
- HRL Chevy 2500HD
- SEO – vehicle unknown

At Site Parking Lot Overnight
- HRL support trailer & materials

At Drill Site Overnight (B-105A)
- CME 55LC track mounted drill rig
- Sullair 375HH trailer mounted air compressor
- New Holland C185 track mounted skid steer

Material

At Site Parking Lot
- About 4 cubic yards (CY) of C-33 fine aggregate sand
- About 4 CY of Chat/minus ¼ inch gravel

Used (B-105A)
- About 2 CY each of fine aggregate sand and Chat/minus ¼ inch gravel staged near B-105
- RJH provided:
  - End Cap
  - 5 ft prepacked well screen 2" PVC (40) pipe
- HRL provided:
  - (7) 10 ft riser 2" PVC (40) pipe
  - 3 buckets of bentonite coated pellets

Carson Lake Water Level

- About 10 feet below the spillway crest

Drilling Progress Summary

- JNH pumped B-101(P) with the 1.5-inch submersible pump and performed a rising head permeability test prior to the start of drilling activities.
- Drilling B-105A was continued via drive casing advancer methods. Prior to start of drilling for the day, groundwater was measured in the boring to about 9.2 feet below ground surface (bgs). A representative from the SEO was onsite during almost the entire drilling process.
- Slow and smooth drilling advanced through clay, gravel, cobbles, and boulders. The air compressor was kept at the "low pressure" setting throughout drilling.
- The depth of groundwater in the boring varied throughout drilling. The groundwater was under positive pressure and water releasing (i.e. bubbling) at the surface was observed when the boring bottom was at about 52.0 feet. The water was releasing between the casing and the boring wall. Drilling was stopped to observed if conditions changed or recovered. A rising head permeability test (K-2) was performed and data was recorded for 67 minutes. The water release at the surface stopped after about 30 minutes of no drilling activity. At about 65 minutes, the water level and water rebound rate was similar to the water depth and rate of water rebound observed in the rising head permeability test (K-1) performed yesterday, 9/17/18.
- After discussions among the field crew, it appeared likely that no damage was caused from today's drilling activities. RJH decided to continue drilling based on the following observations:
    - The water release stopped.
    - The ring bit provided about 3/8-inch total annulus around the casing and boring wall and is the source of space for air and water to travel to the surface.
    - The water rebound rate was observed to be approximately the same as observed during K-1 at a similar water depth.
- The depth of the boring was at a maximum allowable depth of 72.0 feet at the end of the day and bedrock was not encountered. No additional water releases at the surface were observed.
- A split spoon sample was taken from about 72.0 to 73.5 at the end of the day and bedrock was not encountered.
- The drillers began the monitoring well installation at B-105A(P) and completed the backfill materials to about 48.0 feet. Chat/minus ¼ inch gravel was used as the permeable backfill around the prepacked well screen and was placed downhole. Bentonite coated pellets were placed to 48.0 feet and allowed to set overnight.
- The drillers attached the drive casing head in B-105A(P) to cover and protect the integrity of the boring and monitoring well for overnight.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have typical low flow prior to, during, and post daily activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. No changes in the seepage rate or clarity of water were observed prior to, during, and post daily activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to, during, and post daily activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH to meet drillers at Site at 07:30.
- Continue to install monitoring well B-105A(P).
- Move drill rig about 10 feet in any feasible direction and begin drilling B-105B.
- Install monitoring well B-105B at about 10-12 feet to capture non-high-pressure phreatic surface. No sampling necessary at B-105B.
- If time and site conditions allow, begin to move equipment and material to B-104.

Site Coordination Activities

- JNH and Jason Ward discussed the SEO's verbal approval to continuing drilling the remaining monitoring wells along the dam toe. RJH to send summary email to Project partners and SEO regarding approval to continue drilling remaining borings; GOJ to perform this task after tomorrow's activities.

Non-RJH Activities

- About 15-25 fishermen, hunters, and campers were on-site throughout the day; no public were near equipment at dam toe.

City of Grand Junction (City)

- People on Site: 1 man
- Equipment on Site: City pickup truck
- One man from the City was observed onsite on the dam crest appearing to take monitoring well measurements of the crest wells. The water level he measured in B-101(P) will likely be still rebounding from RJH pumping that well and performing a rising head permeability test earlier today.

Photographs

Boring B-105A:

Figure 1. B-105A at start of day, looking west.

Figure 2. B-105A at start of day, looking south.

Figure 3. B-105A equipment set-up during drilling, looking south.

Figure 4. B-105A at the end of day, looking southeast.

Figure 5. B-105A at the end of day, looking west.

Other Photos:

Figure 6. Area where water release was occurring at the surface, circled in red. Drilling stopped.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 009-JNH**
Task 1002 Geotechnical Investigation **Date: Wednesday, September 19, 2018**

Prepared By: JNH
Weather: high mid 60's but mostly mid 50's, partly cloudy to overcast, occasional light rain, light to moderate breeze
Boring(s): B-105A, B-105B, B-104

People on Site (arrival/departure time)

- RJH:
  - JNH (06:55/17:30)
  - GOJ (13:15/14:40)
- HRL:
  - Jose Suarez & Devin Lucero (07:20/17:30)
  - Mark Mumby (10:50/11:15)

Equipment on Site

Mobile
- RJH T3
- GOJ Chevy Silverado
- HRL Chevy 2500HD
- HRL Dodge 3500

At Site Parking Lot Overnight
- HRL support trailer & materials

At Drill Site Overnight
- *At B-104*
  - CME 55LC track mounted drill rig

- *Right of Outlet Works*
  - Sullair 375HH trailer mounted air compressor
  - New Holland C185 track mounted skid steer

Material

At Site Parking Lot
- About 4 cubic yards (CY) of C-33 fine aggregate sand
- About 3 CY of Chat/minus ¼ inch gravel

Used
B-105A(P)
- 7 bags of Portland cement (47# bag)

- ½ bag high yield bentonite powder (50# bag)
- J plug

B-105B(P)

- RJH provided:
  - End cap
  - 5 ft prepacked well screen 2" PVC (40) pipe
- HRL provided:
  - (1) 10 ft riser 2" PVC (40) pipe
  - 1 bucket of bentonite coated pellets
  - 1 bag of Portland cement (47# bag)
  - ¼ bag high yield bentonite powder (50# bag)
  - J plug

B-104

- About 2 CY of fine aggregate sand and Chat/minus ¼ inch gravel moved from near B-105 to near B-104
- Additional about 1 CY Chat/minus ¼ inch gravel moved from Site parking lot to near B-104

Carson Lake Water Level

- About 10 feet below the spillway crest

Drilling Progress Summary

- JNH measured water levels in B-101(P), B-102A(P), and B-103(P) prior to the start of daily activities.
- Groundwater was measured in boring B-105A at about 9.1 feet bgs prior to daily activities.
- The drillers continued to work on the B-105A(P) monitoring well installation. Cement bentonite grout was placed via tremie pipe; the grout needs to set overnight.
- Boring B-105B was drilled about 9.5 feet north of B-105A via drive casing advancer methods to a total depth of about 12.6 feet. Smooth drilling advanced through clay, gravel, cobbles, and a few boulders at the surface. The air compressor was kept at the "low pressure" setting throughout drilling. No sampling was taken at B-105B due to the proximity to B-105A.
- No groundwater was observed in boring B-105B during drilling; however, water was heard entering the boring at a depth of about 12.6 feet and a rising head permeability test was performed. Groundwater raised to about 8.2 feet bgs.
- The drillers began the monitoring well installation at B-105B(P) and completed the backfill materials to about 2 feet. Chat/minus ¼ inch gravel was used as the

permeable backfill around the prepacked well screen and was placed downhole. Bentonite coated pellets were placed to 3 feet and allowed to set for about 1.5 hours. Cement bentonite grout was placed downhole; the grout needs to set overnight.

- The drillers mobilized equipment and material to B-104. Stockpiles of sand and gravel were moved from near B-105 to near B-104. Additional gravel was moved from the Site parking lot to near B-104. The remaining sand and gravel near B-105 was smoothed with rakes and shovels.
- Boring B-104 was drilled via drive casing advancer methods to a depth of about 12.0 feet. Slow and smooth drilling advanced through clay, gravel, cobbles, and boulders; numerous boulders were encountered from about 0 to 6 feet. The air compressor was kept at the "low pressure" setting throughout drilling.
- Groundwater was not encountered during drilling but samples were moist after about 7 feet.
- A Shelby tube was attempted at 12.0 feet but encountered a cobble at 13.0 feet which disturbed the sample. A split spoon sample was taken at about 13.0 feet but only advanced 0.3 feet due to a cobble. Bedrock was not encountered.
- The drillers attached the drive casing head in B-104 to cover and protect the integrity of the boring for overnight.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have typical low flow prior to and post daily activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. No changes in the seepage rate or clarity of water were observed prior to and post daily activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to and post daily activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH to meet drillers at Site at 07:30.
- Continue installation of monitoring wells B-105A(P) and B-105B(P).
- Continue to drill B-104 via drive casing advancer until bedrock is encountered or a depth of about 35 feet is reached.
    - If bedrock is encountered, drill via drive casing advancer until advanced 2-3 feet into bedrock. Once in bedrock, switch to HQ wireline coring, core about 10 feet into bedrock, and perform packer tests.
- Begin monitoring well B-104(P) installation. Install monitoring well in colluvium/alluvium.

Site Coordination Activities

- None

Non-RJH Activities

- About 10-15 fishermen, hunters, and campers were on-site throughout the day; no public were near equipment at dam toe.

Photographs

Boring B-105A:

Figure 1. B-105A at start of day, looking southeast.

Figure 2. B-105A during monitoring well installation, looking south.

Boring B-105B:

Figure 3. B-105B prior to drilling, looking south.

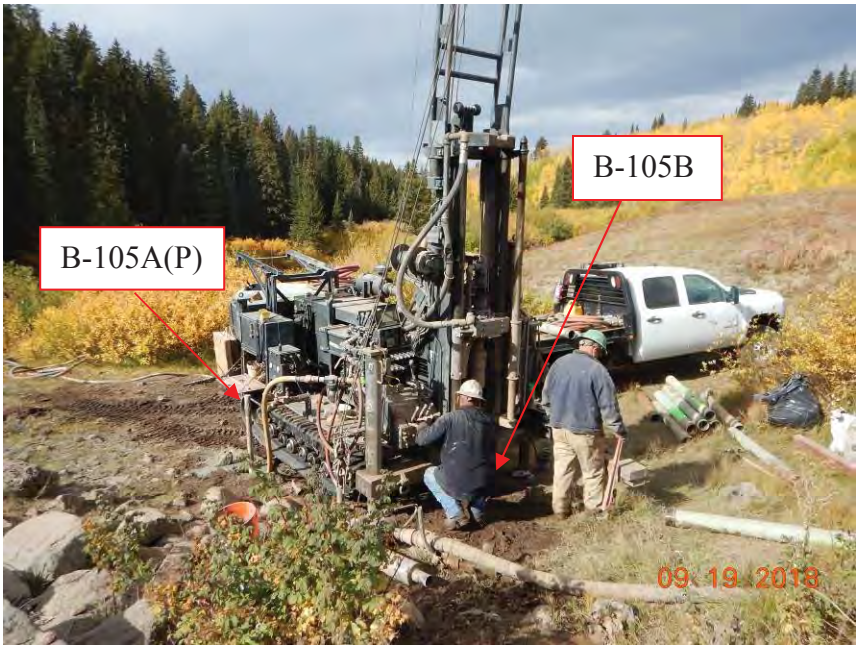


Figure 4. B-105B equipment set-up during drilling, looking west. B-105A(P) riser pipe on left in photo.

Figure 5. B-105B equipment set-up during drilling, looking south.

B-105A(P) and B-105B(P):

Figure 6. Riser pipes to B-105A(P) on right and B-105B(P) on left in photo at the end of day, looking southeast.

Boring B-104:

Figure 7. B-104 prior to drilling, looking south.

Figure 8. B-104 equipment set-up prior to drilling, looking southeast. Air compressor is right of outlet works and drill rig and skid steer are left of outlet works.

Figure 9. B-104 equipment set-up during drilling, looking east.

Figure 10. Sand and gravel stockpiles near B-104, looking west.

Figure 11. B-104 at the end of day, looking south.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 010-JNH**
Task 1002 Geotechnical Investigation **Date: Thursday, September 20, 2018**

Prepared By: JNH
Weather: high about 60°, sunny, light to moderate breeze
Boring(s): B-105A, B-105B, B-104

People on Site (arrival/departure time)

- RJH: JNH (07:00/16:50)
- HRL:
  - Jose Suarez & Devin Lucero (07:25/15:15)
  - Chris (Prior to RJH arrival on Site/07:10)
- SEO: Jackie Blumberg (08:25/13:20)

Equipment on Site

Mobile
- RJH T3
- HRL Chevy 2500HD
- HRL Toyota Tacoma
- SEO Ford F150

At Site Parking Lot Overnight
- HRL support trailer & materials
- Sullair 375HH trailer mounted air compressor

At Drill Site Overnight
- *At B-104*
  - CME 55LC track mounted drill rig

- *Right of Outlet Works*
  - New Holland C185 track mounted skid steer

Material

At Site Parking Lot
- About 4 cubic yards (CY) of C-33 fine aggregate sand
- About 3 CY of Chat/minus ¼ inch gravel

Used
B-105A(P)
- 1 bag medium bentonite chips (50# bag)
- ½ bag of concrete (50# bag)

- ½ bag 10/20 sand (50# bag)
- 5 ft steel riser casing, 4 in x 4 in square

B-105B(P)

- ½ bag of concrete (50# bag)
- ½ bag 10/20 sand (50# bag)
- 5 ft steel riser casing, 4 in x 4 in square

B-104

- About 2 CY of fine aggregate sand and about 3 CY Chat/minus ¼ inch gravel staged near B-104
- 2 bags of Portland cement (47# bag)
- ¼ bag high yield bentonite powder (50# bag)

Carson Lake Water Level

- About 10 feet below the spillway crest

Drilling Progress Summary

- JNH measured water levels in B-101(P), B-102A(P), and B-103(P) prior to the start of daily activities.
- The installation of monitoring wells B-105A(P) and B-105B(P) were completed by adding concrete and setting steel riser casings in each well. The steel riser casings are hinged and lockable; however, RJH did not place a lock on either casing.
- Groundwater was measured in boring B-104 at about 10.4 feet bgs prior to drilling activities.
- Drilling B-104 was continued via drive casing advancer methods to a total depth of about 32 feet. A representative from the SEO was onsite during the drilling process. Slow and smooth drilling advanced through clay, gravel, cobbles, and a few boulders. The air compressor was kept at the “low pressure” setting throughout drilling. Intervals with consistent clay materials took up to about 30 minutes to progress about 1.0 foot; however, typical progression rates were about 30 to 40 minutes to progress 5 feet.
- Two Shelby tubes were attempted but were unable to be pushed due to the presence of gravels and cobbles.
- The depth of groundwater in the boring varied throughout drilling. The groundwater was under positive pressure and water was heard entering the boring at a depth of about 26 to 27 feet and a rising head permeability test was performed.
- The drillers began to prepare for the monitoring well installation at B-104 and completed the backfill of cement bentonite grout to about 12 feet. The grout needs to set overnight and is anticipated to settle to about 15 to 16 feet.

- The drillers attached the drive casing head in B-104 to cover and protect the integrity of the boring for overnight.
- At the end of the day, JNH developed wells B-105A(P) and B-105B(P) with a surge block and submersible pump and performed a rising head permeability test on each well.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have typical low flow prior to, during, and post daily activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. No changes in the seepage rate or clarity of water were observed prior to, during, and post daily activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to, during, and post daily activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH to meet drillers at Site at 08:30.
- HRL to complete installation of monitoring well B-104(P).
- JNH to develop monitoring well B-104(P) with a surge block and submersible pump if conditions allow. JNH to also perform rising head permeability test if conditions allow.
- HRL to demobilize all equipment from Site in the afternoon.

Site Coordination Activities

- None

Non-RJH Activities

- About 10-20 fishermen, hunters, and campers were on-site throughout the day; no public were near equipment at dam toe.

City of Grand Junction (City)

- People on Site: Slade and Craig from about 09:30 to 09:50 and two men from about 11:45 to 11:55.
- Equipment on Site: City Dodge and Ford pickup trucks
- Slade and Craig were onsite measuring groundwater levels in the crest borings. JNH discussed the Project progress and pumping of B-101(P) on Tuesday 9/18/18 morning, prior to Craig's well measurement, which likely influenced Craig's measurement.

- Two other men from the City were onsite on the dam crest; RJH did not meet with these two City representatives.

- **Photographs**

Boring B-104:

Figure 1. B-104 prior to drilling, looking southwest.

Figure 2. B-104 equipment set-up prior to drilling, looking east.

Figure 3. B-104 equipment set-up during drilling, looking east.

Figure 4. B-104 at the end of day, looking east.

B-105A(P) and B-105B(P):

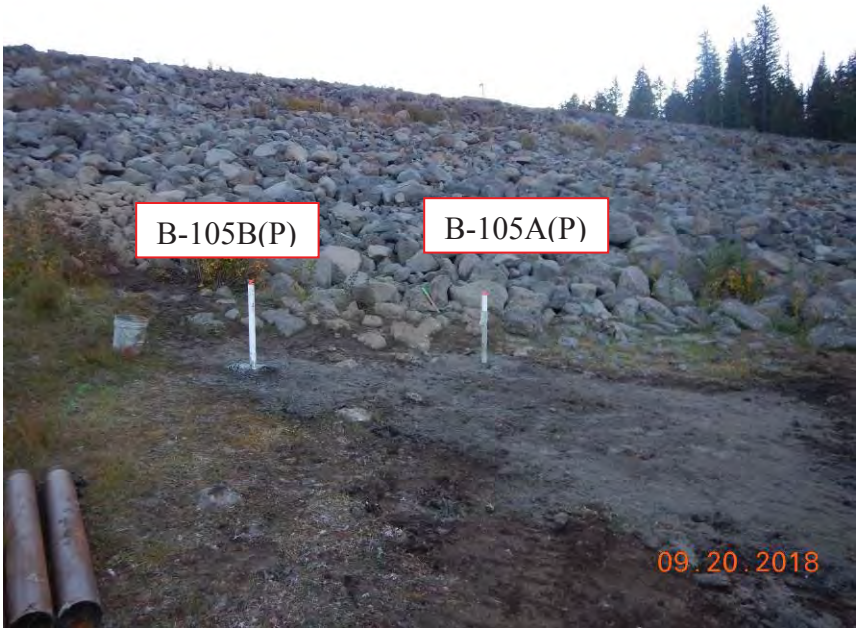


Figure 5. Riser pipes to B-105A(P) on right and B-105B(P) on left in photo at the start of day, looking southeast.

Figure 6. Riser casings to B-105A(P) on right and B-105B(P) on left in photo at the end of day, looking southeast.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 011-JNH**
Task 1002 Geotechnical Investigation **Date: Friday, September 21, 2018**

Prepared By: JNH
Weather: high about 60°, sunny
Boring(s): B-104

People on Site (arrival/departure time)

- RJH: JNH (08:45/14:10)
- HRL:
  - Jose Suarez & Devin Lucero (08:45/12:55)
  - Mark Mumby (11:15/12:55)
  - Chris (11:40/12:55)

Equipment on Site

Mobile
- RJH T3
- HRL Chevy 2500HD
- HRL Dodge diesel pickup truck and trailer
- HRL semi-truck and trailer

Demobilized
- HRL support trailer & materials
- Sullair 375HH trailer mounted air compressor
- CME 55LC track mounted drill rig
- New Holland C185 track mounted skid steer

Material

At Site Parking Lot
- About 4 cubic yards (CY) of C-33 fine aggregate sand
- About 3 CY of Chat/minus ¼ inch gravel

Near B-104
- About 2 CY of C-33 fine aggregate sand
- About 2 CY Chat/minus ¼ inch gravel

Used
B-104
- RJH provided:
  - End cap
  - 5 ft prepacked well screen 2” PVC (40) pipe

- HRL provided:
  - (2) 10 ft riser 2" PVC (40) pipe
  - 1 bucket of coated bentonite pellets
  - 1 bag of medium bentonite chips (50# bag)
  - J plug
  - 1 bag of concrete (50# bag)
  - 2 bags 10/20 sand (50# bag)
  - 5 ft steel riser casing, 4 in x 4 in square

Carson Lake Water Level

- About 10 feet below the spillway crest

Drilling Progress Summary

- JNH measured water levels in B-101(P), B-102A(P), B-103(P), B-105A(P), and B-105B(P) prior to the start of daily activities.
- The cement bentonite grout backfill in B-104 settled to about 17 feet bgs. Coated pellets were placed downhole and allowed to hydrate for 1 hour, bringing the bottom of B-104 up to about 14.5 feet bgs.
- The installation of monitoring well B-104(P) was completed by placing the monitoring well casing, adding backfill and concrete, and setting the steel riser casing. The steel riser casing is hinged and lockable; however, RJH did not place a lock on the casing.
- HRL demobilized all equipment from the Site.
- At the end of the day, JNH developed well B-104(P) with a surge block and submersible pump and performed a rising head permeability test.
- Drilling for this phase of the geotechnical investigation is complete.

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have typical low flow prior to, during, and post daily activities.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. No changes in the seepage rate or clarity of water were observed prior to, during, and post daily activities.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works prior to, during, and post daily activities.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- JNH to measure groundwater levels in monitoring wells B-101(P), B-102A(P), B-103(P), B-104(P), B-105A(P), and B-105B(P).

Site Coordination Activities

- None

Non-RJH Activities

- About 10-20 fishermen, hunters, and campers were on-site throughout the day; no public were near equipment at dam toe.

Photographs

Boring B-104:

Figure 1. B-104 prior to drilling, looking east.

Figure 2. B-104(P) completed well installation at the end of day, looking east.

B-105A(P) and B-105B(P):

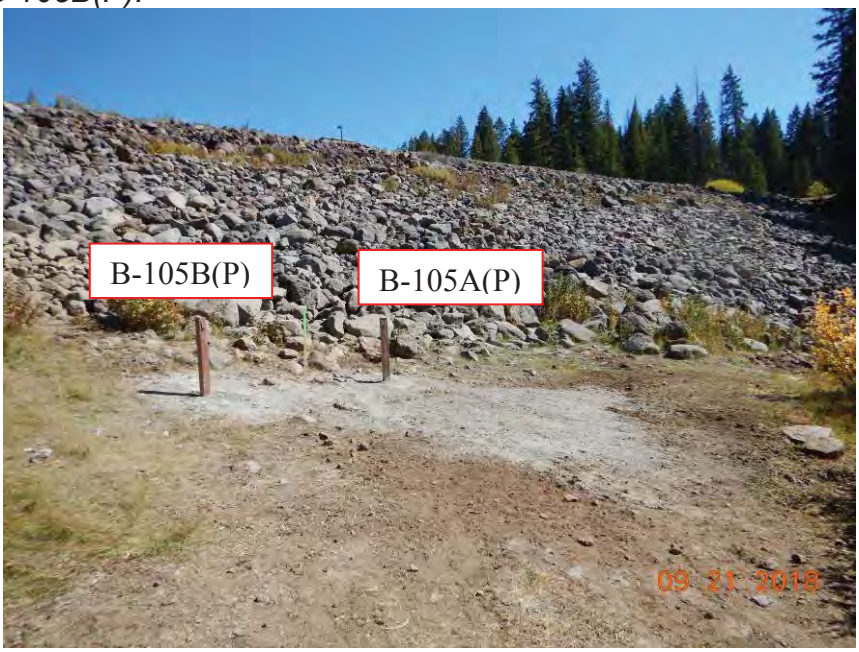


Figure 3. Riser casings to B-105A(P) on right and B-105B(P) on left in photo at the end of day, looking southeast.

Other Photos:

Figure 4. Demobilization of the drill rig and air compressor, looking west.

Figure 5. Site parking lot after equipment demobilization, looking east.

Figure 6. Sand and gravel stockpiles remain near B-104, looking west.

Figure 7. Sand and gravel stockpiles remain at Site parking lot, looking east. Camera case in photo is 6 inches long.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 012-JNH**
Task 1002 Geotechnical Investigation **Date: Saturday, September 22, 2018**

Prepared By: JNH
Weather: high about 45°, clear
Monitoring Wells: B-101, B-102A, B-103, B-104, B-105A, B-105B

People on Site (arrival/departure time)

- RJH: JNH (06:40/07:35)

Equipment on Site

Mobile
- RJH T3

Material

At Site Parking Lot
- About 4 cubic yards (CY) of C-33 fine aggregate sand
- About 3 CY of Chat/minus ¼ inch gravel

Near B-104
- About 2 CY of C-33 fine aggregate sand
- About 2 CY Chat/minus ¼ inch gravel

Used
- None

Carson Lake Water Level

- About 10 feet below the spillway crest

Site Progress Summary

- No drilling occurred today.
- JNH measured water levels in all six monitoring wells:
    - B-101(P)
    - B-102A(P)
    - B-103(P)
    - B-104(P)
    - B-105A(P)
    - B-105B(P)

Seepage Observations

- Seepage behind the outlet works right headwall at toe appeared to have typical low flow.
- Seepage from drain right of outlet works was flowing at a few gallons per minute with little to no turbidity. No changes in the seepage rate or clarity of water were observed.
- No water was observed in either of the two seepage channels in the willows right (north) of the outlet works.
- No other signs of seepage or abnormalities were observed.

Plan for Next Work Day

- No further field work is anticipated to be completed by RJH.

Site Coordination Activities

- RJH removed their temporary keyed lock and closed the gate with the chain link. The gate is not locked. The City is to coordinate with the USFS if needed to replace the lock and secure the access gate.

Non-RJH Activities

- About 5 fishermen and hunters were on-site in the morning; no public were near equipment at dam toe.

Photographs

Access Gate:

Figure 1. Access gate closed with chain link, but is not secured with lock. Looking east.

APPENDIX E

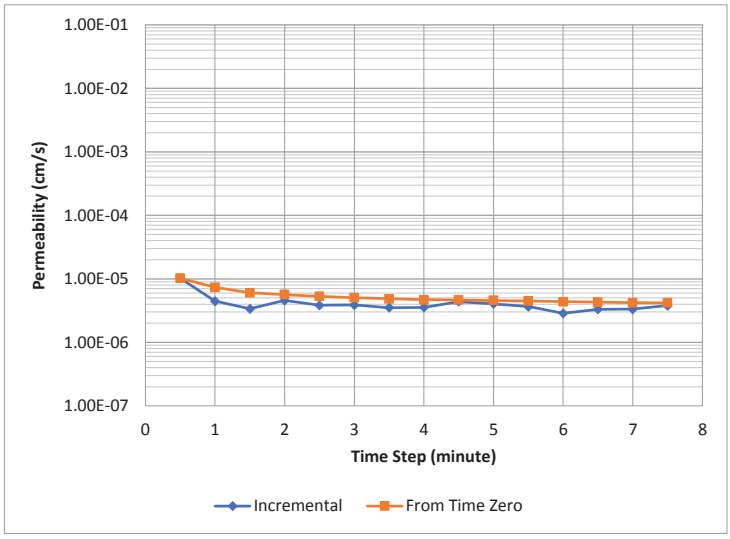
PERMEABILITY RESULTS

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	7/28/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-101(P)**
Test Number:	**K-1**

Depth to top of Ground Water	30.55	ft
Casing Stickup	0.0	ft
Top Depth of Test Interval	53.0	ft
Bottom Depth of Test Interval	74.0	ft
Inside Diameter Pipe	1.50	in
D = Diameter, intake, sample	7.75	in
L = Length, intake, sample	21.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
41.50	0.0	0	11.0	--	--	--	--
41.22	0.5	30	10.7	3.35E-07	1.02E-05	3.35E-07	1.02E-05
41.10	1.0	60	10.6	1.46E-07	4.46E-06	2.41E-07	7.34E-06
41.01	1.5	90	10.5	1.11E-07	3.38E-06	1.98E-07	6.02E-06
40.89	2.0	120	10.3	1.49E-07	4.55E-06	1.85E-07	5.65E-06
40.79	2.5	150	10.2	1.26E-07	3.83E-06	1.74E-07	5.29E-06
40.69	3.0	180	10.1	1.27E-07	3.87E-06	1.66E-07	5.05E-06
40.60	3.5	210	10.1	1.15E-07	3.52E-06	1.59E-07	4.83E-06
40.51	4.0	240	10.0	1.16E-07	3.55E-06	1.53E-07	4.67E-06
40.40	4.5	270	9.9	1.44E-07	4.38E-06	1.52E-07	4.64E-06
40.30	5.0	300	9.8	1.32E-07	4.03E-06	1.50E-07	4.58E-06
40.21	5.5	330	9.7	1.20E-07	3.66E-06	1.47E-07	4.50E-06
40.14	6.0	360	9.6	9.41E-08	2.87E-06	1.43E-07	4.36E-06
40.06	6.5	390	9.5	1.08E-07	3.30E-06	1.40E-07	4.28E-06
39.98	7.0	420	9.4	1.09E-07	3.33E-06	1.38E-07	4.21E-06
39.89	7.5	450	9.3	1.24E-07	3.78E-06	1.37E-07	4.18E-06
Estimated Permeability				**2.9E-06**	**to**	**1.0E-05**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**4.0.E-06**	**(cm/sec)**	

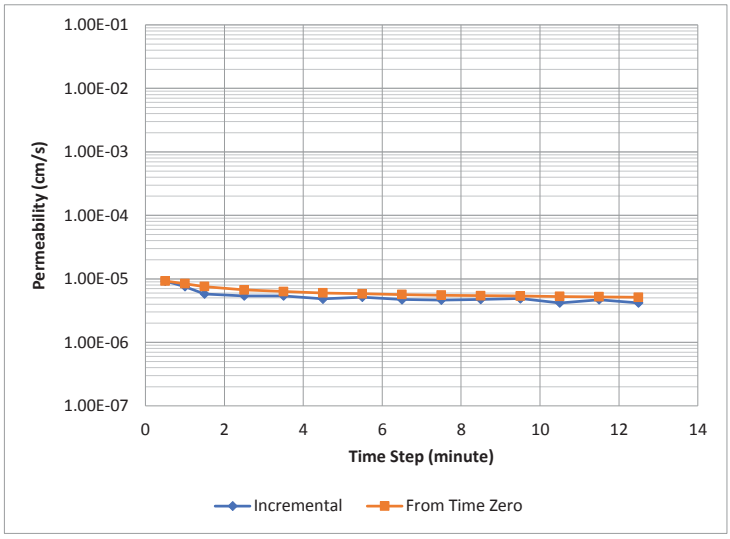
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/18/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-101(P)**
Test Number:	**K-2**

Depth to top of Ground Water	38.84	ft
Casing Stickup	0.0	ft
Top Depth of Test Interval	53.0	ft
Bottom Depth of Test Interval	74.0	ft
Inside Diameter Pipe	1.50	in
D = Diameter, intake, sample	7.75	in
L = Length, intake, sample	21.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
51.77	0.0	0	12.9	--	--	--	--
51.47	0.5	30	12.6	3.04E-07	9.26E-06	3.04E-07	9.26E-06
51.23	1.0	60	12.4	2.48E-07	7.57E-06	2.76E-07	8.42E-06
51.05	1.5	90	12.2	1.89E-07	5.77E-06	2.47E-07	7.53E-06
50.72	2.5	150	11.9	1.77E-07	5.40E-06	2.19E-07	6.68E-06
50.40	3.5	210	11.6	1.77E-07	5.39E-06	2.07E-07	6.31E-06
50.12	4.5	270	11.3	1.59E-07	4.84E-06	1.96E-07	5.98E-06
49.83	5.5	330	11.0	1.69E-07	5.14E-06	1.91E-07	5.83E-06
49.57	6.5	390	10.7	1.55E-07	4.72E-06	1.86E-07	5.66E-06
49.32	7.5	450	10.5	1.53E-07	4.65E-06	1.81E-07	5.53E-06
49.07	8.5	510	10.2	1.56E-07	4.76E-06	1.78E-07	5.44E-06
48.82	9.5	570	10.0	1.60E-07	4.88E-06	1.76E-07	5.38E-06
48.61	10.5	630	9.8	1.38E-07	4.20E-06	1.73E-07	5.26E-06
48.38	11.5	690	9.5	1.54E-07	4.70E-06	1.71E-07	5.22E-06
48.18	12.5	750	9.3	1.37E-07	4.18E-06	1.68E-07	5.13E-06
Estimated Permeability				**4.2E-06**	**to**	**9.3E-06**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**5.3.E-06**	**(cm/sec)**	

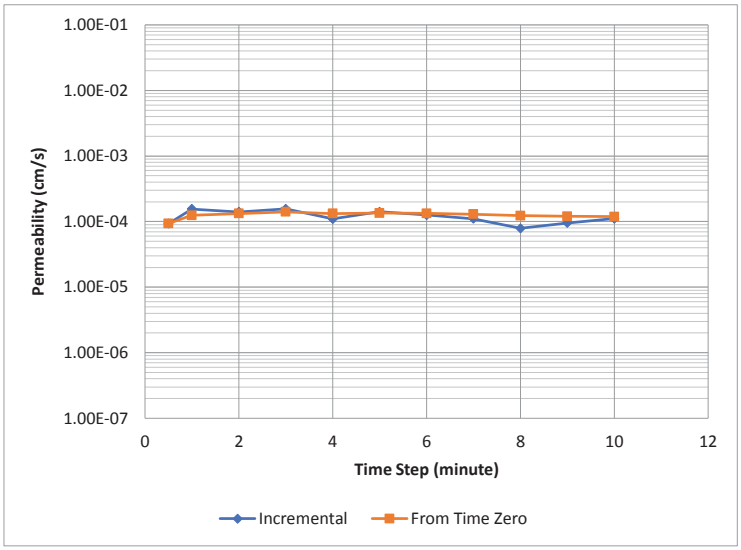
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Falling Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	7/24/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-102A(P)**
Test Number:	**K-1**

Depth to top of Ground Water	35.25	ft
Casing Stickup	3.9	ft
Top Depth of Test Interval	46.5	ft
Bottom Depth of Test Interval	46.5	ft
Inside Diameter Pipe	4.25	in
D = Diameter, intake, sample	4.25	in
L = Length, intake, sample	0.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
6.09	0.0	0	33.1	--	--	--	--
6.12	0.5	30	33.0	3.06E-06	9.33E-05	3.06E-06	9.33E-05
6.17	1.0	60	33.0	5.11E-06	1.56E-04	4.08E-06	1.24E-04
6.26	2.0	120	32.9	4.61E-06	1.40E-04	4.35E-06	1.32E-04
6.36	3.0	180	32.8	5.13E-06	1.56E-04	4.61E-06	1.40E-04
6.43	4.0	240	32.7	3.60E-06	1.10E-04	4.36E-06	1.33E-04
6.52	5.0	300	32.6	4.64E-06	1.42E-04	4.41E-06	1.35E-04
6.60	6.0	360	32.6	4.14E-06	1.26E-04	4.37E-06	1.33E-04
6.67	7.0	420	32.5	3.63E-06	1.11E-04	4.26E-06	1.30E-04
6.72	8.0	480	32.4	2.60E-06	7.92E-05	4.05E-06	1.24E-04
6.78	9.0	540	32.4	3.12E-06	9.52E-05	3.95E-06	1.20E-04
6.85	10.0	600	32.3	3.65E-06	1.11E-04	3.92E-06	1.20E-04
Estimated Permeability				**7.9E-05**	**to**	**1.6E-04**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**1.2.E-04**	**(cm/sec)**	

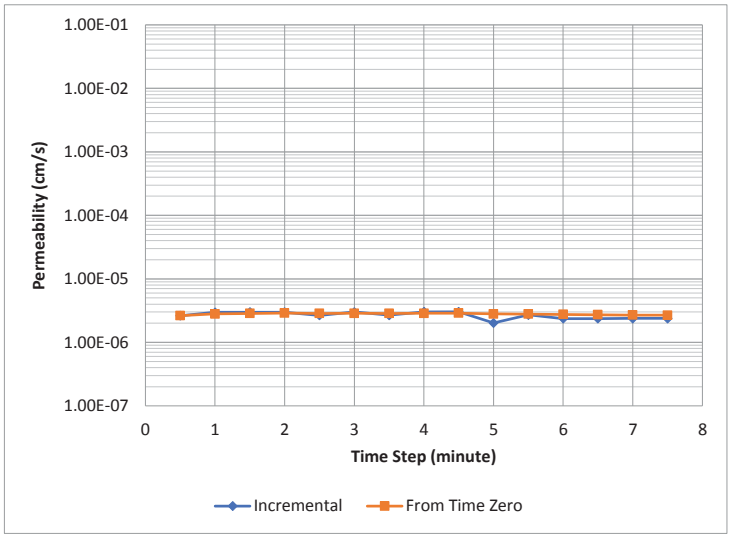
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case C.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	7/26/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-102A(P)**
Test Number:	**K-2**

Depth to top of Ground Water	10.36	ft
Casing Stickup	0.0	ft
Top Depth of Test Interval	36.5	ft
Bottom Depth of Test Interval	48.0	ft
Inside Diameter Pipe	2.00	in
D = Diameter, intake, sample	7.75	in
L = Length, intake, sample	11.5	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
43.70	0.0	0	33.3	--	--	--	--
43.62	0.5	30	33.3	8.64E-08	2.63E-06	8.64E-08	2.63E-06
43.53	1.0	60	33.2	9.75E-08	2.97E-06	9.19E-08	2.80E-06
43.44	1.5	90	33.1	9.77E-08	2.98E-06	9.39E-08	2.86E-06
43.35	2.0	120	33.0	9.80E-08	2.99E-06	9.49E-08	2.89E-06
43.27	2.5	150	32.9	8.73E-08	2.66E-06	9.34E-08	2.85E-06
43.18	3.0	180	32.8	9.85E-08	3.00E-06	9.42E-08	2.87E-06
43.10	3.5	210	32.7	8.78E-08	2.68E-06	9.33E-08	2.84E-06
43.01	4.0	240	32.7	9.90E-08	3.02E-06	9.40E-08	2.87E-06
42.92	4.5	270	32.6	9.93E-08	3.03E-06	9.46E-08	2.88E-06
42.86	5.0	300	32.5	6.63E-08	2.02E-06	9.18E-08	2.80E-06
42.78	5.5	330	32.4	8.86E-08	2.70E-06	9.15E-08	2.79E-06
42.71	6.0	360	32.4	7.77E-08	2.37E-06	9.03E-08	2.75E-06
42.64	6.5	390	32.3	7.79E-08	2.37E-06	8.94E-08	2.72E-06
42.57	7.0	420	32.2	7.81E-08	2.38E-06	8.86E-08	2.70E-06
42.50	7.5	450	32.1	7.82E-08	2.38E-06	8.79E-08	2.68E-06
Estimated Permeability				**2.0E-06**	**to**	**3.0E-06**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**2.7.E-06**	**(cm/sec)**	

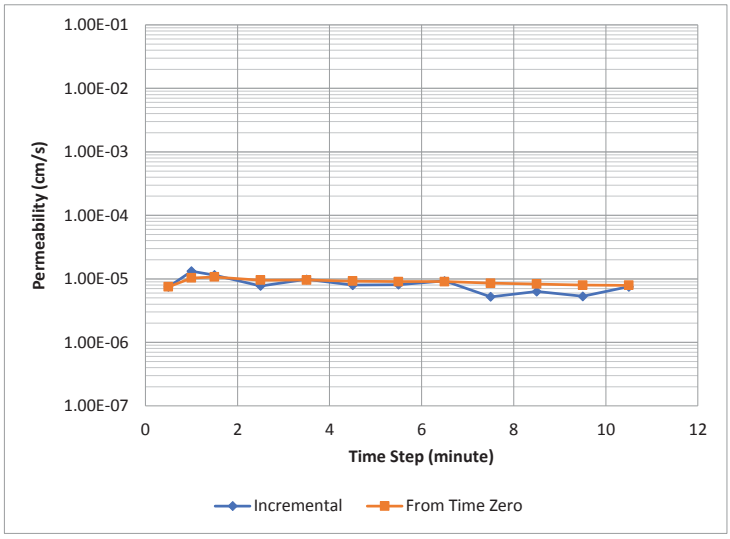
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/17/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-102A(P)**
Test Number:	**K-3**

Depth to top of Ground Water	37.48	ft
Casing Stickup	0.0	ft
Top Depth of Test Interval	36.5	ft
Bottom Depth of Test Interval	48.0	ft
Inside Diameter Pipe	2.00	in
D = Diameter, intake, sample	7.75	in
L = Length, intake, sample	11.5	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
43.37	0.0	0	5.9	--	--	--	--
43.33	0.5	30	5.9	2.45E-07	7.47E-06	2.45E-07	7.47E-06
43.26	1.0	60	5.8	4.33E-07	1.32E-05	3.39E-07	1.03E-05
43.20	1.5	90	5.7	3.75E-07	1.14E-05	3.51E-07	1.07E-05
43.12	2.5	150	5.6	2.53E-07	7.72E-06	3.12E-07	9.51E-06
43.02	3.5	210	5.5	3.22E-07	9.81E-06	3.15E-07	9.59E-06
42.94	4.5	270	5.5	2.62E-07	7.97E-06	3.03E-07	9.23E-06
42.86	5.5	330	5.4	2.65E-07	8.09E-06	2.96E-07	9.03E-06
42.77	6.5	390	5.3	3.03E-07	9.25E-06	2.97E-07	9.06E-06
42.72	7.5	450	5.2	1.71E-07	5.21E-06	2.80E-07	8.55E-06
42.66	8.5	510	5.2	2.07E-07	6.31E-06	2.72E-07	8.28E-06
42.61	9.5	570	5.1	1.74E-07	5.32E-06	2.62E-07	7.97E-06
42.54	10.5	630	5.1	2.47E-07	7.53E-06	2.60E-07	7.93E-06
Estimated Permeability				**5.2E-06**	**to**	**1.3E-05**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**8.0.E-06**	**(cm/sec)**	

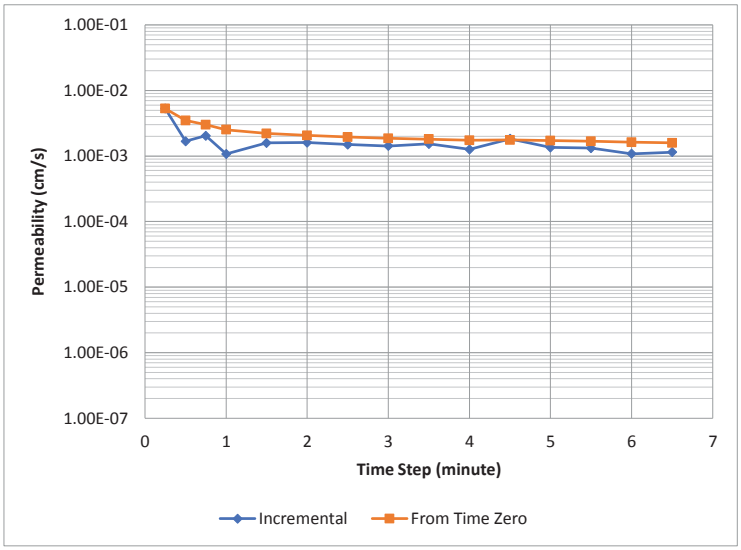
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Falling Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	7/23/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-103(P)**
Test Number:	**K-1**

Depth to top of Ground Water	25.5	ft
Casing Stickup	0.4	ft
Top Depth of Test Interval	30.0	ft
Bottom Depth of Test Interval	30.0	ft
Inside Diameter Pipe	4.25	in
D = Diameter, intake, sample	4.25	in
L = Length, intake, sample	0.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
1.87	0.0	0	24.0	--	--	--	--
2.48	0.3	15	23.4	1.73E-04	5.28E-03	1.73E-04	5.28E-03
2.67	0.5	30	23.2	5.49E-05	1.67E-03	1.14E-04	3.48E-03
2.90	0.8	45	23.0	6.71E-05	2.05E-03	9.85E-05	3.00E-03
3.02	1.0	60	22.9	3.53E-05	1.08E-03	8.27E-05	2.52E-03
3.37	1.5	90	22.5	5.20E-05	1.58E-03	7.24E-05	2.21E-03
3.72	2.0	120	22.2	5.28E-05	1.61E-03	6.75E-05	2.06E-03
4.04	2.5	150	21.9	4.90E-05	1.49E-03	6.38E-05	1.95E-03
4.34	3.0	180	21.6	4.66E-05	1.42E-03	6.10E-05	1.86E-03
4.66	3.5	210	21.2	5.04E-05	1.54E-03	5.94E-05	1.81E-03
4.92	4.0	240	21.0	4.15E-05	1.27E-03	5.72E-05	1.74E-03
5.29	4.5	270	20.6	6.00E-05	1.83E-03	5.75E-05	1.75E-03
5.56	5.0	300	20.3	4.45E-05	1.36E-03	5.62E-05	1.71E-03
5.82	5.5	330	20.1	4.34E-05	1.32E-03	5.50E-05	1.68E-03
6.03	6.0	360	19.9	3.54E-05	1.08E-03	5.34E-05	1.63E-03
6.25	6.5	390	19.7	3.75E-05	1.14E-03	5.22E-05	1.59E-03
Estimated Permeability				**1.1E-03**	**to**	**5.3E-03**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**1.6.E-03**	**(cm/sec)**	

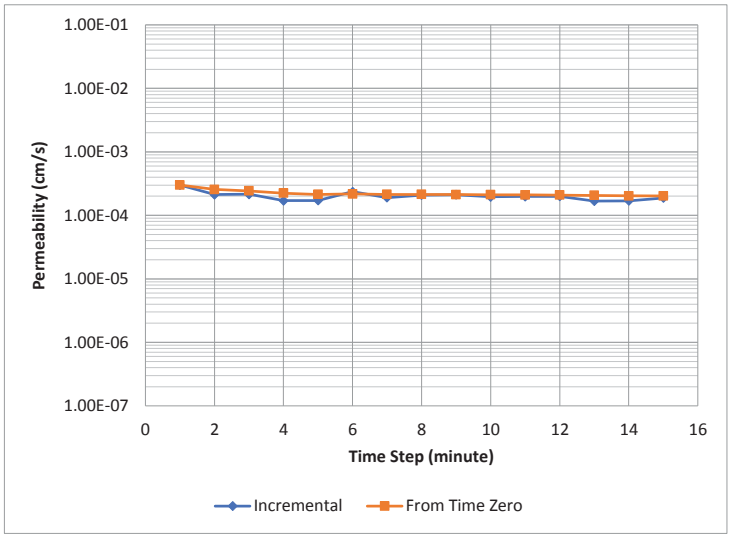
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case C.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/20/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-104(P)**
Test Number:	**K-1**

Depth to top of Ground Water	10.4	ft
Casing Stickup	2.1	ft
Top Depth of Test Interval	26.0	ft
Bottom Depth of Test Interval	27.0	ft
Inside Diameter Pipe	5.75	in
D = Diameter, intake, sample	5.75	in
L = Length, intake, sample	1.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
27.00	0.0	0	14.5	--	--	--	--
26.80	1.0	60	14.3	9.84E-06	3.00E-04	9.84E-06	3.00E-04
26.66	2.0	120	14.16	6.97E-06	2.13E-04	8.41E-06	2.56E-04
26.52	3.0	180	14.0	7.04E-06	2.15E-04	7.95E-06	2.42E-04
26.41	4.0	240	13.9	5.58E-06	1.70E-04	7.36E-06	2.24E-04
26.30	5.0	300	13.8	5.63E-06	1.72E-04	7.01E-06	2.14E-04
26.15	6.0	360	13.65	7.75E-06	2.36E-04	7.14E-06	2.18E-04
26.03	7.0	420	13.53	6.26E-06	1.91E-04	7.01E-06	2.14E-04
25.90	8.0	480	13.4	6.84E-06	2.09E-04	6.99E-06	2.13E-04
25.77	9.0	540	13.3	6.91E-06	2.11E-04	6.98E-06	2.13E-04
25.65	10.0	600	13.2	6.44E-06	1.96E-04	6.93E-06	2.11E-04
25.53	11.0	660	13.03	6.50E-06	1.98E-04	6.89E-06	2.10E-04
25.41	12.0	720	12.91	6.56E-06	2.00E-04	6.86E-06	2.09E-04
25.31	13.0	780	12.81	5.51E-06	1.68E-04	6.76E-06	2.06E-04
25.21	14.0	840	12.7	5.56E-06	1.69E-04	6.67E-06	2.03E-04
25.10	15.0	900	12.6	6.16E-06	1.88E-04	6.64E-06	2.02E-04
Estimated Permeability				**1.7E-04**	**to**	**3.0E-04**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**2.0.E-04**	**(cm/sec)**	

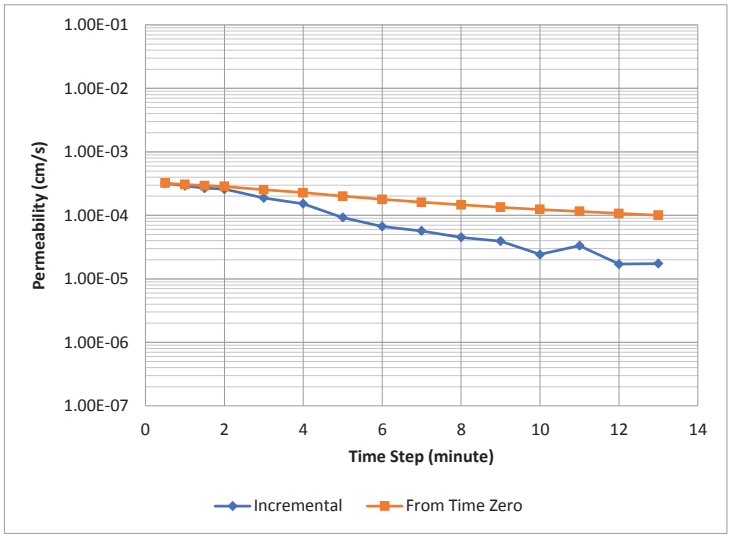
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/21/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-104(P)**
Test Number:	**K-2**

Depth to top of Ground Water	5.99	ft
Casing Stickup	2.9	ft
Top Depth of Test Interval	8.9	ft
Bottom Depth of Test Interval	14.5	ft
Inside Diameter Pipe	2.00	in
D = Diameter, intake, sample	5.75	in
L = Length, intake, sample	5.6	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
13.10	0.0	0	4.21	--	--	--	--
12.47	0.5	30	3.58	1.06E-05	3.22E-04	1.06E-05	3.22E-04
11.98	1.0	60	3.09	9.58E-06	2.92E-04	1.01E-05	3.07E-04
11.59	1.5	90	2.70	8.78E-06	2.68E-04	9.64E-06	2.94E-04
11.26	2.0	120	2.37	8.49E-06	2.59E-04	9.35E-06	2.85E-04
10.85	3.0	180	1.96	6.18E-06	1.88E-04	8.29E-06	2.53E-04
10.57	4.0	240	1.68	5.02E-06	1.53E-04	7.48E-06	2.28E-04
10.42	5.0	300	1.53	3.04E-06	9.28E-05	6.59E-06	2.01E-04
10.32	6.0	360	1.43	2.20E-06	6.71E-05	5.86E-06	1.79E-04
10.24	7.0	420	1.35	1.87E-06	5.71E-05	5.29E-06	1.61E-04
10.18	8.0	480	1.29	1.48E-06	4.51E-05	4.81E-06	1.47E-04
10.13	9.0	540	1.24	1.29E-06	3.92E-05	4.42E-06	1.35E-04
10.10	10.0	600	1.21	7.97E-07	2.43E-05	4.06E-06	1.24E-04
10.06	11.0	660	1.17	1.09E-06	3.33E-05	3.79E-06	1.15E-04
10.04	12.0	720	1.15	5.61E-07	1.71E-05	3.52E-06	1.07E-04
10.02	13.0	780	1.13	5.71E-07	1.74E-05	3.29E-06	1.00E-04
Estimated Permeability				**1.7E-05**	**to**	**3.2E-04**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**7.9.E-05**	**(cm/sec)**	

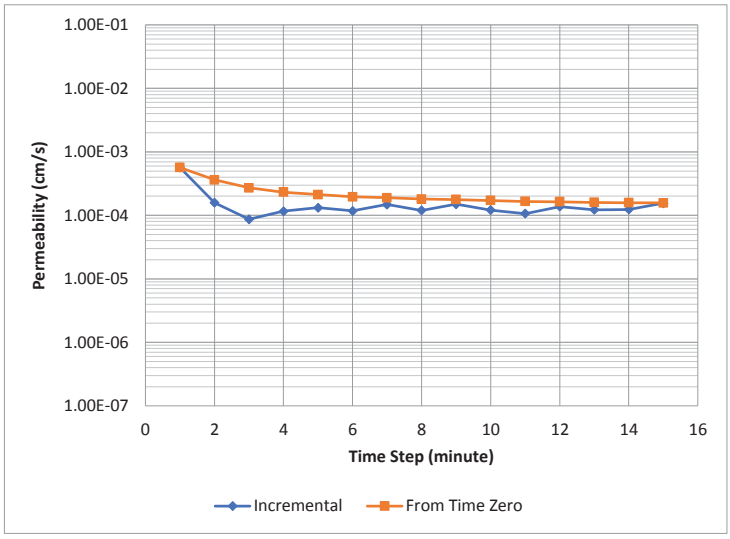
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/17/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-105A(P)**
Test Number:	**K-1**

Depth to top of Ground Water	4.8	ft
Casing Stickup	2.9	ft
Top Depth of Test Interval	21.0	ft
Bottom Depth of Test Interval	22.0	ft
Inside Diameter Pipe	5.75	in
D = Diameter, intake, sample	5.75	in
L = Length, intake, sample	1.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
23.15	0.0	0	15.45	--	--	--	--
22.75	1.0	60	15.05	1.86E-05	5.67E-04	1.86E-05	5.67E-04
22.64	2.0	120	14.94	5.20E-06	1.58E-04	1.19E-05	3.63E-04
22.58	3.0	180	14.88	2.85E-06	8.69E-05	8.88E-06	2.71E-04
22.50	4.0	240	14.80	3.82E-06	1.16E-04	7.62E-06	2.32E-04
22.41	5.0	300	14.71	4.32E-06	1.32E-04	6.96E-06	2.12E-04
22.33	6.0	360	14.63	3.87E-06	1.18E-04	6.44E-06	1.96E-04
22.23	7.0	420	14.53	4.86E-06	1.48E-04	6.22E-06	1.89E-04
22.15	8.0	480	14.45	3.91E-06	1.19E-04	5.93E-06	1.81E-04
22.05	9.0	540	14.35	4.92E-06	1.50E-04	5.82E-06	1.77E-04
21.97	10.0	600	14.27	3.96E-06	1.21E-04	5.63E-06	1.72E-04
21.90	11.0	660	14.20	3.49E-06	1.06E-04	5.44E-06	1.66E-04
21.81	12.0	720	14.11	4.51E-06	1.37E-04	5.36E-06	1.63E-04
21.73	13.0	780	14.03	4.03E-06	1.23E-04	5.26E-06	1.60E-04
21.65	14.0	840	13.95	4.05E-06	1.24E-04	5.17E-06	1.58E-04
21.55	15.0	900	13.85	5.10E-06	1.55E-04	5.17E-06	1.57E-04
Estimated Permeability				**8.7E-05**	**to**	**5.7E-04**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**1.4.E-04**	**(cm/sec)**	

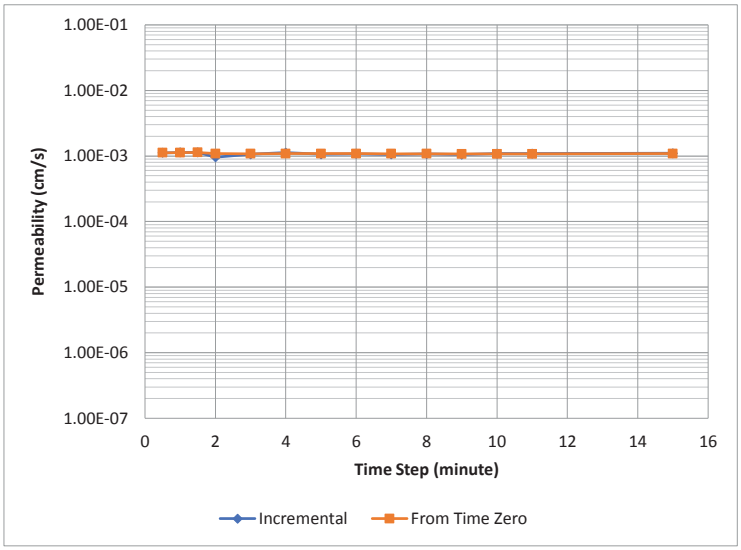
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/18/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-105A(P)**
Test Number:	**K-2**

Depth to top of Ground Water	9.18	ft
Casing Stickup	1.3	ft
Top Depth of Test Interval	52.0	ft
Bottom Depth of Test Interval	52.0	ft
Inside Diameter Pipe	5.38	in
D = Diameter, intake, sample	5.38	in
L = Length, intake, sample	0.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
45.40	0.0	0	34.92	--	--	--	--
45.10	0.5	30	34.62	3.68E-05	1.12E-03	3.68E-05	1.12E-03
44.80	1.0	60	34.32	3.71E-05	1.13E-03	3.70E-05	1.13E-03
44.50	1.5	90	34.02	3.74E-05	1.14E-03	3.71E-05	1.13E-03
44.25	2.0	120	33.77	3.15E-05	9.59E-04	3.57E-05	1.09E-03
43.70	3.0	180	33.22	3.50E-05	1.07E-03	3.55E-05	1.08E-03
43.13	4.0	240	32.65	3.69E-05	1.12E-03	3.58E-05	1.09E-03
42.60	5.0	300	32.12	3.49E-05	1.06E-03	3.56E-05	1.09E-03
42.07	6.0	360	31.59	3.55E-05	1.08E-03	3.56E-05	1.09E-03
41.56	7.0	420	31.08	3.47E-05	1.06E-03	3.55E-05	1.08E-03
41.05	8.0	480	30.57	3.53E-05	1.08E-03	3.55E-05	1.08E-03
40.56	9.0	540	30.08	3.45E-05	1.05E-03	3.53E-05	1.08E-03
40.06	10.0	600	29.58	3.57E-05	1.09E-03	3.54E-05	1.08E-03
39.57	11.0	660	29.09	3.56E-05	1.09E-03	3.54E-05	1.08E-03
37.66	15.0	900	27.18	3.62E-05	1.10E-03	3.56E-05	1.09E-03
Estimated Permeability				**9.6E-04**	**to**	**1.1E-03**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**1.1.E-03**	**(cm/sec)**	

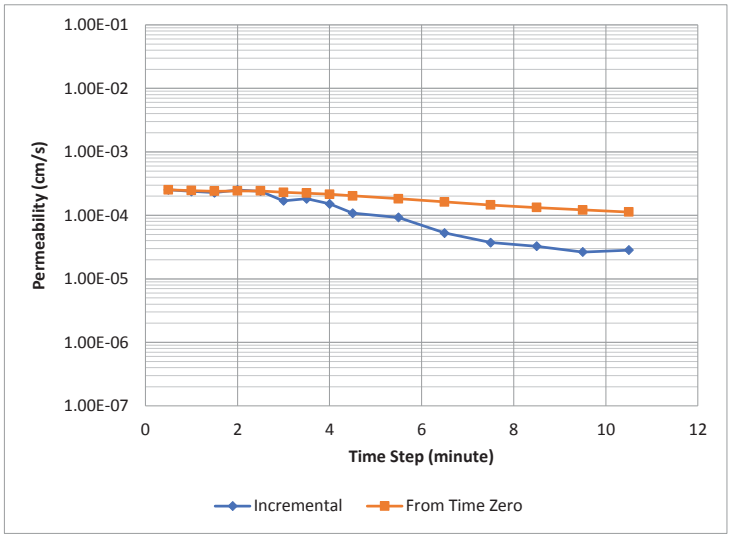
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case C.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/20/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-105A(P)**
Test Number:	**K-3**

Depth to top of Ground Water	9.74	ft
Casing Stickup	2.8	ft
Top Depth of Test Interval	53.0	ft
Bottom Depth of Test Interval	73.5	ft
Inside Diameter Pipe	2.00	in
D = Diameter, intake, sample	5.75	in
L = Length, intake, sample	20.5	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
21.11	0.0	0	8.57	--	--	--	--
18.70	0.5	30	6.16	8.29E-06	2.53E-04	8.29E-06	2.53E-04
17.05	1.0	60	4.51	7.83E-06	2.39E-04	8.06E-06	2.46E-04
15.89	1.5	90	3.35	7.47E-06	2.28E-04	7.87E-06	2.40E-04
14.95	2.0	120	2.41	8.27E-06	2.52E-04	7.97E-06	2.43E-04
14.30	2.5	150	1.76	7.90E-06	2.41E-04	7.95E-06	2.42E-04
13.95	3.0	180	1.41	5.57E-06	1.70E-04	7.56E-06	2.30E-04
13.65	3.5	210	1.11	6.01E-06	1.83E-04	7.33E-06	2.24E-04
13.45	4.0	240	0.91	4.99E-06	1.52E-04	7.04E-06	2.15E-04
13.33	4.5	270	0.79	3.55E-06	1.08E-04	6.65E-06	2.03E-04
13.16	5.5	330	0.62	3.04E-06	9.28E-05	6.00E-06	1.83E-04
13.08	6.5	390	0.54	1.74E-06	5.29E-05	5.34E-06	1.63E-04
13.03	7.5	450	0.49	1.22E-06	3.72E-05	4.79E-06	1.46E-04
12.99	8.5	510	0.45	1.07E-06	3.26E-05	4.35E-06	1.33E-04
12.96	9.5	570	0.42	8.67E-07	2.64E-05	3.99E-06	1.22E-04
12.93	10.5	630	0.39	9.31E-07	2.84E-05	3.70E-06	1.13E-04
Estimated Permeability				**2.6E-05**	**to**	**2.5E-04**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**1.0.E-04**	**(cm/sec)**	

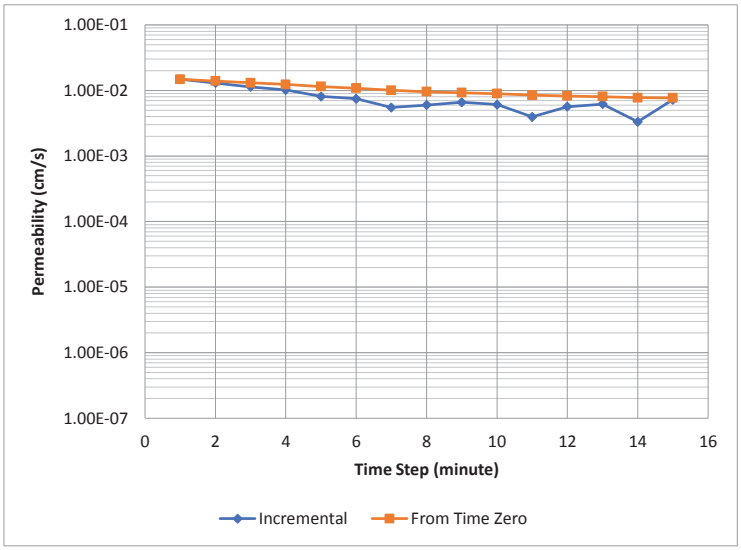
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/19/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-105B(P)**
Test Number:	**K-1**

Depth to top of Ground Water	8.24	ft
Casing Stickup	1.4	ft
Top Depth of Test Interval	12.6	ft
Bottom Depth of Test Interval	12.6	ft
Inside Diameter Pipe	5.38	in
D = Diameter, intake, sample	5.38	in
L = Length, intake, sample	0.0	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
11.65	0.0	0	2.01	--	--	--	--
11.24	1.0	60	1.60	4.86E-04	1.48E-02	4.86E-04	1.48E-02
10.95	2.0	120	1.31	4.26E-04	1.30E-02	4.56E-04	1.39E-02
10.74	3.0	180	1.10	3.73E-04	1.14E-02	4.28E-04	1.31E-02
10.58	4.0	240	0.94	3.35E-04	1.02E-02	4.05E-04	1.23E-02
10.47	5.0	300	0.83	2.65E-04	8.09E-03	3.77E-04	1.15E-02
10.38	6.0	360	0.74	2.45E-04	7.46E-03	3.55E-04	1.08E-02
10.32	7.0	420	0.68	1.80E-04	5.50E-03	3.30E-04	1.01E-02
10.26	8.0	480	0.62	1.97E-04	6.00E-03	3.13E-04	9.55E-03
10.20	9.0	540	0.56	2.17E-04	6.61E-03	3.03E-04	9.23E-03
10.15	10.0	600	0.51	1.99E-04	6.08E-03	2.92E-04	8.91E-03
10.12	11.0	660	0.48	1.29E-04	3.94E-03	2.78E-04	8.46E-03
10.08	12.0	720	0.44	1.86E-04	5.65E-03	2.70E-04	8.23E-03
10.04	13.0	780	0.40	2.03E-04	6.19E-03	2.65E-04	8.07E-03
10.02	14.0	840	0.38	1.09E-04	3.33E-03	2.54E-04	7.73E-03
9.98	15.0	900	0.34	2.37E-04	7.23E-03	2.53E-04	7.70E-03
Estimated Permeability				**3.3E-03**	**to**	**1.5E-02**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**7.1.E-03**	**(cm/sec)**	

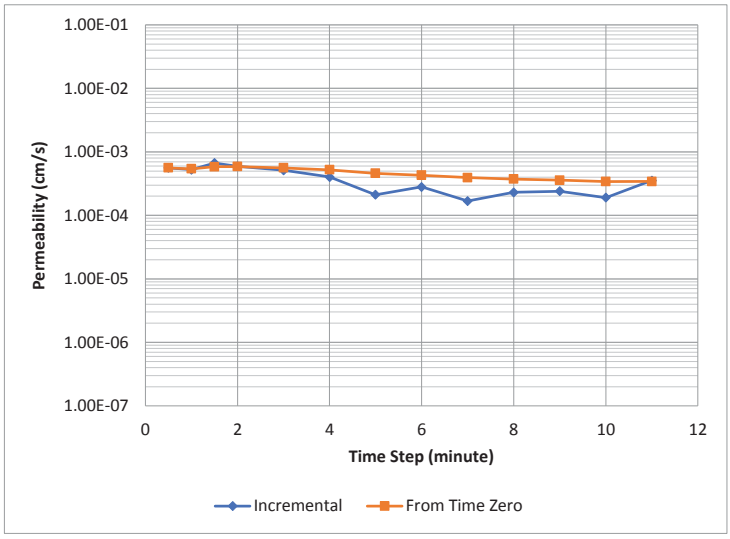
(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case C.

Rising Head Test - Data Reduction Calculation Sheet

Field Engineer/Geologist:	JNH	9/20/2018
Calculated By:	JNH	11/20/2018
Checked By:	ALC	11/20/2018
Approved By:	GOJ	11/26/2018

Project Number:	**18115**
Boring:	**B-105B(P)**
Test Number:	**K-2**

Depth to top of Ground Water	5.55	ft
Casing Stickup	2.9	ft
Top Depth of Test Interval	8.6	ft
Bottom Depth of Test Interval	12.6	ft
Inside Diameter Pipe	2.00	in
D = Diameter, intake, sample	5.75	in
L = Length, intake, sample	4.1	ft
M = Transformation Ratio	1.00	--



Depth to Water Surface In Pipe From Top of Riser Pipe	Time, t	Time, t	Length of Water Column at time, H	Incremental		From Time Zero	
				Permeability	Permeability	Permeability	Permeability
(ft)	(min)	(sec)	(ft)	(ft/s)	(cm/s)	(ft/s)	(cm/s)
12.25	0.0	0	3.8	--	--	--	--
11.48	0.5	30	3.0	1.83E-05	5.58E-04	1.83E-05	5.58E-04
10.90	1.0	60	2.5	1.72E-05	5.24E-04	1.78E-05	5.41E-04
10.32	1.5	90	1.9	2.19E-05	6.66E-04	1.91E-05	5.83E-04
9.92	2.0	120	1.5	1.95E-05	5.94E-04	1.92E-05	5.86E-04
9.42	3.0	180	1.0	1.68E-05	5.13E-04	1.84E-05	5.61E-04
9.15	4.0	240	0.7	1.32E-05	4.02E-04	1.71E-05	5.21E-04
9.04	5.0	300	0.6	6.92E-06	2.11E-04	1.51E-05	4.59E-04
8.92	6.0	360	0.5	9.20E-06	2.80E-04	1.41E-05	4.30E-04
8.86	7.0	420	0.4	5.52E-06	1.68E-04	1.29E-05	3.92E-04
8.79	8.0	480	0.3	7.57E-06	2.31E-04	1.22E-05	3.72E-04
8.73	9.0	540	0.3	7.85E-06	2.39E-04	1.17E-05	3.57E-04
8.69	10.0	600	0.2	6.24E-06	1.90E-04	1.12E-05	3.41E-04
8.63	11.0	660	0.2	1.16E-05	3.55E-04	1.12E-05	3.42E-04
Estimated Permeability				**1.7E-04**	**to**	**6.7E-04**	**(cm/sec)**
Geometric Mean of Incremental Permeability					**3.4.E-04**	**(cm/sec)**	

(1) Calculations above the water table are from USBR, Engineering Geology Field Manual, Volume 2, pp 162-165.
Calculations below the water table are from Hvorslev printed in Lambe & Whitman, Soil Mechanics, 1969, pp 285, case G.

APPENDIX F

FIELD INVEST WATER LEVEL DATA

18115 Hogchute Dam Safety Evaluation
Field Investigation Water Levels

	B-101(P)			B-102A(P)			B-103(P)			B-104(P)			B-105A(P)			B-105B(P)			Reservoir Level		
Top of Casing or PVC Riser Elevation (ft)	**Same as ground**			**Same as ground**			**Same as ground**			**9849.0**			**9868.5**			**9869.8**					
Ground Elevation (ft)	**9902.2 CAD survey elev**			**9902.1 CAD survey elev**			**9901.8 CAD survey elev**			**9846.1 CAD survey elev**			**9865.7 CAD survey elev**			**9866.9 CAD survey elev**			**9902.0 CAD survey elev**		
Note:																					
1. B-101, B-102A, and B-103: depths are referenced from top of ground surface. B-104, B-105A, and B-105B: depths are referenced from top of pvc riser pipe																					
Date	**Depth[1]**	**El**	**Notes**	**Depth[1]**	**El**	**Notes**	**Depth[1]**	**El**	**Notes**	**Depth[1]**	**El**	**Notes**	**Depth[1]**	**El**	**Notes**	**Depth[1]**	**El**	**Notes**	**Depth[1]**	**El**	**Notes**
7/23/2018		#N/A			#N/A		25.5	9876.3	During drilling. Time: 1605, 80° mostly sunny.		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
7/24/2018		#N/A		41.5	9860.6	During drilling. Time: 1425, 75° mostly sunny.		#N/A			#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
7/24/2018		#N/A		35.3	9866.9	During drilling, prior to K-1. Time: 1520, 75° mostly sunny.		#N/A			#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
7/25/2018		#N/A			#N/A		21.7	9880.1	In well, well developed 7/24/18. Time: 0845, 70° sunny.		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
7/26/2018	24.2	9878.0	During drilling. Perched zone in embankment? Time: 0840, 65° mostly sunny.		#N/A		22.5	9879.3	Time: 0805, 65° mostly sunny.		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
7/26/2018	55.3	9846.9	During drilling. Level in foundation. Time: 1405, 73° mostly sunny.		#N/A			#N/A			#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
7/27/2018	38.3	9863.9	During drilling. Time: 0803, 60° mostly sunny.	33.6	9868.5	In well, well developed 7/26/18. Time: 0813, 60° mostly sunny.	22.6	9879.2	Time: 0822, 60° mostly sunny.		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
7/28/2018	30.6	9871.7	In well. Prior to developing well. Time: 0745, 55° partly cloudy. Well developed after measurement.	33.6	9868.5	Time: 0845, 70° mostly cloudy.	22.6	9879.3	Time: 0840, 65° mostly cloudy.		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
8/3/2018	34.2	9868.0	~60°	32.9	9869.2	~60°	Dry	#N/A	~60°		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
8/9/2018	34.2	9868.0	Measured by SEO.	32.9	9869.2	Measured by SEO.	Dry	#N/A	Measured by SEO.		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
8/23/2018	27.7	9874.5	Measured by City.	33.6	9868.5	Measured by City.	22.5	9879.3	Measured by City.		#N/A			#N/A			#N/A		7.0	9895.0	Reservoir elevation assumed ~ 7' below crest.
9/6/2018	32.0	9870.2	Measured by City.	33.6	9868.5	Measured by City.	22.7	9879.1	Measured by City.		#N/A			#N/A			#N/A		17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.

	B-101(P)			B-102A(P)			B-103(P)			B-104(P)			B-105A(P)			B-105B(P)			Reservoir Level		
Date	Depth[1]	El	Notes	Depth[1]	El	Notes	Depth[1]	El	Notes	Depth[1]	El	Notes	Depth[1]	El	Notes	Depth[1]	El	Notes	Depth[1]	El	Notes
9/11/2018	36.9	9865.4	Measured by City.	35.2	9866.9	Measured by City.	22.5	9879.3	Measured by City.		#N/A			#N/A			#N/A		17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/13/2018	37.6	9864.6	Measured by City.	36.4	9865.8	Measured by City.	22.6	9879.2	Measured by City.		#N/A			#N/A			#N/A		17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/17/2018	38.6	9863.6	Time: 1036, 65° mostly sunny.	37.5	9864.6	Time: 1042, 65° mostly sunny.	22.6	9879.2	Time: 1050, 65° mostly sunny.		#N/A		4.8	9863.7	During drilling. Time: 1345, 70° sunny.		#N/A		17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/18/2018	38.8	9863.4	Time: 0655, 50° clear. Well pumped with submersible pump after reading.	37.7	9864.4	Time: 1732, 70° sunny.	22.6	9879.2	Measured by City.		#N/A		9.2	9859.3	In boring, measured prior to daily drilling. Time: 0740, 55° clear.		#N/A		17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/18/2018	39.7	9862.5	Time: 1726, 70° sunny. Measured after well pumped this morning. Recharging.		#N/A			#N/A			#N/A			#N/A			#N/A		17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/19/2018	39.0	9863.2	Time: 0724, 50° partly cloudy.	37.8	9864.3	Time: 0728, 50° partly cloudy.	22.6	9879.2	Time: 0732, 50° partly cloudy.		#N/A		9.1	9859.5	In boring, measured prior to daily activity. Time: 0755, 50° partly cloudy.	8.2	9861.5	During drilling. Time: 1120, 55° overcast.	17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/20/2018	39.1	9863.1	Time: 0727, 50° clear.	37.9	9864.2	Time: 0731, 50° clear.	22.6	9879.2	Time: 0735, 50° clear.	10.4	9838.6	During drilling. Time: 0825, 50° clear.	12.5	9856.0	In well. Prior to develop well. Time: 1445, 60° sunny. Developed well after measurement.	8.6	9861.2	In well. Prior to develop well. Time: 1448, 60° sunny. Developed well after measurement.	17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/21/2018	39.0	9863.2	Time: 0907, 50° sunny.	38.0	9864.1	Time: 0913, 50° sunny.	22.6	9879.2	Time: 0917, 50° sunny.	8.9	9840.1	In well. Prior to develop well. Time: 1315, 60° sunny. Developed well after measurement.	12.6	9855.9	Time: 0923, 50° sunny.	8.3	9861.4	Time: 0925, 50° sunny.	17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.
9/22/2018	39.1	9863.1	Time: 0704, 45° clear.	38.2	9863.9	Time: 0707, 45° clear.	22.6	9879.2	Time: 0711, 45° clear.	8.5	9840.5	Time: 0658, 45° clear.	12.6	9855.9	Time: 0652, 45° clear.	8.4	9861.4	Time: 0654, 45° clear.	17.0	9885.0	Reservoir elevation lowered 10' below normal, so ~ 17' below crest.

Jacquelyn Hagbery

From: Mark Mumby <mmumby@hrlcomp.com>
Sent: Monday, August 06, 2018 1:31 PM
To: Garrett Jackson
Cc: Jacquelyn Hagbery
Subject: Hogchute Dam Water Levels

Good afternoon Garrett,

Great news on the additional drilling. I did get the J-Plug installed on B 101.

I also collected a round of water levels which are as follows:

B-101 34.20 feet from TOC
B-102 32.94 feet From TOC
B-103 was dry.

I have us penciled in for the 17th of September

Regards

Mark

Mark Mumby, RPG | Drilling Program Manager
HRL Compliance Solutions, Inc.
2385 F 1/2 Road | Grand Junction, CO 81505
main 970.243.3271 Ex. 404 | mobile 970.260.1576
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Hogchute Dam (DAMID 420127) Monitoring Record

Piezometers

		B-101(P)		B-102(P)		B-103(P)		
		TOC[1] Elev.	151.00	TOC[1] Elev.	151.00	TOC[1] Elev.	151.00	
Date	Res. Elev.	DTW[2]	Water Elev.	DTW[2]	Water Elev.	DTW[2]	Water Elev.	Comments
7/28/2018	144.00	--	--	--	--	22.55	128.45	Top of Casing not yet surveyed, used dam crest elev = 151. Reservoir elevation assumed ~ 7' below crest.
8/3/2018	144.00	34.20	116.80	32.94	118.06	dry (22)	--	Top of Casing not yet surveyed, used dam crest elev = 151. Reservoir elevation assumed ~ 7' below crest.
8/9/2018	144.00	34.16	116.84	32.87	118.13	dry (22)	--	Top of Casing not yet surveyed, used dam crest elev = 151. Reservoir elevation assumed ~ 7' below crest.
8/23/2018	144.00	27.68	123.32	33.62	117.38	22.50	128.50	Top of Casing not yet surveyed, used dam crest elev = 151. Reservoir elevation assumed ~ 7' below crest.

Notes:

1 - TOC = Top of Casing elevation (to be surveyed by City)

2 - DTW = Depth to Water below TOC

City of Grand Junction
Carson Reservoir
Piezometer Readings

Date	9/6/2018	9/11/2018	9/13/2018	9/18/2018	9/20/2018			
Recorder	SC/CP	SC/CP	SC/CP	CP	CP			
PZ-N	22.7	22.5	22.6	22.58	22.58			
PZ-M	33.6	35.2	36.35	37.71	38.01			
PZ-S	32	36.85	37.62	41.2--RJH pumped down	39.02			
Toe Drain	5 gpm	5 gpm	4.25 gpm					
Level	47.6	41.8	38.35	38.35	38.35			

AF	637	425	320	320	320

APPENDIX G

LABORATORY TESTING RESULTS

Moisture Content and Density
ASTM D2216 and D7263

ADVANCED TERRA TESTING

Moisture and Density
ASTM D 2216 and ASTM D 7263

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	LOCATION	--
PROJECT NO.	18115		

BORING NO.	B-101
DEPTH	30-31'
SAMPLE NO.	CA-11
DATE SAMPLED	
DATE TESTED	10/26/18
TECHNICIAN	BDF
DESCRIPTION	
Mass of Wet Pan and Soil (g):	262.24
Mass of Dry Pan and Soil (g):	230.30
Mass of Pan (g):	6.71
Moisture (%):	**14.3**
Diameter (in):	1.94
Height (in):	2.22
Mass of Wet Soil and Ring (g):	956.35
Mass of Ring (g):	740.98
Wet Density (lbs/ft³):	**125.3**
Dry Density (lbs/ft³):	**109.6**
Wet Density (kg/m³):	**2007**
Dry Density (kg/m³):	**1756**

BORING NO.	
DEPTH	
SAMPLE NO.	
DATE SAMPLED	
DATE TESTED	
TECHNICIAN	
DESCRIPTION	
Mass of Wet Pan and Soil (g):	
Mass of Dry Pan and Soil (g):	
Mass of Pan (g):	
Moisture (%):	
Diameter (in):	
Height (in):	
Mass of Wet Soil and Ring (g):	
Mass of Ring (g):	
Wet Density (lbs/ft³):	
Dry Density (lbs/ft³):	
Wet Density (kg/m³):	
Dry Density (kg/m³):	
NOTES	Significant amount of filling required.

Data entry by:	CAL	Date:	10/29/2018
Checked by:	KMS	Date:	10/29/18
File name:	2679130__Moisture and Density ASTM D7236_1.xls		

ADVANCED TERRA TESTING

Moisture and Density
ASTM D 2216 and ASTM D 7263

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	LOCATION	--
PROJECT NO.	18115		

BORING NO.	B-102A
DEPTH	44-45'
SAMPLE NO.	CA-20
DATE SAMPLED	07/24/18
DATE TESTED	10/19/18
TECHNICIAN	SKS
DESCRIPTION	
Mass of Wet Pan and Soil (g):	75.44
Mass of Dry Pan and Soil (g):	61.76
Mass of Pan (g):	7.92
Moisture (%):	**25.4**
Diameter (in):	2.03
Height (in):	4.20
Mass of Wet Soil and Ring (g):	535.00
Mass of Ring (g):	108.80
Wet Density (lbs/ft³):	**119.8**
Dry Density (lbs/ft³):	**95.6**
Wet Density (kg/m³):	**1920**
Dry Density (kg/m³):	**1531**

BORING NO.	
DEPTH	
SAMPLE NO.	
DATE SAMPLED	
DATE TESTED	
TECHNICIAN	
DESCRIPTION	
Mass of Wet Pan and Soil (g):	
Mass of Dry Pan and Soil (g):	
Mass of Pan (g):	
Moisture (%):	
Diameter (in):	
Height (in):	
Mass of Wet Soil and Ring (g):	
Mass of Ring (g):	
Wet Density (lbs/ft³):	
Dry Density (lbs/ft³):	
Wet Density (kg/m³):	
Dry Density (kg/m³):	
NOTES	

Data entry by:	SKS	Date:	10/23/2018
Checked by:	KMS	Date:	10/24/18
File name:	2679130__Moisture and Density ASTM D7236_0.xls		

ADVANCED TERRA TESTING

Moisture and Density
ASTM D 2216 and ASTM D 7263

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	LOCATION	--
PROJECT NO.	18115		

BORING NO.	B-103
DEPTH	15-16'
SAMPLE NO.	CA-8
DATE SAMPLED	
DATE TESTED	10/26/18
TECHNICIAN	BDF
DESCRIPTION	
Mass of Wet Pan and Soil (g):	598.84
Mass of Dry Pan and Soil (g):	566.35
Mass of Pan (g):	266.20
Moisture (%):	**10.8**
Diameter (in):	1.94
Height (in):	2.50
Mass of Wet Soil and Ring (g):	648.18
Mass of Ring (g):	422.73
Wet Density (lbs/ft³):	**116.5**
Dry Density (lbs/ft³):	**105.1**
Wet Density (kg/m³):	**1866**
Dry Density (kg/m³):	**1683**

BORING NO.	
DEPTH	
SAMPLE NO.	
DATE SAMPLED	
DATE TESTED	
TECHNICIAN	
DESCRIPTION	
Mass of Wet Pan and Soil (g):	
Mass of Dry Pan and Soil (g):	
Mass of Pan (g):	
Moisture (%):	
Diameter (in):	
Height (in):	
Mass of Wet Soil and Ring (g):	
Mass of Ring (g):	
Wet Density (lbs/ft³):	
Dry Density (lbs/ft³):	
Wet Density (kg/m³):	
Dry Density (kg/m³):	
NOTES	Filling required for density.

Data entry by:	SPH	Date:	10/30/2018
Checked by:	KMS	Date:	10/31/18
File name:	2679130__Moisture and Density ASTM D7236_2.xls		

Atterberg Limits
ASTM D4318

ADVANCED TERRA TESTING

Atterberg Limits
ASTM D 4318

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	25-40'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-13
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	SAMPLED BY	--
DATE TESTED	11/06/18	DESCRIPTION	--
TECHNICIAN	SPH		

Plastic Limits

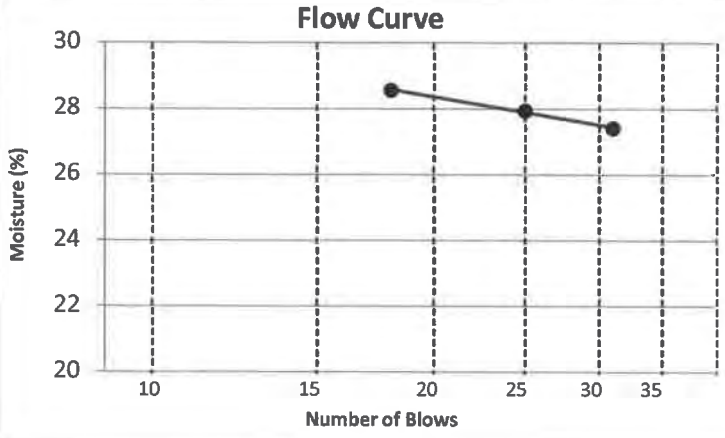
Mass of Wet Pan and Soil (g):	9.93	9.90
Mass of Dry Pan and Soil (g):	8.68	8.65
Mass of Pan (g):	1.16	1.15
Moisture (%)	**16.7**	**16.7**

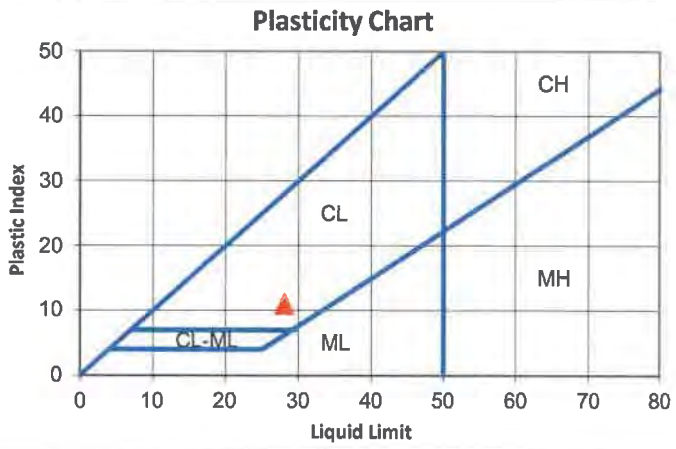
Liquid Limits

Number of Blows	18	25	31
Mass of Wet Pan and Soil (g):	8.94	9.50	9.99
Mass of Dry Pan and Soil (g):	7.21	7.68	8.10
Mass of Pan (g):	1.13	1.14	1.18
Moisture (%)	**28.6**	**27.9**	**27.4**

Plastic Index

Plastic Limit:	**17**	Atterberg Classification:	CL
Liquid Limit:	**28**	Method:	A
Plastic Index:	**11**		





NOTES

Data entry by: SPH Date: 11/7/2018
Checked by: KMS Date: 11/8/18
File name: 2679130__Atterberg ASTM D4318_1.xlsm

ADVANCED TERRA TESTING

Atterberg Limits
ASTM D 4318

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	51-65'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-20
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	SAMPLED BY	--
DATE TESTED	11/07/18	DESCRIPTION	--
TECHNICIAN	AD		

Plastic Limits

Mass of Wet Pan and Soil (g):	7.61	7.53
Mass of Dry Pan and Soil (g):	6.77	6.71
Mass of Pan (g):	1.09	1.11
Moisture (%)	**14.9**	**14.8**

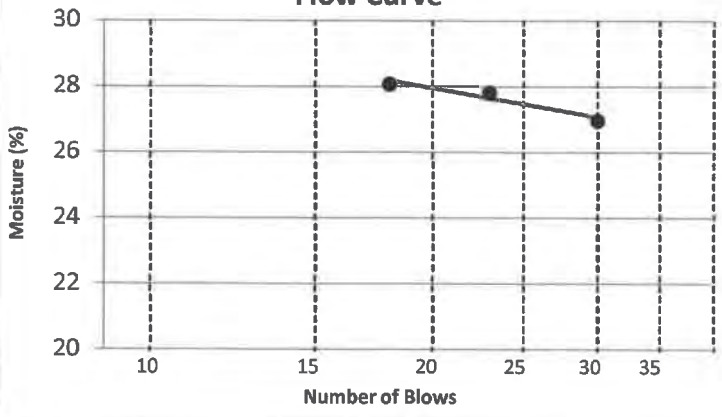
Liquid Limits

Number of Blows	18	23	30
Mass of Wet Pan and Soil (g):	11.55	10.96	11.78
Mass of Dry Pan and Soil (g):	9.27	8.83	9.52
Mass of Pan (g):	1.14	1.14	1.13
Moisture (%)	**28.1**	**27.8**	**27.0**

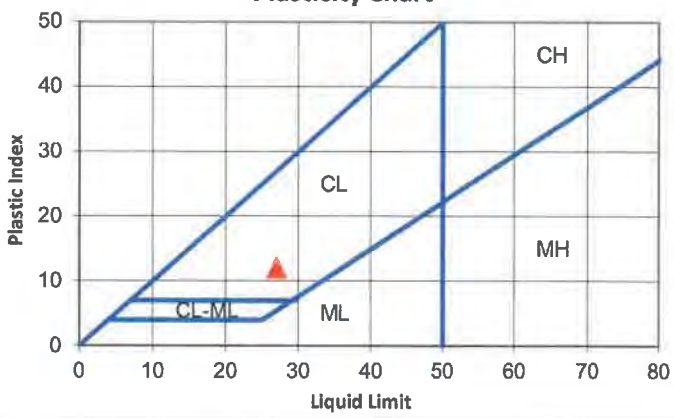
Plastic Index

Plastic Limit:	**15**	Atterberg Classification:	CL
Liquid Limit:	**27**	Method:	A
Plastic Index:	**12**		

Flow Curve



Plasticity Chart



NOTES

Data entry by: KMS Date: 11/8/2018

Checked by: KJ2 Date: 11/8/18

File name: 2679130__Atterberg ASTM D4318_4.xlsm

ADVANCED TERRA TESTING

Atterberg Limits
ASTM D 4318

CLIENT	RJH Consultants	BORING NO.	B-102A
JOB NO.	2679-130	DEPTH	18-41.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-10
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	SAMPLED BY	--
DATE TESTED	11/05/18	DESCRIPTION	--
TECHNICIAN	SPH		

Plastic Limits

Mass of Wet Pan and Soil (g):	9.17	9.20
Mass of Dry Pan and Soil (g):	8.02	8.03
Mass of Pan (g):	1.17	1.07
Moisture (%)	**16.7**	**16.7**

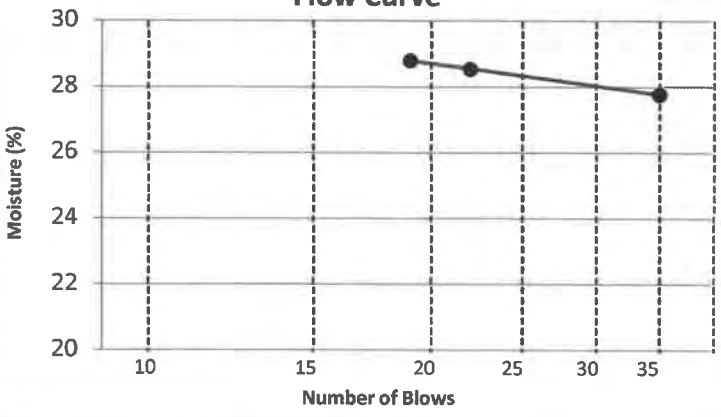
Liquid Limits

Number of Blows	19	22	35
Mass of Wet Pan and Soil (g):	13.22	14.23	13.44
Mass of Dry Pan and Soil (g):	10.51	11.32	10.76
Mass of Pan (g):	1.12	1.14	1.10
Moisture (%)	**28.8**	**28.5**	**27.8**

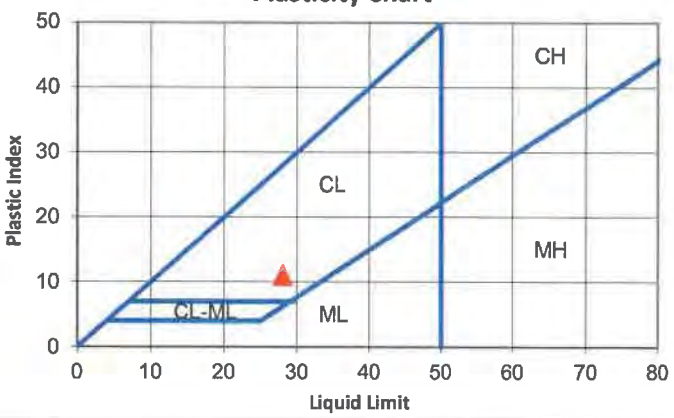
Plastic Index

Plastic Limit:	**17**	Atterberg Classification:	CL
Liquid Limit:	**28**	Method:	A
Plastic Index:	**11**		

Flow Curve



Plasticity Chart



NOTES

Data entry by: SPH Date: 11/7/2018
Checked by: KMS Date: 11/7/18
File name: 2679130__Atterberg ASTM D4318_2.xlsm

ADVANCED TERRA TESTING

Atterberg Limits
ASTM D 4318

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	10-17.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-11
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	SAMPLED BY	--
DATE TESTED	11/07/18	DESCRIPTION	--
TECHNICIAN	CAL		

Plastic Limits

Mass of Wet Pan and Soil (g):	9.35	8.35
Mass of Dry Pan and Soil (g):	8.24	7.38
Mass of Pan (g):	1.15	1.13
Moisture (%)	**15.7**	**15.6**

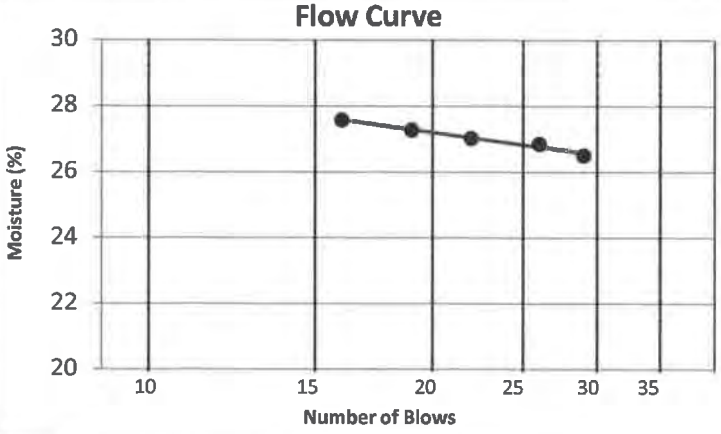
Liquid Limits

Number of Blows	16	19	22	26	29
Mass of Wet Pan and Soil (g):	10.62	10.48	9.39	11.05	10.09
Mass of Dry Pan and Soil (g):	8.57	8.48	7.63	8.96	8.22
Mass of Pan (g):	1.14	1.18	1.15	1.15	1.14
Moisture (%)	**27.6**	**27.3**	**27.0**	**26.9**	**26.5**

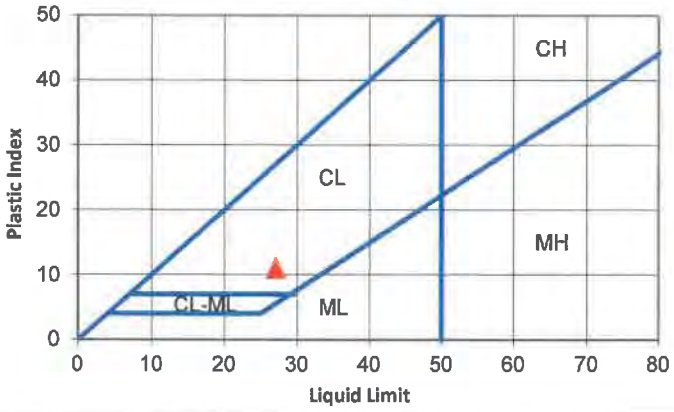
Plastic Index

Plastic Limit:	**16**	Atterberg Classification:	CL
Liquid Limit:	**27**	Method:	A
Plastic Index:	**11**		

Flow Curve



Plasticity Chart



NOTES

Data entry by: CAL
Checked by: KMS
File name: 2679130__Atterberg ASTM D4318_3.xlsm

Date: 11/8/2018
Date: 11/8/18

ADVANCED TERRA TESTING

Atterberg Limits
ASTM D 4318

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	SAMPLED BY	--
DATE TESTED	10/25/18	DESCRIPTION	--
TECHNICIAN	KJT		

Plastic Limits

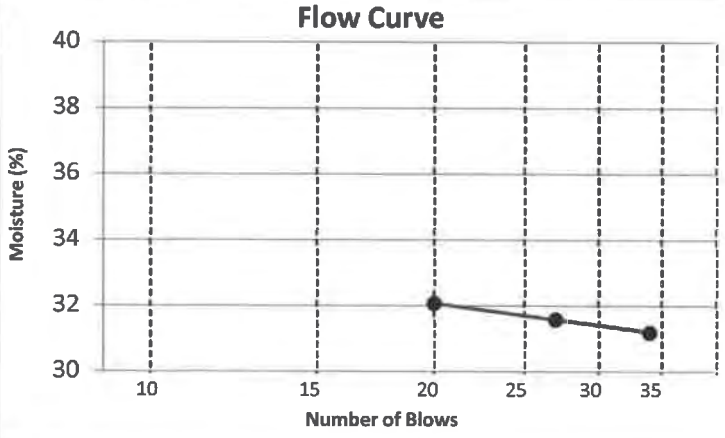
Mass of Wet Pan and Soil (g):	7.69	7.79
Mass of Dry Pan and Soil (g):	6.64	6.74
Mass of Pan (g):	1.13	1.15
Moisture (%)	**19.0**	**18.7**

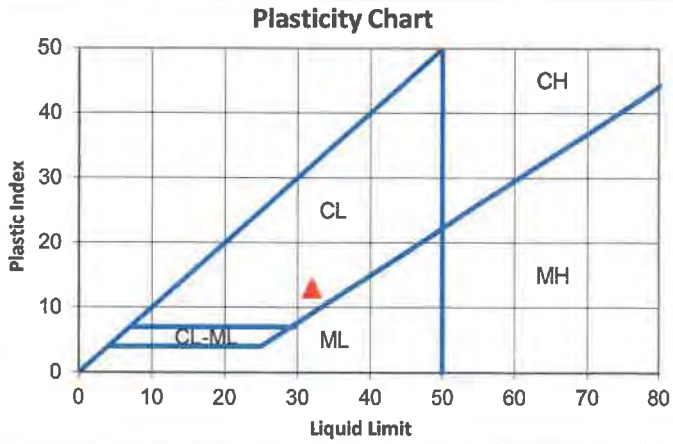
Liquid Limits

Number of Blows	20	27	34
Mass of Wet Pan and Soil (g):	10.64	11.64	10.28
Mass of Dry Pan and Soil (g):	8.32	9.12	8.09
Mass of Pan (g):	1.08	1.11	1.09
Moisture (%)	**32.1**	**31.6**	**31.2**

Plastic Index

Plastic Limit:	**19**	Atterberg Classification:	CL
Liquid Limit:	**32**	Method:	A
Plastic Index:	**13**		





NOTES

Data entry by: SPH Date: 10/26/2018

Checked by: CJL Date: 10/26/18

File name: 2679130__Atterberg ASTM D4318_0.xlsm

Grain Size Analysis
ASTM D6913

ADVANCED TERRA TESTING

Grain Size Analysis
ASTM D 6913

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	25-40'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-13
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/01/18		
TECHNICIAN	AD		

Hygroscopic Moisture of Fines

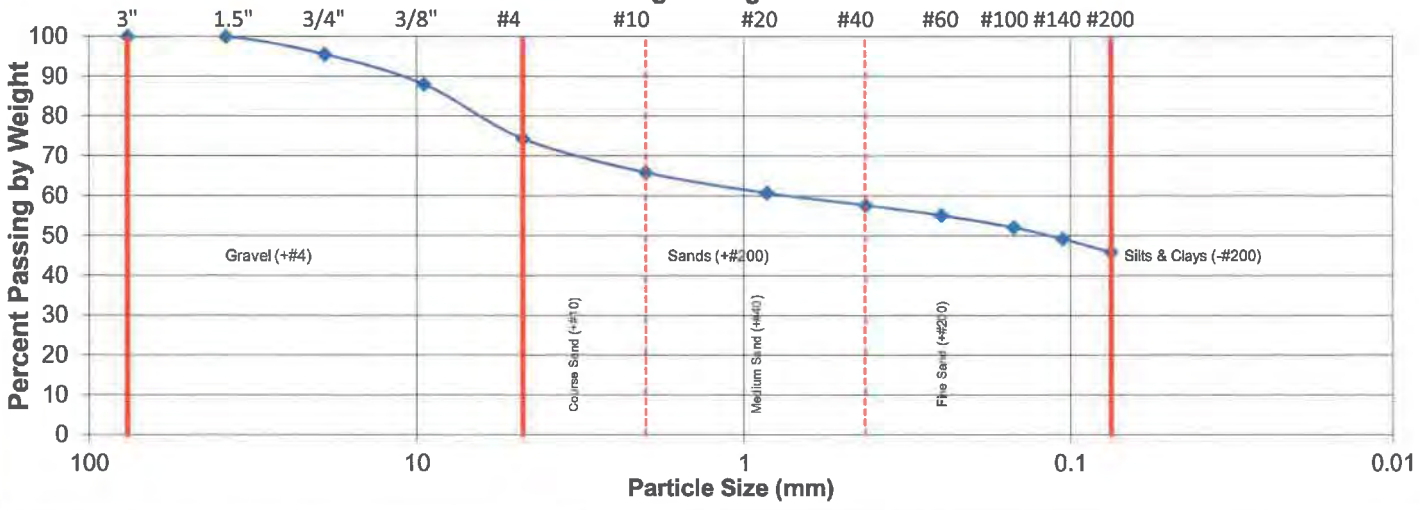
Mass Wet Pan and Soil (g): 1409.38
Mass Dry Pan and Soil (g): 1399.33
Mass of Pan (g): 782.51
Moisture (%): **1.6**

Sample Data

Total Wet Mass of Sample (g): 23501.7
Total Dry Mass of Sample (g): 23169.2
Split Fraction: 3/8"
Mass of Sub-Sample Fraction (g): 626.87

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	--	--	--	--	**--**
1.5"	38.1	0.0	--	--	--	**100%**
3/4"	19.05	1014.6	--	1014.6	1.00	**95.6**
3/8"	9.53	1748.4	--	1748.4	1.00	**88.1**
#4	4.75	95.3	--	95.3	0.88	**74.5**
#10	2.00	59.5	--	59.5	0.88	**66.0**
#20	0.850	36.0	--	36.0	0.88	**60.8**
#40	0.425	21.7	--	21.7	0.88	**57.7**
#60	0.250	17.9	--	17.9	0.88	**55.2**
#100	0.150	21.1	--	21.1	0.88	**52.2**
#140	0.106	20.1	--	20.1	0.88	**49.3**
#200	0.075	23.6	--	23.6	0.88	**45.9**

Percent Passing vs Log of Particle Size



USCS Classification ASTM D 2487

Atterberg Classification: CL
Group Symbol: SC
USCS Classification: Clayey Sand With Gravel

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: SPH Date: 11/7/2018
Checked by: KMG Date: 11/7/18
File name: 2679130__Grain Size Analysis ASTM D6913_2.xlsm

ADVANCED TERRA TESTING

Grain Size Analysis
ASTM D 6913

CLIENT	RJH Consultants	BORING NO.	B-102A
JOB NO.	2679-130	DEPTH	18-41.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-10
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/05/18		
TECHNICIAN	AD		

Hygroscopic Moisture of Fines

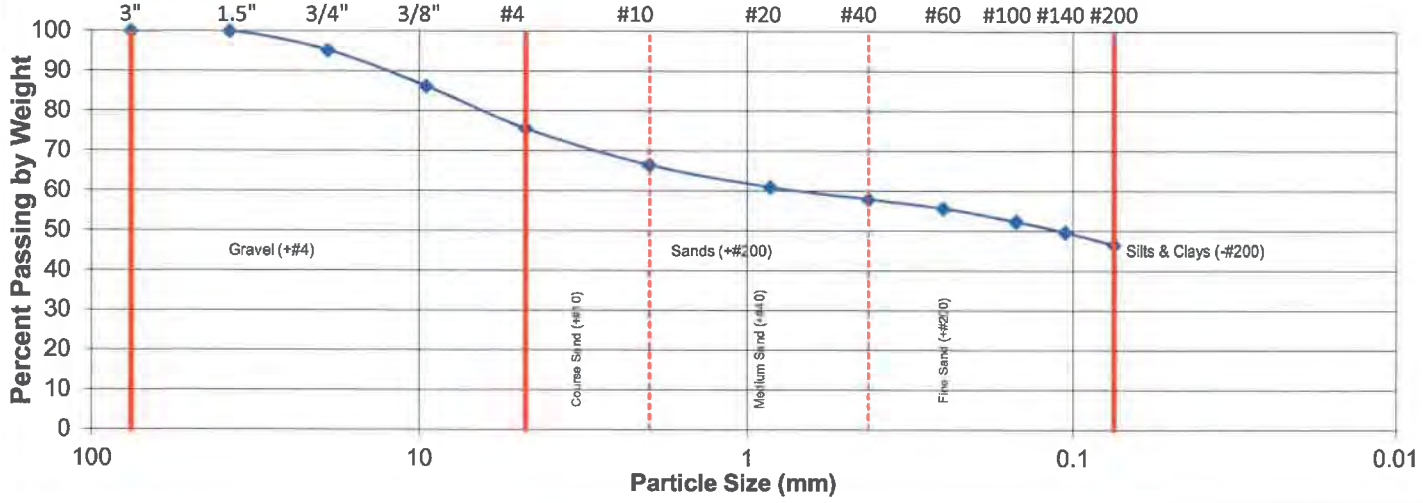
Mass Wet Pan and Soil (g): 1809.96
Mass Dry Pan and Soil (g): 1794.70
Mass of Pan (g): 843.18
Moisture (%): **1.6**

Sample Data

Total Wet Mass of Sample (g): 24771.5
Total Dry Mass of Sample (g): 24433.6
Split Fraction: 3/8"
Mass of Sub-Sample Fraction (g): 966.78

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	--	--	--	--	**--**
1.5"	38.1	0.0	--	--	--	**100%**
3/4"	19.05	1165.8	--	1165.8	1.00	**95.2**
3/8"	9.53	2201.0	--	2201.0	1.00	**86.2**
#4	4.75	116.4	--	116.4	0.86	**75.7**
#10	2.00	100.5	--	100.5	0.86	**66.6**
#20	0.850	61.0	--	61.0	0.86	**61.0**
#40	0.425	33.7	--	33.7	0.86	**58.0**
#60	0.250	24.9	--	24.9	0.86	**55.7**
#100	0.150	36.7	--	36.7	0.86	**52.4**
#140	0.106	29.3	--	29.3	0.86	**49.7**
#200	0.075	35.2	--	35.2	0.86	**46.6**

Percent Passing vs Log of Particle Size



USCS Classification ASTM D 2487

Atterberg Classification: CL
Group Symbol: SC
USCS Classification: Clayey Sand With Gravel

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: SPH Date: 11/7/2018
Checked by: KMS Date: 11/7/18
File name: 2679130__Grain Size Analysis ASTM D6913_3.xlsm

ADVANCED TERRA TESTING

Grain Size Analysis
ASTM D 6913

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	27.5-29.9
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	u-14
PROJECT NO.	18115	DATE SAMPLED	07/23/18
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/30/18		
TECHNICIAN	KJT		

Hygroscopic Moisture of Fines

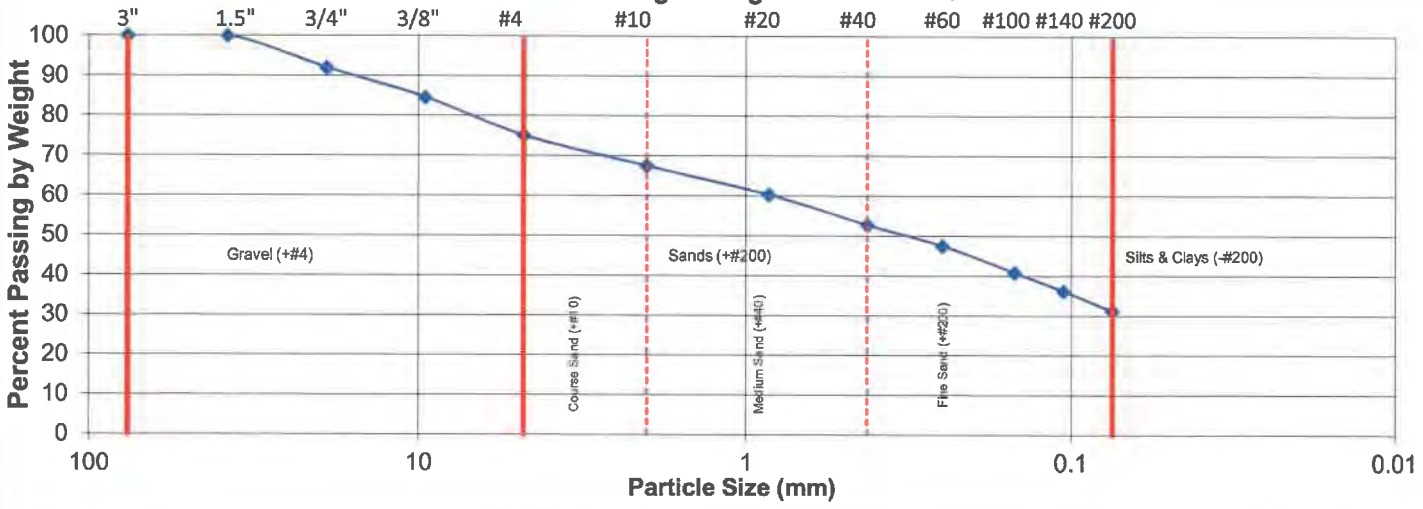
Mass Wet Pan and Soil (g): 1325.03
Mass Dry Pan and Soil (g): 1317.04
Mass of Pan (g): 811.12
Moisture (%): **1.6**

Sample Data

Total Wet Mass of Sample (g): 687.1
Total Dry Mass of Sample (g): 679.0
Split Fraction: #4
Mass of Sub-Sample Fraction (g): 513.86

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	--	--	--	--	**--**
1.5"	38.1	0.0	--	--	--	**100%**
3/4"	19.05	53.9	--	53.9	1.00	**92.1**
3/8"	9.53	50.2	--	50.2	1.00	**84.7**
#4	4.75	64.3	--	64.3	1.00	**75.2**
#10	2.00	51.4	--	51.4	0.75	**67.6**
#20	0.850	48.6	--	48.6	0.75	**60.3**
#40	0.425	50.7	--	50.7	0.75	**52.8**
#60	0.250	35.2	--	35.2	0.75	**47.6**
#100	0.150	44.8	--	44.8	0.75	**40.9**
#140	0.106	31.1	--	31.1	0.75	**36.3**
#200	0.075	33.7	--	33.7	0.75	**31.3**

Percent Passing vs Log of Particle Size



USCS Classification ASTM D 2487

Atterberg Classification: --
Group Symbol: --
USCS Classification: --

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: SPH Date: 11/5/2018
Checked by: CAL Date: 11/5/18
File name: 2679130__Grain Size Analysis ASTM D6913_1.xlsm

ADVANCED TERRA TESTING

Grain Size Analysis
ASTM D 6913

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32.0-33.0
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	09/20/18
LOCATION	--	DESCRIPTION	Lean Clay w/ Sand
DATE TESTED	10/19/18		
TECHNICIAN	CAL		

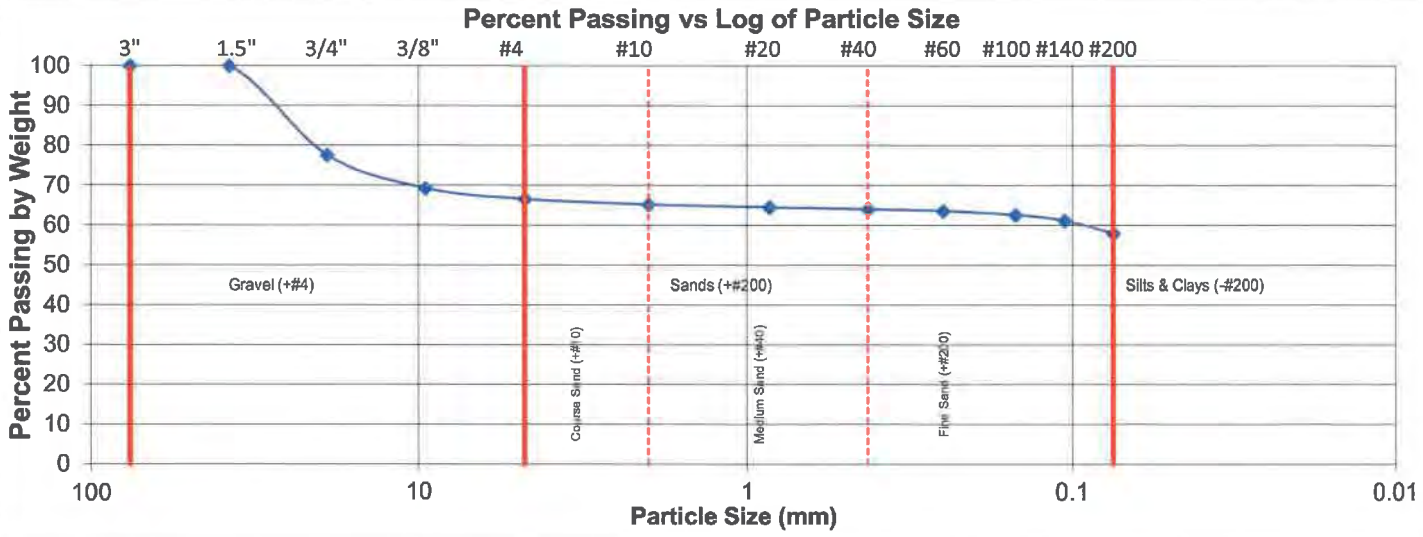
Hygroscopic Moisture

Mass Wet Pan and Soil (g): 531.36
Mass Dry Pan and Soil (g): 466.78
Mass of Pan (g): 139.65
Moisture (%): **19.7**

Sample Data

Total Wet Mass of Sample (g): 391.7
Total Dry Mass of Sample (g): 327.1

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	--	--	--	--	**--**
1.5"	38.1	--	--	--	--	**--**
3/4"	19.05	73.1	--	73.1	1.00	**77.6**
3/8"	9.53	27.1	--	27.1	1.00	**69.4**
#4	4.75	9.0	--	9.0	1.00	**66.6**
#10	2.00	4.4	--	4.4	1.00	**65.2**
#20	0.850	2.2	--	2.2	1.00	**64.6**
#40	0.425	1.5	--	1.5	1.00	**64.1**
#60	0.250	1.4	--	1.4	1.00	**63.7**
#100	0.150	3.3	--	3.3	1.00	**62.7**
#140	0.106	4.6	--	4.6	1.00	**61.3**
#200	0.075	10.5	--	10.5	1.00	**58.1**



USCS Classification ASTM D 2487

Atterberg Classification: CL
Group Symbol: CL
USCS Classification: Gravelly Lean Clay

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: SPH Date: 10/26/2018
Checked by: CW Date: 10/26/18
File name: 2679130__Grain Size Analysis ASTM D6913_0.xlsm

Grain Size Analysis with Hydrometer
ASTM D6913 and D7928

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	51-65'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-20
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hygroscopic Moisture of Fines

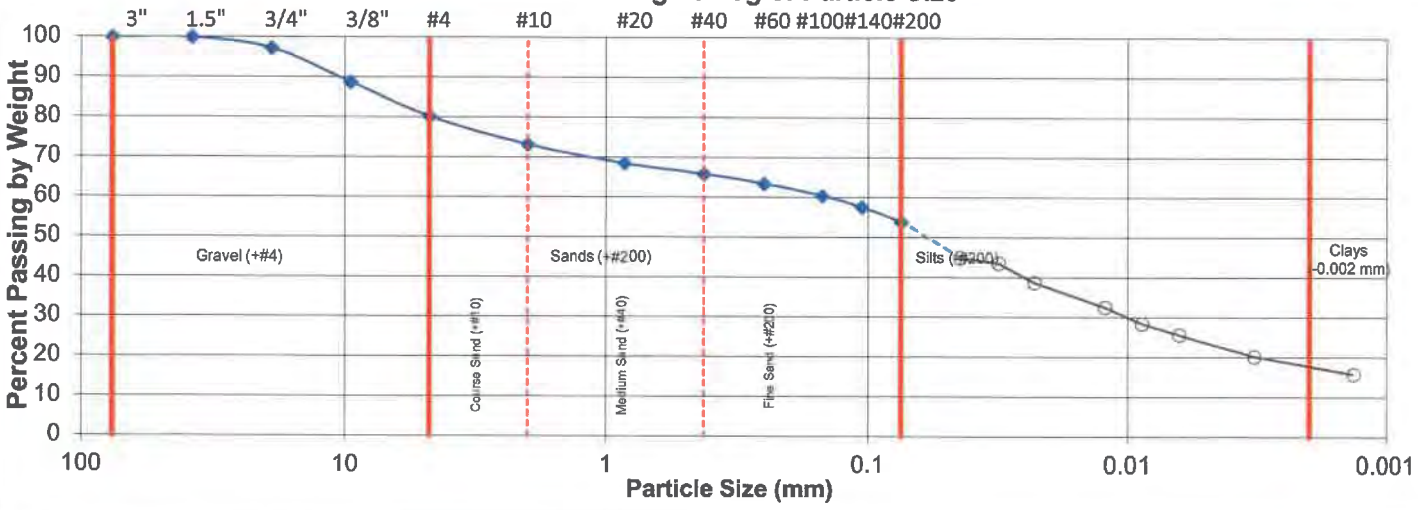
Mass Wet Pan and Soil (g): 116.73
Mass Dry Pan and Soil (g): 114.50
Mass of Pan (g): 3.16
Moisture (%): **2.0**

Sample Data

Total Wet Mass of Sample (g): 19133.2
Total Dry Mass of Sample (g): 18857.9
Split Fraction: #10 3/8"
Mass of Sub-Sample Fraction (g): 60.91 1123.80

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	0.0	--	--	--	**--**
1.5"	38.1	0.0	--	--	--	**--**
3/4"	19.05	507.2	--	507.2	1.00	**97.3**
3/8"	9.53	1605.3	--	1605.3	1.00	**88.8**
#4	4.75	106.8	--	106.8	0.89	**80.2**
#10	2.00	87.4	--	87.39	0.888	**73.2**
#20	0.850	3.8	--	3.80	0.731	**68.5**
#40	0.425	2.2	--	2.15	0.731	**65.9**
#60	0.250	2.0	--	1.95	0.731	**63.5**
#100	0.150	2.5	--	2.50	0.731	**60.5**
#140	0.106	2.3	--	2.31	0.731	**57.6**
#200	0.075	3.0	--	2.98	0.731	**54.0**

Percent Passing vs Log of Particle Size



USCS Classification ASTM D 2487

Atterberg Classification: CL
Group Symbol: CL
USCS Classification: Sandy Lean Clay With Gravel

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: KMS Date: 11/8/2018
Checked by: KP Date: 11/8/18
File name: 2679130__Grain Size with Hydrometer ASTM D6913 D7928_0.xlsm

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	51-65'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-20
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hydrometer and Flask Parameters

Hydrometer ID:	0805	Flask ID:	1186
Average Mass Offset (g/L):	9.87	Flask Volume (cm^3):	996.8
Hydrometer Bulb Volume (cm^3):	56.50	Flask Surface Area (cm^2):	28.60
Meniscus Correction (g/L):	1.00	Assumed Specific Gravity	2.65
H_b (cm):	24.5	Hydrometer Type:	152H
H_{cb} (cm):	6.8		
H_s (cm):	8.2	Percent Finer by Mass at 2 µm:	**17.3**

Hydrometer Data

Elapsed Time (minutes)	Hydrometer Reading (g/L)	Offset Reading (g/L)	Temperature (°C)	Effective Depth (cm)	Maximum Particle Diameter in Suspension (mm)	Percent Finer by Mass (%)
1	38.00	5.34	23.1	10.62	0.044	**44.9**
2	37.00	5.34	23.1	10.79	0.032	**43.5**
4	33.50	5.34	23.1	11.36	0.023	**38.7**
15	29.00	5.34	23.1	12.10	0.012	**32.5**
30	26.00	5.34	23.1	12.60	0.009	**28.4**
60	24.00	5.34	23.1	12.93	0.006	**25.7**
240	20.00	5.30	23.2	13.58	0.003	**20.2**
1440	17.00	5.53	22.6	14.08	0.001	**15.8**

NOTES:

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	10-17.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-11
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hygroscopic Moisture of Fines

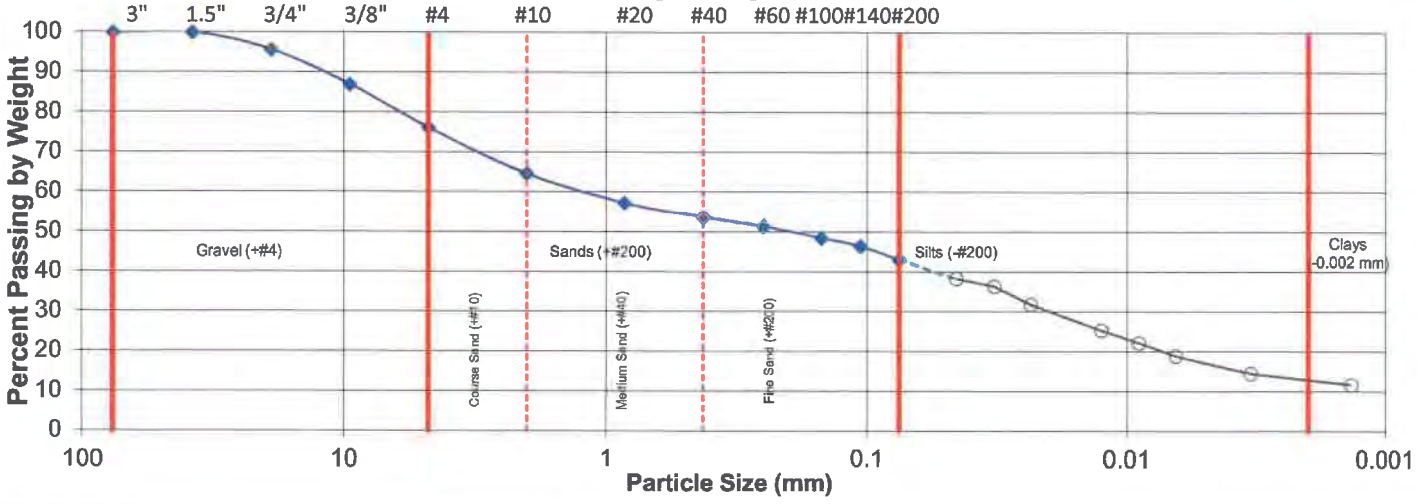
Mass Wet Pan and Soil (g): 121.30
Mass Dry Pan and Soil (g): 119.10
Mass of Pan (g): 3.15
Moisture (%): **1.9**

Sample Data

Total Wet Mass of Sample (g): 26704.1
Total Dry Mass of Sample (g): 26381.7
Split Fraction: #10 3/8"
Mass of Sub-Sample Fraction (g): 57.98 1427.90

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	0.0	--	--	--	**--**
1.5"	38.1	0.0	--	--	--	**--**
3/4"	19.05	1138.5	--	1138.5	1.00	**95.7**
3/8"	9.53	2287.5	--	2287.5	1.00	**87.0**
#4	4.75	175.9	--	175.9	0.87	**76.1**
#10	2.00	186.6	--	186.55	0.870	**64.6**
#20	0.850	6.5	--	6.52	0.646	**57.2**
#40	0.425	3.1	--	3.12	0.646	**53.7**
#60	0.250	2.0	--	2.00	0.646	**51.4**
#100	0.150	2.5	--	2.52	0.646	**48.6**
#140	0.106	1.8	--	1.83	0.646	**46.5**
#200	0.075	3.1	--	3.05	0.646	**43.0**

Percent Passing vs Log of Particle Size



USCS Classification ASTM D 2487

Atterberg Classification: CL
Group Symbol: SC
USCS Classification: Clayey Sand With Gravel

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: KMS Date: 11/8/2018
Checked by: KP Date: 11/8/18
File name: 2679130__Grain Size with Hydrometer ASTM D6913 D7928_1.xlsm

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	10-17.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-11
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hydrometer and Flask Parameters

Hydrometer ID:	0805	Flask ID:	1185
Average Mass Offset (g/L):	9.87	Flask Volume (cm^3):	995.9
Hydrometer Bulb Volume (cm^3):	56.50	Flask Surface Area (cm^2):	28.45
Meniscus Correction (g/L):	1.00	Assumed Specific Gravity	2.65
H_b (cm):	24.5	Hydrometer Type:	152H
H_{cb} (cm):	6.8		
H_s (cm):	8.2	Percent Finer by Mass at 2 µm:	**12.6**

Hydrometer Data

Elapsed Time (minutes)	Hydrometer Reading (g/L)	Offset Reading (g/L)	Temperature (°C)	Effective Depth (cm)	Maximum Particle Diameter in Suspension (mm)	Percent Finer by Mass (%)
1	35.00	5.49	22.7	11.11	0.045	**38.3**
2	33.50	5.49	22.7	11.36	0.032	**36.4**
4	30.00	5.49	22.7	11.93	0.024	**31.8**
15	25.00	5.49	22.7	12.76	0.013	**25.3**
30	22.50	5.45	22.8	13.17	0.009	**22.1**
60	20.00	5.45	22.8	13.58	0.006	**18.9**
240	16.50	5.34	23.1	14.16	0.003	**14.5**
1440	14.50	5.53	22.6	14.48	0.001	**11.7**

NOTES:

Percent Minus #200
ASTM D1140

ADVANCED TERRA TESTING

Percent Minus #200
ASTM D 1140

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	42.5-43.5
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-16
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/31/18		
TECHNICIAN	AD		

Hygroscopic Moisture of Fines

Mass Wet Pan and Soil (g):	1261.71
Mass Dry Pan and Soil (g):	1250.69
Mass of Pan (g):	829.11
Moisture (%):	**2.6**

Sample Data

Total Wet Mass of Sample (g):	625.3
Total Dry Mass of Sample (g):	614.0
Split Fraction:	#4
Mass of Sub-Sample Fraction (g):	432.59

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
#4	4.75	184.7	--	184.7	1.00	**69.9**
#200	0.075	184.9	--	184.9	0.70	**39.3**

USCS Classification ASTM D 2487

Atterberg Classification: --
Group Symbol: --
USCS Classification: --

NOTES

Data entry by: SPH Date: 11/1/2018
Checked by: CH Date: 11/02/18
File name: 2679130__Percent Minus #200 ASTM D1140_1.xls

ADVANCED TERRA TESTING

Percent Minus #200
ASTM D 1140

CLIENT	RJH Consultants	BORING NO.	B-102A
JOB NO.	2679-130	DEPTH	29-30'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-14
PROJECT NO.	18115	DATE SAMPLED	07/24/18
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/30/18		
TECHNICIAN	KJT		

Hygroscopic Moisture

Mass Wet Pan and Soil (g): 1620.42
Mass Dry Pan and Soil (g): 1613.74
Mass of Pan (g): 1024.97
Moisture (%): **1.1**

Sample Data

Total Wet Mass of Sample (g): 595.5
Total Dry Mass of Sample (g): 588.8

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
#4	4.75	335.6	0.0	335.6	1.00	**43.0**
#200	0.075	1484.5	1025.0	459.6	1.00	**21.9**

USCS Classification ASTM D 2487

Atterberg Classification: --
Group Symbol: --
USCS Classification: --

NOTES

Data entry by: CAL Date: 11/5/2018
Checked by: SPH Date: 11-5-18
File name: 2679130__Percent Minus #200 ASTM D1140_2.xls

ADVANCED TERRA TESTING

Percent Minus #200
ASTM D 1140

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	15-16'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-8
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/29/18		
TECHNICIAN	CAL		

Hygroscopic Moisture

Mass Wet Pan and Soil (g): 598.84
Mass Dry Pan and Soil (g): 566.35
Mass of Pan (g): 266.20
Moisture (%): **10.8**

Sample Data

Total Wet Mass of Sample (g): 332.6
Total Dry Mass of Sample (g): 300.2

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
#4	4.75	138.1	--	138.1	1.00	**54.0**
#200	0.075	468.5	266.2	202.2	1.00	**32.6**

USCS Classification ASTM D 2487

Atterberg Classification: --
Group Symbol: --
USCS Classification: --

NOTES

Data entry by: SPH Date: 10/30/2018
Checked by: KR Date: 11/26/18
File name: 2679130__Percent Minus #200 ASTM D1140_0.xls

Standard Proctor Compaction
ASTM D698

ADVANCED TERRA TESTING

Laboratory Compaction Characteristics

ASTM D 698

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	15-25'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-6
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/22/18		
TECHNICIAN	WAR		

Laboratory Compaction Characteristics

Hygroscopic Moisture

Mass of Wet Pan and Soil (g):	321.59
Mass of Dry Pan and Soil (g):	316.49
Mass of Pan (g):	14.17
Moisture (%):	1.7

Rock Correction ASTM D 4718

Method:	B
Course Fraction (%):	12.4
Rock Correction Applied:	YES
Mass of Dry Aggregate (g):	3374.2
Mass of SSD Aggregate (g):	3494.0
Mass of Aggregate in Water (g):	2173.3
Rock Specific Gravity:	2.55
Zero Air Voids Specific Gravity:	2.8

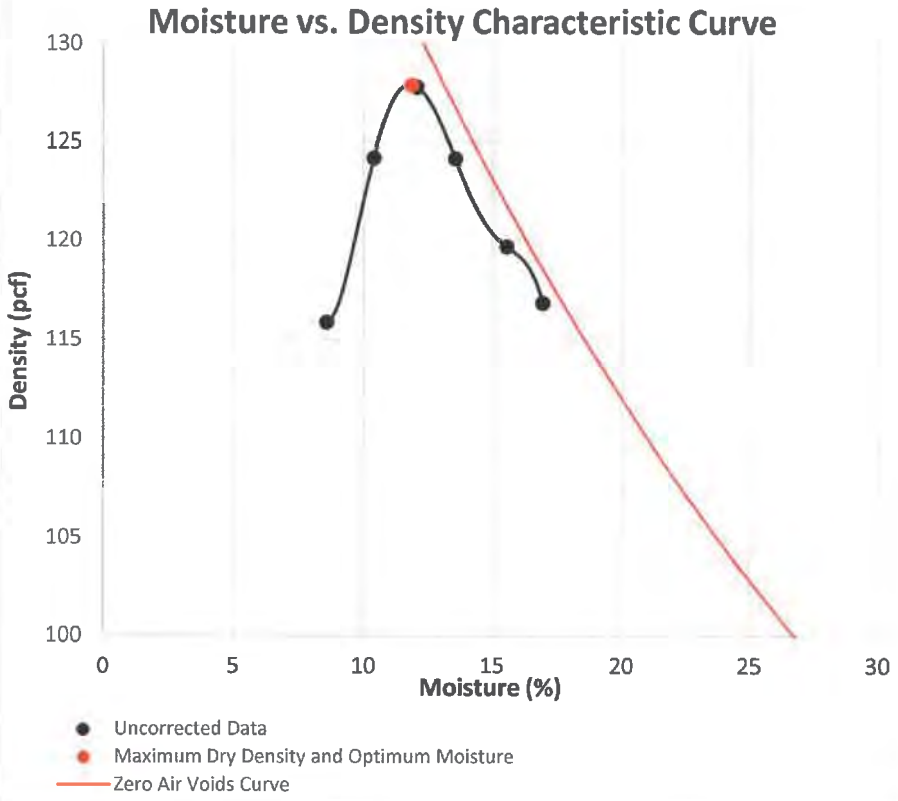
Optimum Dry Density and Moisture

Uncorrected

Dry Density (pcf):	**127.9**
Dry Density (kg/m³):	**2048**
Moisture (%):	**11.9**

Corrected

Dry Density (pcf):	**131.1**
Dry Density (kg/m³):	**2100**
Moisture (%):	**10.4**



Sample Number:	1	2	3	4	5	6
Mass of Wet Pan and Soil (g):	515.29	469.19	502.78	484.74	579.30	328.99
Mass of Dry Soil and Pan (g):	441.62	406.98	443.62	433.27	525.32	303.47
Mass of Pan (g);	6.79	6.74	6.78	6.62	6.69	6.62
Moisture (%):	16.9	15.5	13.5	12.1	10.4	8.6
Mass of Wet Soil and Mold (g):	6650.0	6675.1	6715.4	6748.9	6657.1	6486.6
Mass of Mold (g):	4584.1	4584.1	4584.1	4584.1	4584.1	4584.1
Wet Density (pcf):	125.8	137.1	143.2	141.0	138.3	136.6
Dry Density (pcf):	115.9	124.2	127.8	124.1	119.7	116.8
Wet Density (kg/m³):	2016	2196	2293	2258	2215	2189
Dry Density (kg/m³):	1856	1989	2047	1989	1917	1872

Data entry by: SPH Date: 10/25/2018
Checked by: KMS Date: 11/8/18
File name: 2679130__compaction ASTM D698 D1557_1.xls

ADVANCED TERRA TESTING

Laboratory Compaction Characteristics

ASTM D 698

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	51-65'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-20
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/25/18		
TECHNICIAN	BDF		

Laboratory Compaction Characteristics

Hygroscopic Moisture

Mass of Wet Pan and Soil (g):	293.70
Mass of Dry Pan and Soil (g):	288.60
Mass of Pan (g):	6.61
Moisture (%):	1.8

Rock Correction ASTM D 4718

Method:	B
Course Fraction (%):	11.9
Rock Correction Applied:	YES
Mass of Dry Aggregate (g):	2131.3
Mass of SSD Aggregate (g):	2205.6
Mass of Aggregate in Water (g):	1377.7
Rock Specific Gravity:	2.57
Zero Air Voids Specific Gravity:	2.76

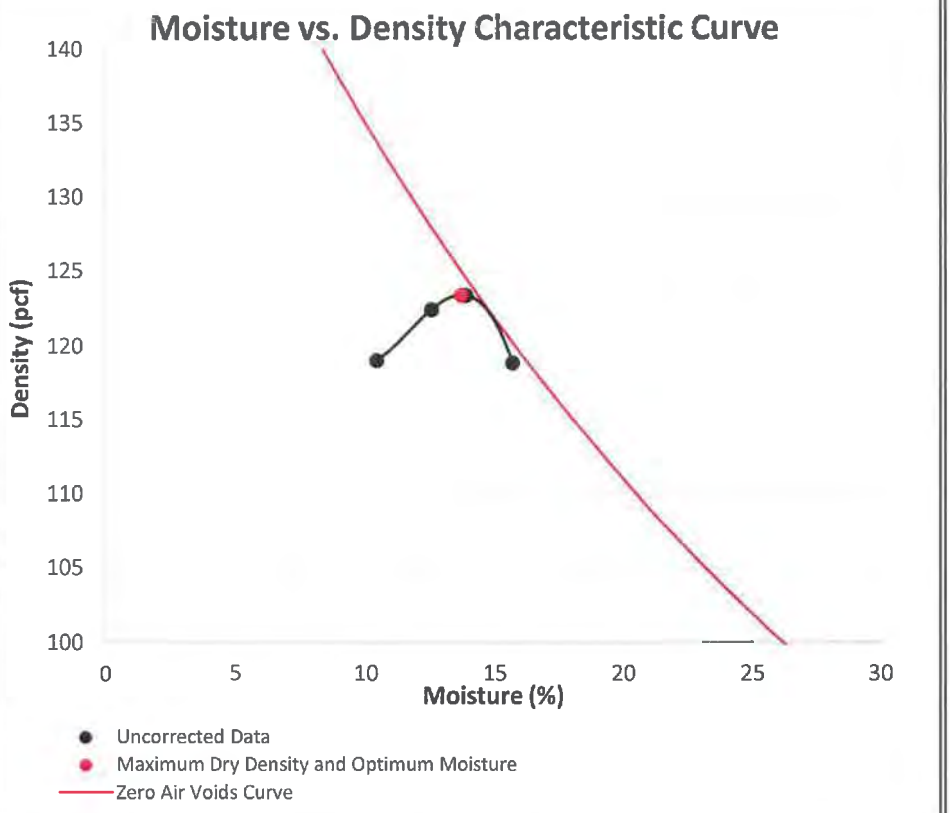
Optimum Dry Density and Moisture

Uncorrected

Dry Density (pcf):	**123.4**
Dry Density (kg/m³):	**1977**
Moisture (%):	**13.7**

Corrected

Dry Density (pcf):	**126.9**
Dry Density (kg/m³):	**2033**
Moisture (%):	**12.1**



Sample Number:	1	2	3	4
Mass of Wet Pan and Soil (g):	310.62	360.16	431.81	288.79
Mass of Dry Soil and Pan (g):	276.78	317.10	374.17	262.13
Mass of Pan (g);	6.92	6.62	6.55	6.56
Moisture (%):	12.5	13.9	15.7	10.4
Mass of Wet Soil and Mold (g):	6667.1	6708.3	6662.5	6570.8
Mass of Mold (g):	4584.0	4584.0	4584.0	4584.0
Wet Density (pcf):	131.4	137.8	140.5	137.5
Dry Density (pcf):	119.0	122.4	123.4	118.8
Wet Density (kg/m³):	2105	2207	2251	2202
Dry Density (kg/m³):	1906	1961	1976	1904

Data entry by: CAL Date: 10/29/2018

Checked by: SPH Date: 10-30-18

File name: 2679130__compaction ASTM D698 D1557_2.xls

ADVANCED TERRA TESTING

Laboratory Compaction Characteristics

ASTM D 698

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	10-17.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-11
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/22/18		
TECHNICIAN	WAR		

Laboratory Compaction Characteristics

Hygroscopic Moisture

Mass of Wet Pan and Soil (g):	396.44
Mass of Dry Pan and Soil (g):	389.75
Mass of Pan (g):	6.56
Moisture (%):	1.7

Rock Correction ASTM D 4718

Method:	B
Course Fraction (%):	14.1
Rock Correction Applied:	YES
Mass of Dry Aggregate (g):	3506.1
Mass of SSD Aggregate (g):	3587.6
Mass of Aggregate in Water (g):	2237.2
Rock Specific Gravity:	2.60
Zero Air Voids Specific Gravity:	2.75

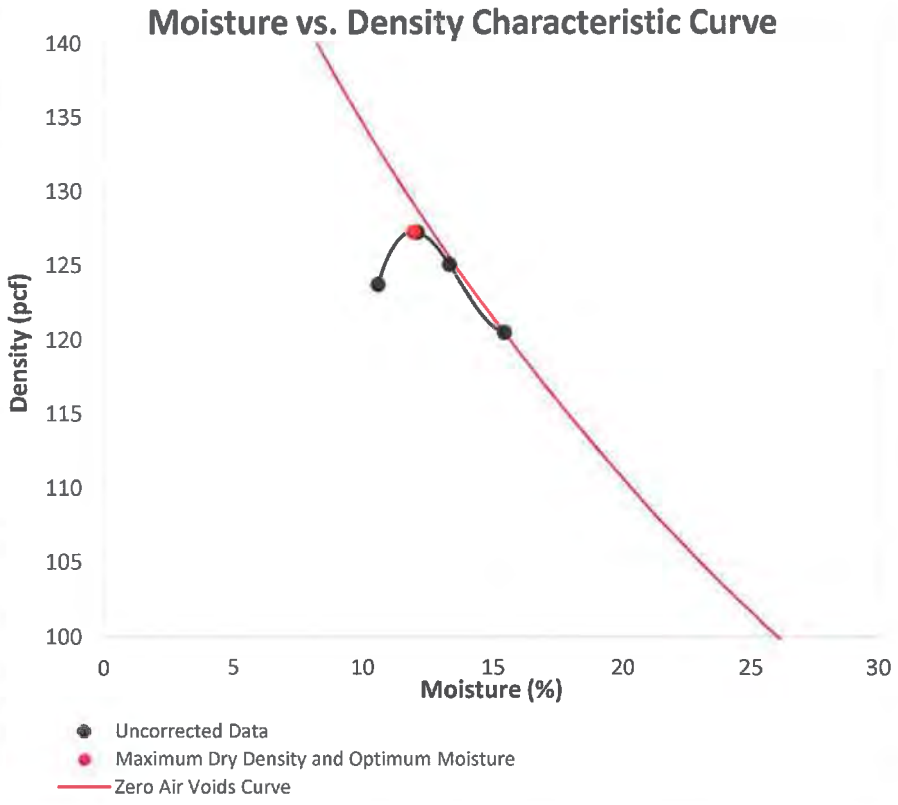
Optimum Dry Density and Moisture

Uncorrected

Dry Density (pcf):	**127.3**
Dry Density (kg/m³):	**2039**
Moisture (%):	**11.9**

Corrected

Dry Density (pcf):	**131.3**
Dry Density (kg/m³):	**2103**
Moisture (%):	**10.3**



Sample Number:	1	2	3	4
Mass of Wet Pan and Soil (g):	397.72	461.20	491.64	419.00
Mass of Dry Soil and Pan (g):	345.67	407.78	439.33	379.57
Mass of Pan (g);	8.51	6.74	6.78	7.11
Moisture (%):	15.4	13.3	12.1	10.6
Mass of Wet Soil and Mold (g):	6687.3	6727.4	6741.1	6653.1
Mass of Mold (g):	4584.1	4584.1	4584.1	4584.1
Wet Density (pcf):	136.8	142.7	141.8	139.1
Dry Density (pcf):	123.7	127.3	125.1	120.5
Wet Density (kg/m³):	2192	2285	2271	2228
Dry Density (kg/m³):	1982	2039	2004	1930

Data entry by: SPH Date: 10/25/2018

Checked by: CM Date: 10/26/18

File name: 2679130__compaction ASTM D698 D1557_0.xls

One-Dimensional Consolidation
ASTM D2435

ADVANCED TERRA TESTING

One-Dimensional Consolidation

ASTM D 2435

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	27.5-29.9'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	u-14
PROJECT NO.	18115	DATE SAMPLED	07/23/18
LOCATION	--	SAMPLED BY	--
DATE TESTED	10/22/18	DESCRIPTION	--
TECHNICIAN	AD		

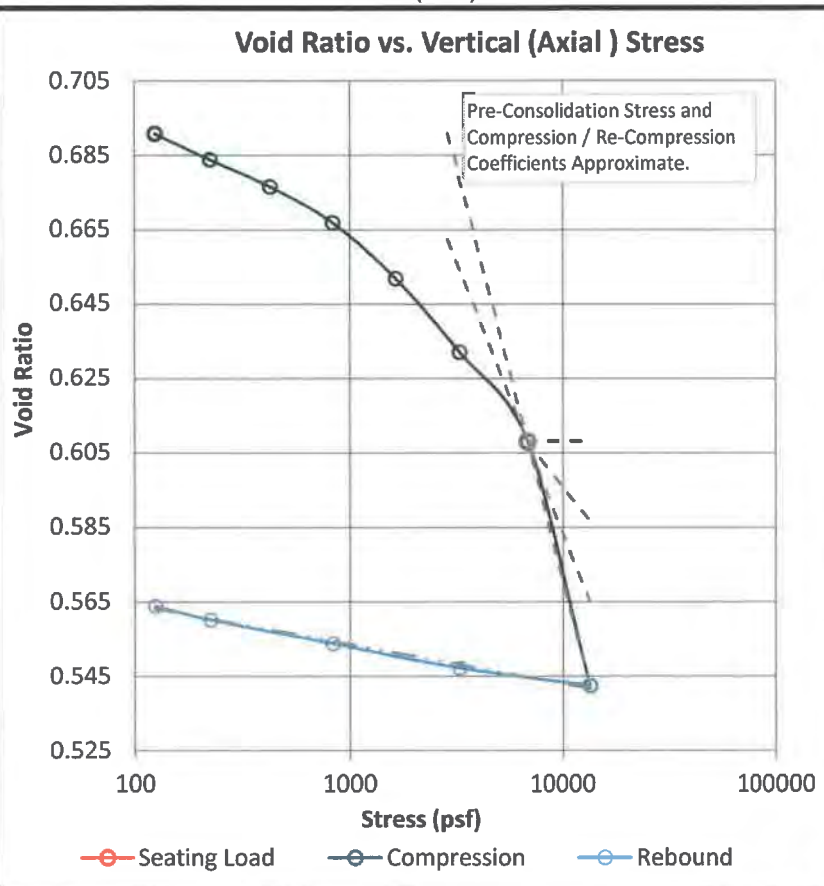
Sample Conditions

Before Test Mass of Wet Soil and Ring (g):	189.07	Initial Wet Density (pcf):	123.2
After Test Mass of Wet Soil and Ring (g):	185.90	Initial Dry Density (pcf):	97.8
Mass of Dry Soil, Ring, and Pan (g):	161.80	Initial Wet Density (kg/m³):	1973
Diameter (in):	2.41	Initial Dry Density (kg/m³):	1567
Initial Height (in):	1.00	Initial Moisture (%):	25.9
Mass of Ring (g):	41.57	Final Wet Density (pcf):	130.3
Mass of Pan (g):	3.08	Final Dry Density (pcf):	105.8
Assumed Specific Gravity:	2.65	Final Wet Density (kg/m³):	2088
Initial Saturation (%):	99.6	Final Dry Density (kg/m³):	1694
Final Saturation (%):	100.0	Final Moisture (%):	23.2

Swell / Collapse Data

Coefficient of Compression:	0.224	Pre-Consolidation Stress (psf):	6780
Coefficient of Re-Compression:	0.011	Pre-Consolidation Stress (kPa):	325

Load (psf)	Void Ratio	Deformation (in)	Strain (%)
125	0.691	0.0000	0.00
Inundation	0.691	0.0001	0.01
225	0.684	0.0042	0.42
428	0.677	0.0085	0.85
832	0.667	0.0142	1.42
1636	0.652	0.0231	2.31
3248	0.632	0.0348	3.48
6780	0.608	0.0490	4.90
13314	0.543	0.0878	8.78
Rebound			
13314	0.543	0.0878	8.78
3248	0.547	0.0850	8.50
832	0.554	0.0811	8.11
225	0.560	0.0773	7.73
125	0.564	0.0751	7.51



Data entry by: SPH Date: 11/5/2018
Checked by: Date: 11/7/18
File name: 2679130__Consol ASTM D2435_0.xlsm

ADVANCED TERRA TESTING

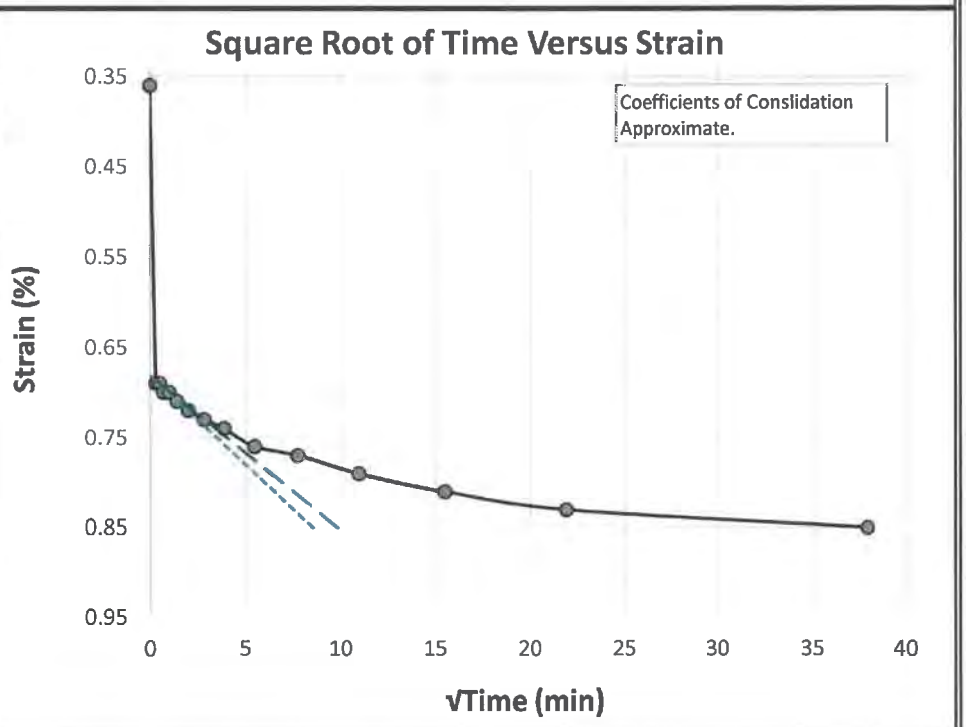
One-Dimensional Consolidation

ASTM D 2435

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	27.5-29.9'
PROJECT	Hogchute Dam Safety Evaluatior	SAMPLE NO.	u-14
PROJECT NO.	18115	DATE SAMPLED	07/23/18
LOCATION	--	SAMPLED BY	--
DATE TESTED	10/22/18	DESCRIPTION	--
TECHNICIAN	AD		

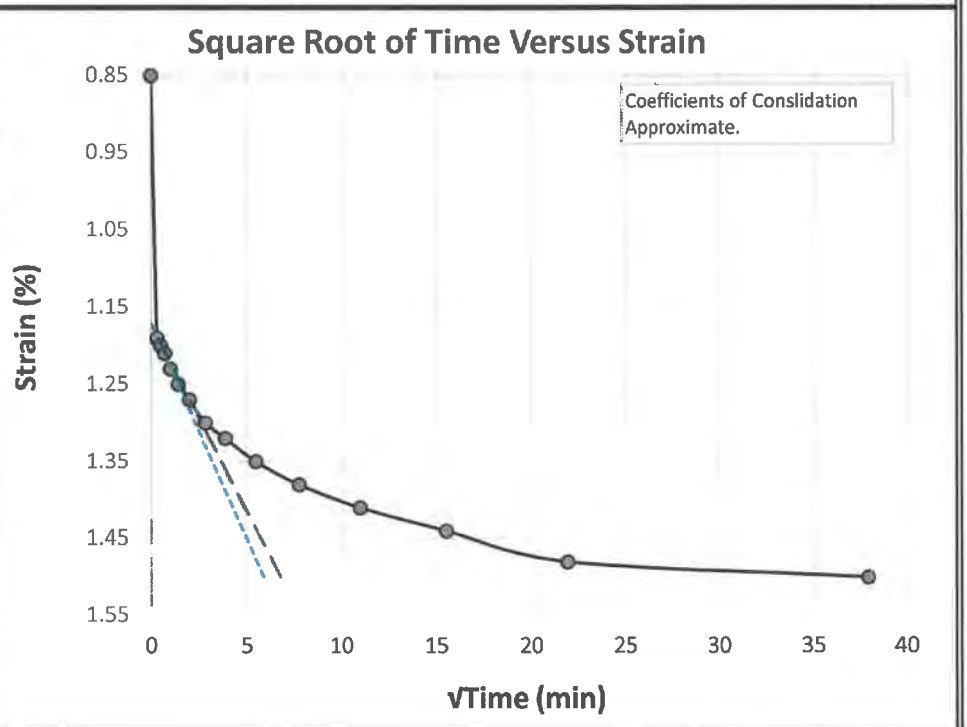
Coefficient of Consolidation (cm^2/s)	T90 (min)	Load (psf)
0.003	8.321	428

Ellapsed Time (min)	Deformation (in)	Strain (%)
0	-0.0045	0.36
0.1	-0.0078	0.69
0.25	-0.0078	0.69
0.5	-0.0079	0.70
1	-0.0079	0.70
2	-0.0080	0.71
4	-0.0081	0.72
8	-0.0082	0.73
15	-0.0083	0.74
30	-0.0085	0.76
60	-0.0086	0.77
120	-0.0088	0.79
240	-0.0090	0.81
480	-0.0092	0.83
1439	-0.0094	0.85



Coefficient of Consolidation (cm^2/s)	T90 (min)	Load (psf)
0.005	4.280	832

Ellapsed Time (min)	Deformation (in)	Strain (%)
0	-0.0094	0.85
0.1	-0.0128	1.19
0.25	-0.0129	1.20
0.5	-0.0130	1.21
1	-0.0132	1.23
2	-0.0134	1.25
4	-0.0136	1.27
8	-0.0139	1.30
15	-0.0141	1.32
30	-0.0144	1.35
60	-0.0147	1.38
120	-0.0150	1.41
240	-0.0153	1.44
480	-0.0157	1.48
1438	-0.0159	1.50



File name: 2679130__Consol ASTM D2435_1.xlsm

ADVANCED TERRA TESTING

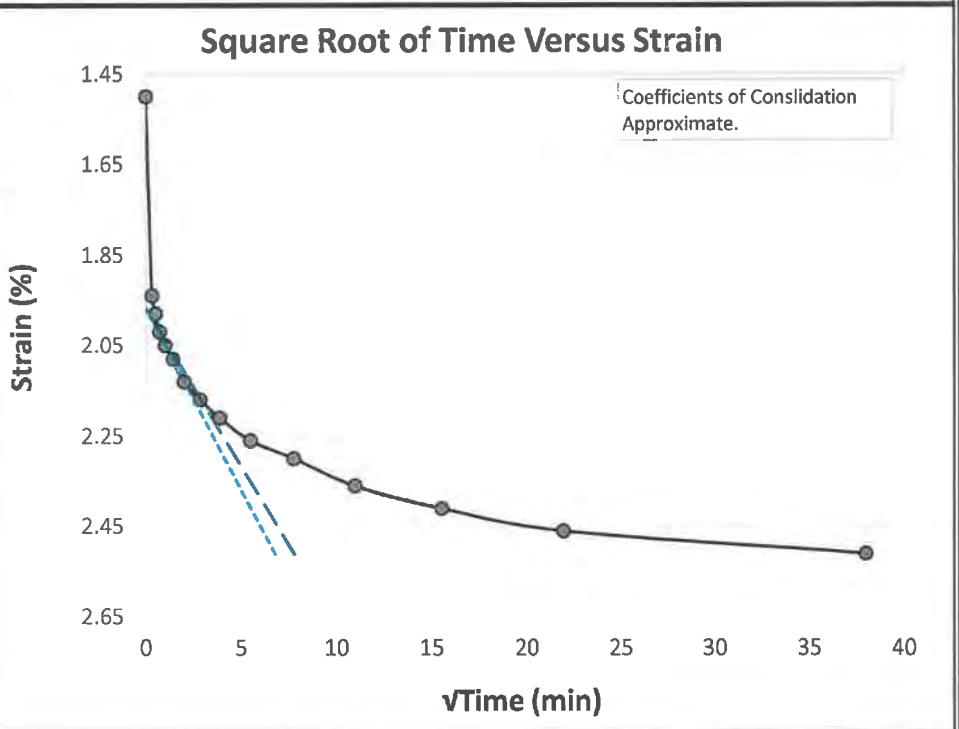
One-Dimensional Consolidation

ASTM D 2435

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	27.5-29.9'
PROJECT	Hogchute Dam Safety Evaluatior	SAMPLE NO.	u-14
PROJECT NO.	18115	DATE SAMPLED	07/23/18
LOCATION	--	SAMPLED BY	--
DATE TESTED	10/22/18	DESCRIPTION	--
TECHNICIAN	AD		

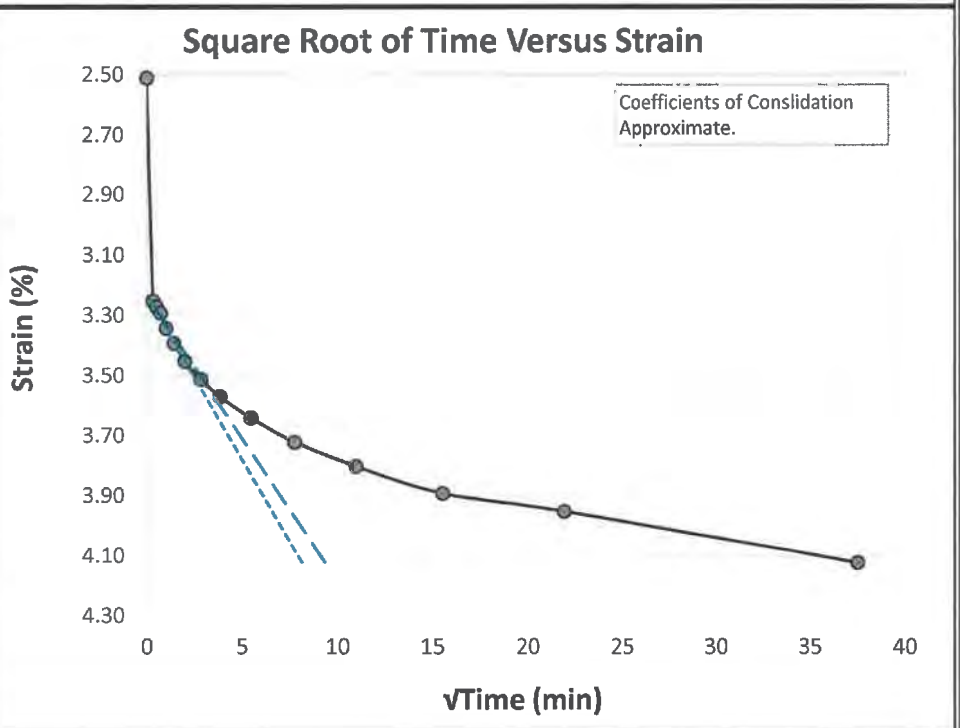
Coefficient of Consolidation (cm^2/s)	T90 (min)	Load (psf)
0.003	8.720	1636

Ellapsed Time (min)	Deformation (in)	Strain (%)
0	-0.0159	1.50
0.1	-0.0203	1.94
0.25	-0.0207	1.98
0.5	-0.0211	2.02
1	-0.0214	2.05
2	-0.0217	2.08
4	-0.0222	2.13
8	-0.0226	2.17
15	-0.0230	2.21
30	-0.0235	2.26
60	-0.0239	2.30
120	-0.0245	2.36
240	-0.0250	2.41
480	-0.0255	2.46
1442	-0.0260	2.51



Coefficient of Consolidation (cm^2/s)	T90 (min)	Load (psf)
0.002	9.319	3248

Ellapsed Time (min)	Deformation (in)	Strain (%)
0	-0.0260	2.51
0.1	-0.0334	3.25
0.25	-0.0336	3.27
0.5	-0.0338	3.29
1	-0.0343	3.34
2	-0.0348	3.39
4	-0.0354	3.45
8	-0.0360	3.51
15	-0.0366	3.57
30	-0.0373	3.64
60	-0.0381	3.72
120	-0.0389	3.80
240	-0.0398	3.89
480	-0.0404	3.95
1406	-0.0421	4.12



File name: 2679130__Consol ASTM D2435_0.xlsm

Consolidated Undrained
Triaxial Compression
ASTM D4767

Consolidated Undrained Triaxial Compression Test for Cohesive Soils
ASTM D 4767

Client	RJH Consultants	Boring Number:	B-103
Job Number	2679-130	Depth:	27.5-29.9'
Project	Hogchute Dam Safety Evaluation	Sample Number:	u-14 Pts A, B & C
Location	--	Sampled Date:	7/23/2018
Project Number	18115	Sampled By:	--

Tested By: SPH/CAL

TEST TYPE TX/CUPP

σ_3 Confining Stresses (psf)	
SAMPLE A	10000
SAMPLE B	6000
SAMPLE C	3000

Peak Points	p' (psf)	q (psf)
SAMPLE A	8852	5102
SAMPLE B	4720	2810
SAMPLE C	3584	2096

Stress Condition at Maximum Deviator Stress (PSF)				
	σ3	σ1	σ'3	σ'1
SAMPLE A	10000	20204	3750	13954
SAMPLE B	6000	11620	1910	7530
SAMPLE C	3000	7192	1488	5680

SAMPLE A DATA

σ_3' (psf)	σ_1' (psf)	Deviator Stress $(\sigma_1-\sigma_3)$ (psf)	$p' = (\sigma_1'+\sigma_3')/2$ (psf)	$q = (\sigma_1-\sigma_3)/2$ (psf)
10000	10000	0	10000	0
9870	10502	632	10186	316
9640	10879	1239	10260	620
9078	11602	2524	10340	1262
8387	12029	3642	10208	1821
7710	12259	4549	9985	2275
7062	12282	5220	9672	2610
6486	12213	5727	9350	2864
5982	12075	6093	9029	3047
5550	11963	6413	8757	3207
5176	11790	6614	8483	3307
4874	11668	6794	8271	3397
3894	11161	7267	7528	3634
3448	10977	7529	7213	3765
3218	10957	7739	7088	3870
3102	11053	7951	7078	3976
3045	11363	8318	7204	4159
3074	11728	8654	7401	4327
3146	12171	9025	7659	4513
3232	12551	9319	7892	4660
3333	12897	9564	8115	4782
3419	13135	9716	8277	4858
3506	13367	9861	8437	4931
3578	13536	9958	8557	4979
3621	13692	10071	8657	5036
3693	13814	10121	8754	5061
3750	13954	10204	8852	5102
3808	14010	10202	8909	5101

SAMPLE B DATA

σ_3' (psf)	σ_1' (psf)	Deviator Stress $(\sigma_1-\sigma_3)$ (psf)	$p' = (\sigma_1'+\sigma_3')/2$ (psf)	$q = (\sigma_1-\sigma_3)/2$ (psf)
6000	6000	0	6000	0
5942	6119	177	6031	89
5784	6259	475	6022	238
5237	6932	1695	6085	848
4618	7232	2614	5925	1307
4070	7260	3190	5665	1595
3638	7171	3533	5405	1767
3278	7061	3783	5170	1892
3005	6969	3964	4987	1982
2789	6864	4075	4827	2038
2616	6802	4186	4709	2093
2098	6583	4485	4341	2243
1882	6525	4643	4204	2322
1795	6574	4779	4185	2390
1752	6664	4912	4208	2456
1694	6802	5108	4248	2554
1723	7019	5296	4371	2648
1795	7231	5436	4513	2718
1838	7389	5551	4614	2776
1910	7530	5620	4720	2810
1954	7554	5600	4754	2800
1982	7458	5476	4720	2738
1968	7238	5270	4603	2635
1910	7018	5108	4464	2554
1882	6851	4969	4367	2485
1853	6684	4831	4269	2416
1781	6471	4690	4126	2345

SAMPLE C DATA

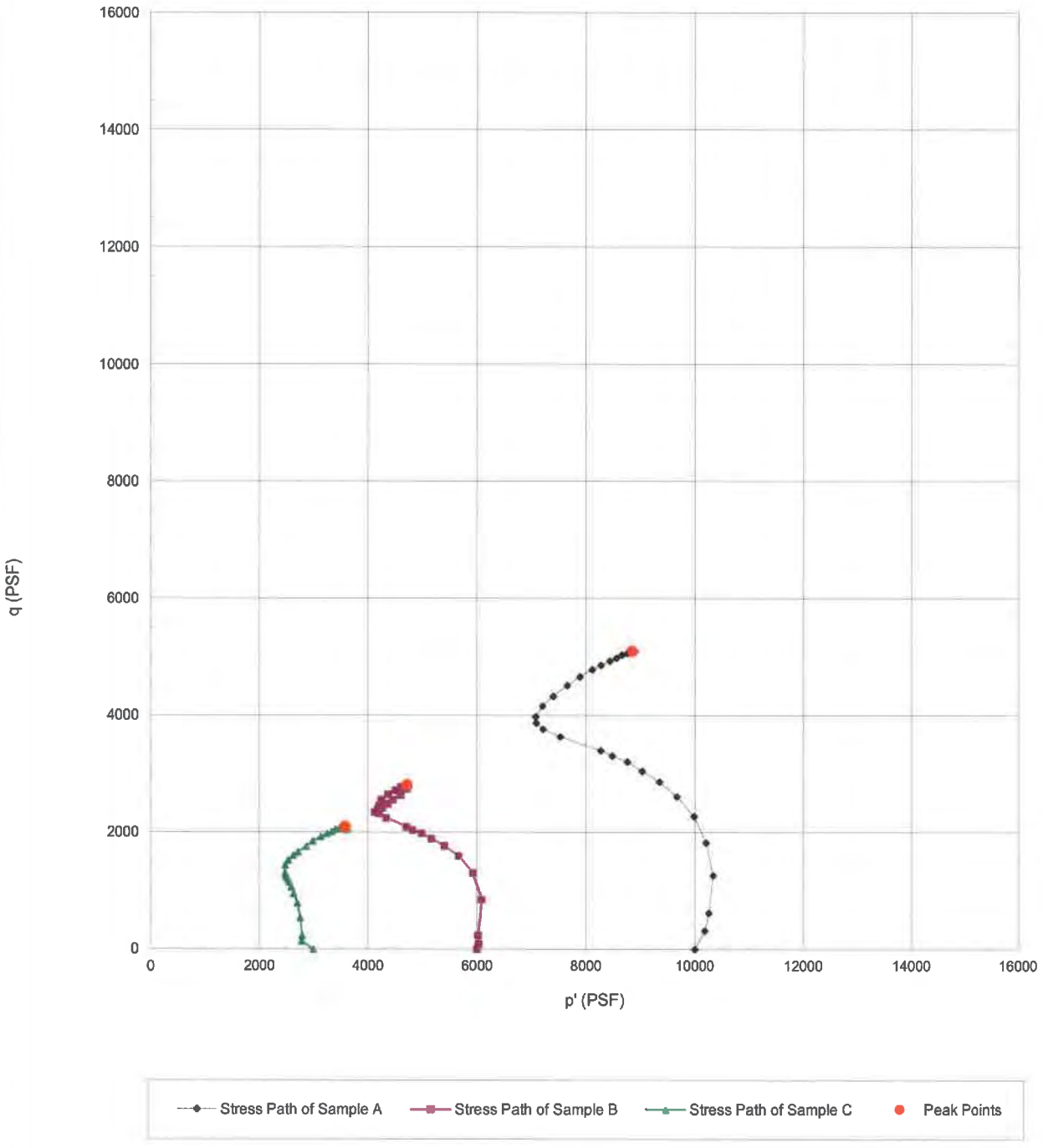
σ_3' (psf)	σ_1' (psf)	Deviator Stress $(\sigma_1-\sigma_3)$ (psf)	$p' = (\sigma_1'+\sigma_3')/2$ (psf)	$q = (\sigma_1-\sigma_3)/2$ (psf)
3000	3000	0	3000	0
2654	2931	277	2793	139
2568	3028	460	2798	230
2222	3302	1080	2762	540
1920	3503	1583	2712	792
1690	3593	1903	2642	952
1531	3662	2131	2597	1066
1416	3705	2289	2561	1145
1330	3730	2400	2530	1200
1258	3747	2489	2503	1245
1214	3769	2555	2492	1278
1171	3792	2621	2482	1311
1042	3921	2879	2482	1440
1013	4058	3045	2536	1523
1027	4237	3210	2632	1605
1056	4383	3327	2720	1664
1114	4630	3516	2872	1758
1142	4841	3699	2992	1850
1214	5068	3854	3141	1927
1272	5235	3963	3254	1982
1330	5354	4024	3342	2012
1358	5465	4107	3412	2054
1416	5560	4144	3488	2072
1459	5618	4159	3539	2080
1488	5680	4192	3584	2096
1531	5715	4184	3623	2092
1546	5701	4155	3624	2078
1560	5647	4087	3604	2044

Data entry by: SPH
Date: 10/24/18
FileName: 2679_130_PQPlots-ASTM-D4767-withmetric-R2_0.xls

Data checked by: CJK
Date: 10/25/18

Effective Stress Path Analysis - p' q Plots

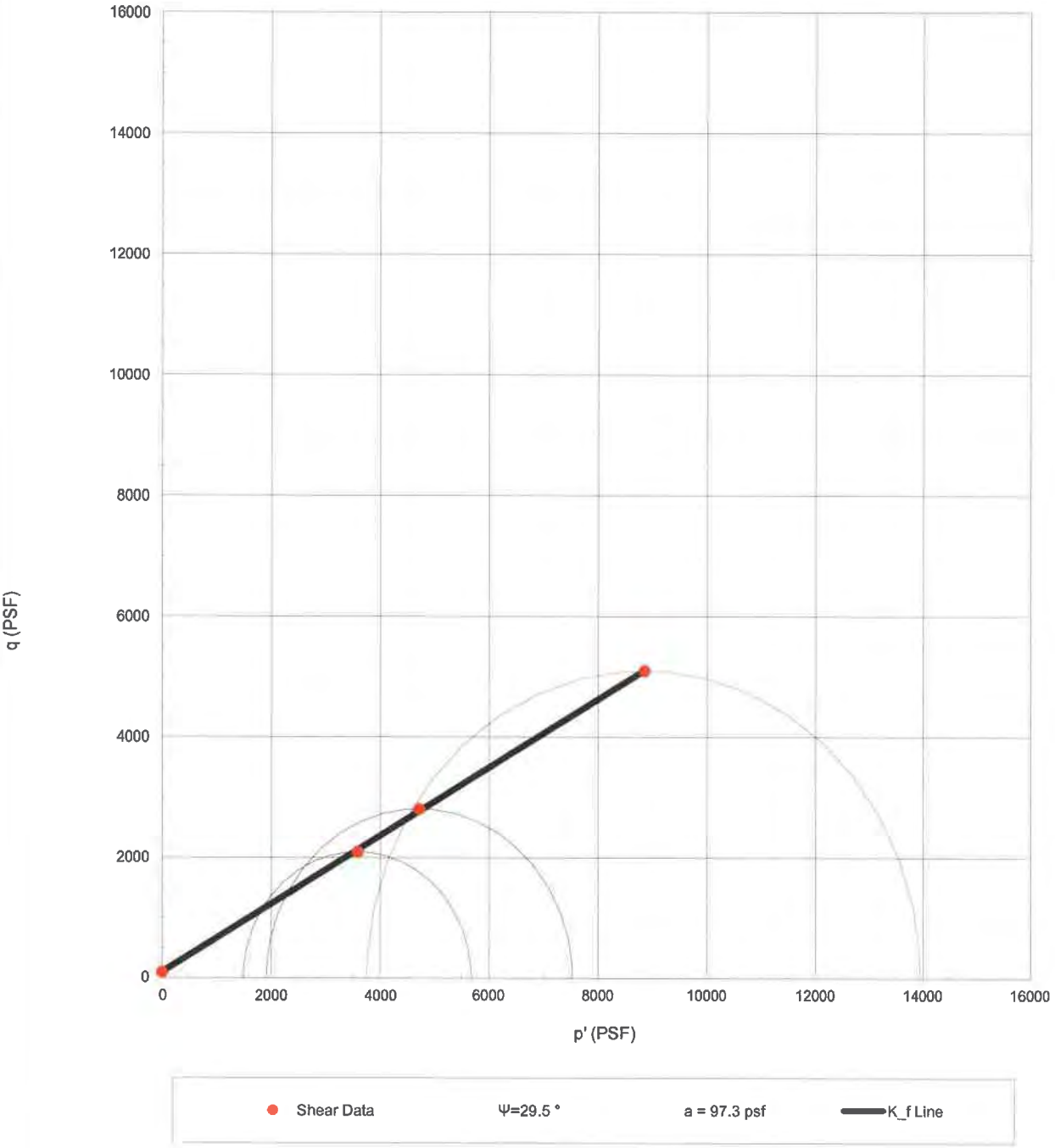
--,B-103,u-14 Pts A, B & C,27.5-29.9'



NOTE 1: The peak points shown in the plot represent maximum values of q [(σ1-σ3)/2].

Effective Stress Path Analysis - p'-q Regression Plot at Maximum q

--,B-103,u-14 Pts A, B & C,27.5-29.9'



NOTE 2: The line presented in the graph is the K_f line taken at Peak q values defined by the equation $q=a+p' \tan(\Psi)$ where a = the intercept on the q-axis in stress units and Ψ = the angle of the K_f line with respect to the horizontal in degrees.

NOTE 3: The K_f is NOT the Mohr-Coulomb failure envelope defined by the equation $\tau=c+\sigma \tan(\Phi)$. The equations $\sin(\Phi)=\tan(\Psi)$ and $c=a/\cos(\Phi)$ may be used to approximate values for Φ and c at the effective stress condition described in NOTE 1.

ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/19/2018
BORING NO.	B-103	CELL NUMBER	24S
DEPTH	27.5-29.9' (29.15-29.65)	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. A	CONF. PRES. (psf)	10000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

MOISTURE/DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture (g)	1266.40	1226.21
Wt. Wet Soil & Pan (g)	1280.85	1240.66
Wt. Dry Soil & Pan (g)	1017.19	1017.19
Wt. Lost Moisture (g)	263.66	223.47
Wt. of Pan Only (g)	14.45	14.45
Wt. of Dry Soil (g)	1002.74	1002.74
Moisture Content %	26.3	22.3
Wet Density (pcf)	127.3	133.3
Dry Density (pcf)	100.8	109.0
Init. Diameter (in)	2.872	
Init. Area (sq in)	6.478	
Init. Height (in)	5.852	
Vol. Bef. Consol. (cu ft)	0.02194	
Vol. After Consol. (cu ft)	0.02028	

Notes & Comments: Did not use slotted filter paper drains due to permeability test prior to shear.

Data entry by: SPH Date: 10/23/2018 Technician CAL
Data checked by: CAL Date: 10/23/18
FileName: 2679_130_TX_CUpp_ASTMD_4767_R0_1.xls

ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/19/2018
BORING NO.	B-103	CELL NUMBER	24S
DEPTH	27.5-29.9' (29.15-29.65)	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. A	CONF. PRES. (psf)	10000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

SATURATION DATA

Cell Pressure (psi)	Back Pressure (psi)	Burette Reading (cc)		Pore Pressure (psi)		Change	B
		Close	Open	Close	Open		
40.0	38.0	3.2	19.3				
50.0		20.0	20.3	38.7	48.4	9.7	0.97

CONSOLIDATION DATA

Elapsed Time (min)	SQRT TIME (min)	Burette Reading (cc)	Volume Deflection (cc)
0.00	0.0	0.80	0.00
0.25	0.5	34.60	-33.80
0.5	0.7	36.70	-35.90
1	1.0	37.80	-37.00
2	1.4	38.60	-37.80
4	2.0	39.30	-38.50
9	3.0	40.10	-39.30
16	4.0	40.80	-40.00
30	5.5	41.50	-40.70
88	9.4	42.70	-41.90
120	11.0	43.10	-42.30
240	15.5	43.90	-43.10
360	19.0	44.30	-43.50
1450	38.1	45.90	-45.10

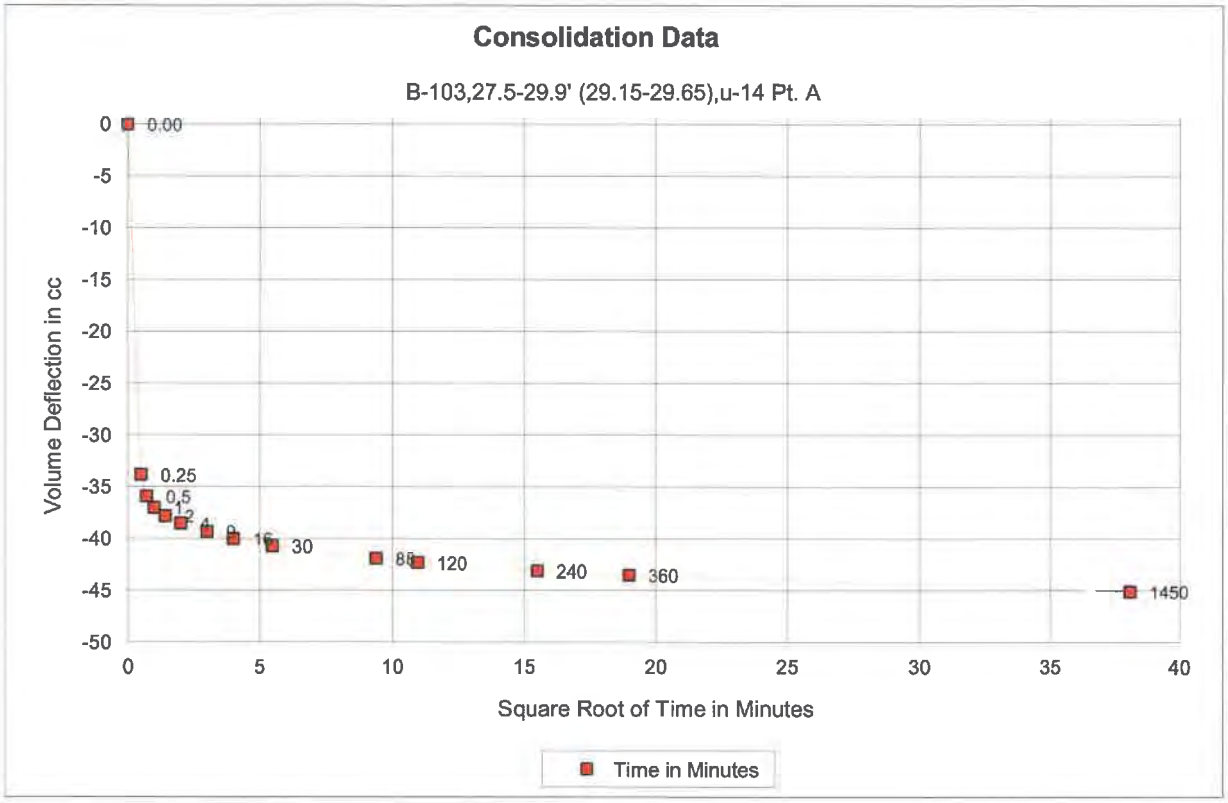
Initial Height (in)	5.852	Initial Vol. (cc)	621.358
Height Change (in)	0.151	Vol. Change (cc)	62.800
Ht. After Cons. (in)	5.701	Cell Exp. (cc)	15.703
Initial Area (sq in)	6.478	Net Change (cc)	47.097
Area After Cons. (sq in)	6.146	Cons. Vol. (cc)	574.261

ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D 4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/19/2018
BORING NO.	B-103	CELL NUMBER	24S
DEPTH	27.5-29.9' (29.15-29.65)	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. A	CONF. PRES. (psf)	10000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		
Init. Ht. (in)	5.852	Init. Area (sq in)	6.478
Consol. Ht. (in)	5.701	Consol. Area (sq in)	6.146
Back Pres. (psi)	38.3	Strain Rate (in/min)	0.0027

Axial Load (lbs.)	Axial Load (psf)	Delta Ht. (in)	Axial Strain (%)	Area Final (sq in)	Dev. Stress (psf)	Pore Pres. (psi)	Delta Pres. (psf)	Sigma 3' (psf)	Sigma 1' (psf)	Prin. Stress Ratio
0.0	0	0.000	0.00	6.146	0	38.3	0	10000	10000	1.00
27	633	0.005	0.09	6.151	632	39.2	130	9870	10502	1.06
53	1242	0.011	0.19	6.158	1239	40.8	360	9640	10879	1.13
108	2531	0.015	0.26	6.162	2524	44.7	922	9078	11602	1.28
156	3655	0.021	0.37	6.169	3642	49.5	1613	8387	12029	1.43
195	4569	0.026	0.46	6.174	4548	54.2	2290	7710	12259	1.59
224	5248	0.031	0.54	6.179	5220	58.7	2938	7062	12282	1.74
246	5764	0.037	0.65	6.186	5727	62.7	3514	6486	12213	1.88
262	6139	0.043	0.75	6.193	6093	66.2	4018	5982	12075	2.02
276	6467	0.048	0.84	6.198	6412	69.2	4450	5550	11963	2.16
285	6678	0.054	0.95	6.205	6614	71.8	4824	5176	11790	2.28
293	6865	0.059	1.03	6.210	6794	73.9	5126	4874	11668	2.39
315	7381	0.088	1.54	6.242	7267	80.7	6106	3894	11161	2.87
328	7685	0.116	2.03	6.273	7529	83.8	6552	3448	10977	3.18
339	7943	0.146	2.56	6.307	7740	85.4	6782	3218	10957	3.41
350	8201	0.174	3.05	6.339	7950	86.2	6898	3102	11053	3.56
370	8669	0.231	4.05	6.405	8318	86.6	6955	3045	11363	3.73
389	9114	0.288	5.05	6.473	8654	86.4	6926	3074	11728	3.82
410	9607	0.345	6.05	6.542	9025	85.9	6854	3146	12171	3.87
428	10028	0.403	7.07	6.613	9319	85.3	6768	3232	12551	3.88
444	10403	0.460	8.07	6.685	9564	84.6	6667	3333	12897	3.87
456	10684	0.517	9.07	6.759	9715	84.0	6581	3419	13135	3.84
468	10966	0.574	10.07	6.834	9861	83.4	6494	3506	13367	3.81
478	11200	0.632	11.09	6.912	9958	82.9	6422	3578	13536	3.78
489	11458	0.690	12.10	6.992	10071	82.6	6379	3621	13692	3.78
497	11645	0.746	13.09	7.071	10121	82.1	6307	3693	13814	3.74
507	11879	0.804	14.10	7.155	10204	81.7	6250	3750	13954	3.72
513	12020	0.862	15.12	7.241	10202	81.3	6192	3808	14010	3.68


Consolidation Data
B-103,27.5-29.9' (29.15-29.65),u-14 Pt. A
Volume Deflection in cc
Square Root of Time in Minutes
0.00
0.25
0.5
1
2
4
8
15
30
60
120
240
360
1450
Time in Minutes

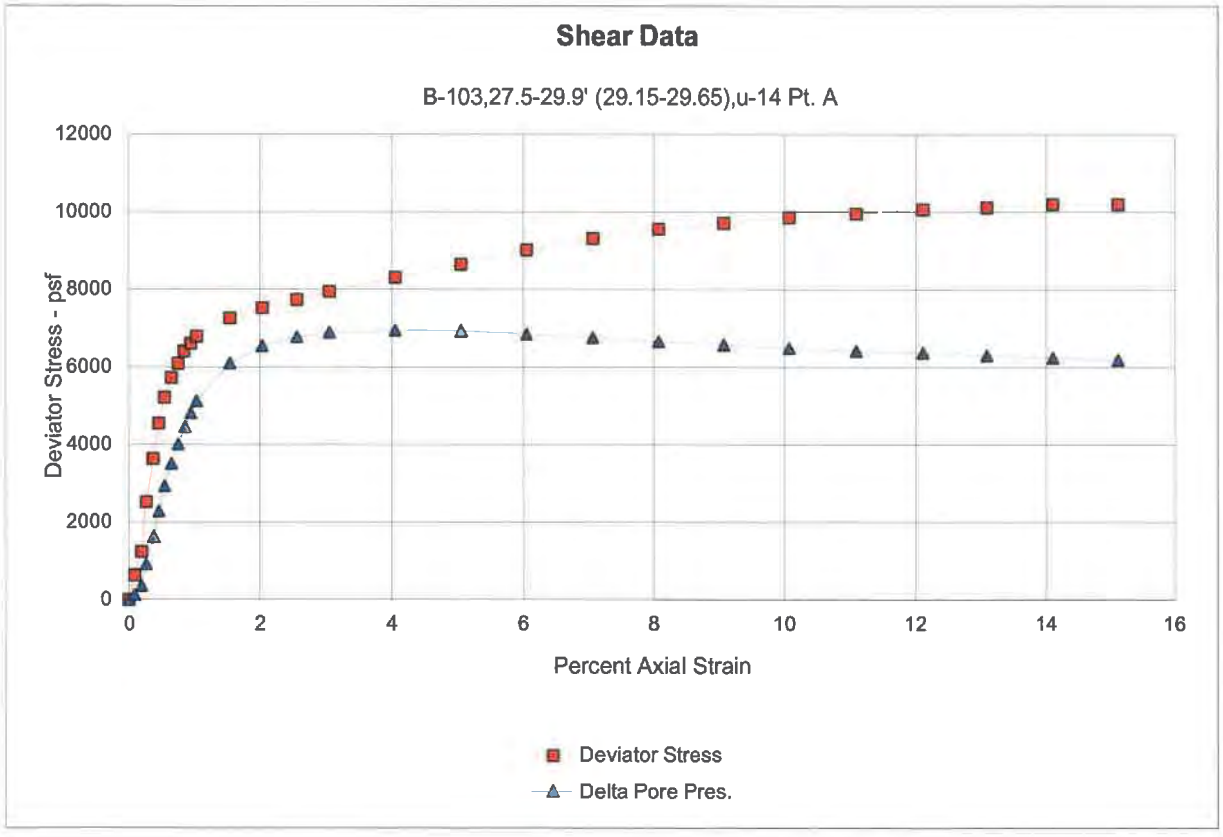
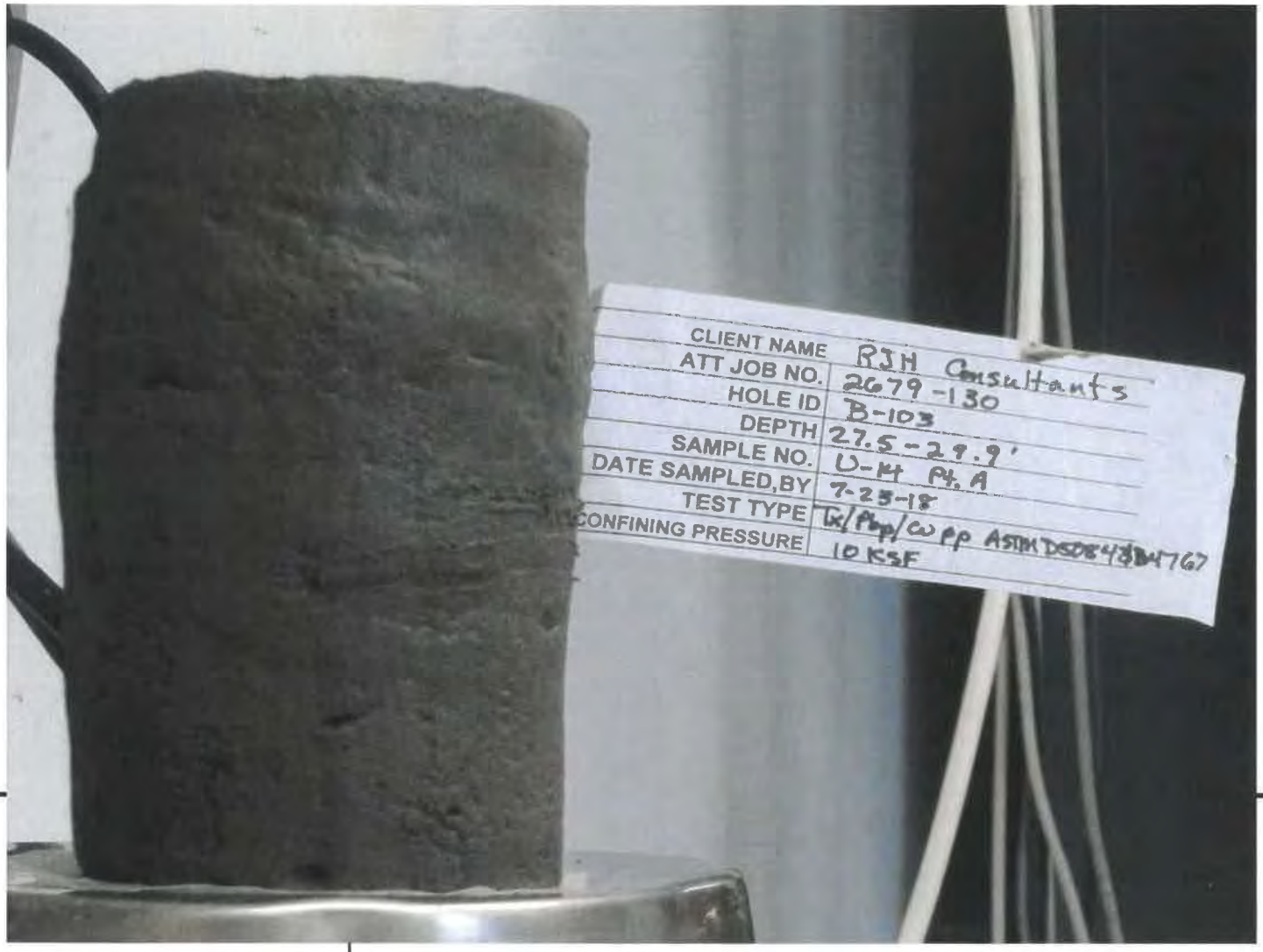

Shear Data
B-103,27.5-29.9' (29.15-29.65),u-14 Pt. A
Deviator Stress - psf
Percent Axial Strain
Deviator Stress
Delta Pore Pres.

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants
JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation
PROJECT NO.	18115
LOCATION	--



File name: 2679130__Image_18_10_22_06_53_56

ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/22/2018
BORING NO.	B-103	CELL NUMBER	18S
DEPTH	27.5-29.9' (28.65-29.15')	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. B	CONF. PRES. (psf)	6000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

MOISTURE/DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture (g)	1281.53	1261.98
Wt. Wet Soil & Pan (g)	1295.34	1275.79
Wt. Dry Soil & Pan (g)	1037.24	1037.24
Wt. Lost Moisture (g)	258.10	238.55
Wt. of Pan Only (g)	13.81	13.81
Wt. of Dry Soil (g)	1023.43	1023.43
Moisture Content %	25.2	23.3
Wet Density (pcf)	126.4	132.4
Dry Density (pcf)	101.0	107.3
Init. Diameter (in)	2.869	
Init. Area (sq in)	6.465	
Init. Height (in)	5.973	
Vol. Bef. Consol. (cu ft)	0.02235	
Vol. After Consol. (cu ft)	0.02102	

Notes & Comments: Did not use slotted filter paper drains due to permeability test prior to shear.

Data entry by: CAL Date: 10/24/2018 Technician CAL
Data checked by: SPH Date: 10-24-18
FileName: 2679_130_TX_CUpp_ASTMD_4767_R0_2.xls

ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/22/2018
BORING NO.	B-103	CELL NUMBER	18S
DEPTH	27.5-29.9' (28.65-29.15')	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. B	CONF. PRES. (psf)	6000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

SATURATION DATA

Cell Pressure (psi)	Back Pressure (psi)	Burette Reading (cc)		Pore Pressure (psi)		Change	B
		Close	Open	Close	Open		
40.0	38.0	4.0	18.3				
50.0	48.0	19.3	20.1	38.4	47.8	9.4	0.94
60.0		20.4	20.5	48.5	58.0	9.5	0.95

CONSOLIDATION DATA

Elapsed Time (min)	SQRT TIME (min)	Burette Reading (cc)	Volume Deflection (cc)
0.00	0.0	0.30	0.00
0.25	0.5	24.30	-24.00
0.5	0.7	25.40	-25.10
1	1.0	26.10	-25.80
2	1.4	26.60	-26.30
4	2.0	27.20	-26.90
9	3.0	27.80	-27.50
16	4.0	28.30	-28.00
30	5.5	28.90	-28.60
60	7.7	29.50	-29.20
120	11.0	30.10	-29.80
240	15.5	30.80	-30.50
360	19.0	31.10	-30.80
1440	37.9	32.20	-31.90

Initial Height (in)	5.973	Initial Vol. (cc)	632.882
Height Change (in)	0.116	Vol. Change (cc)	49.500
Ht. After Cons. (in)	5.857	Cell Exp. (cc)	11.962
Initial Area (sq in)	6.465	Net Change (cc)	37.538
Area After Cons. (sq in)	6.202	Cons. Vol. (cc)	595.343

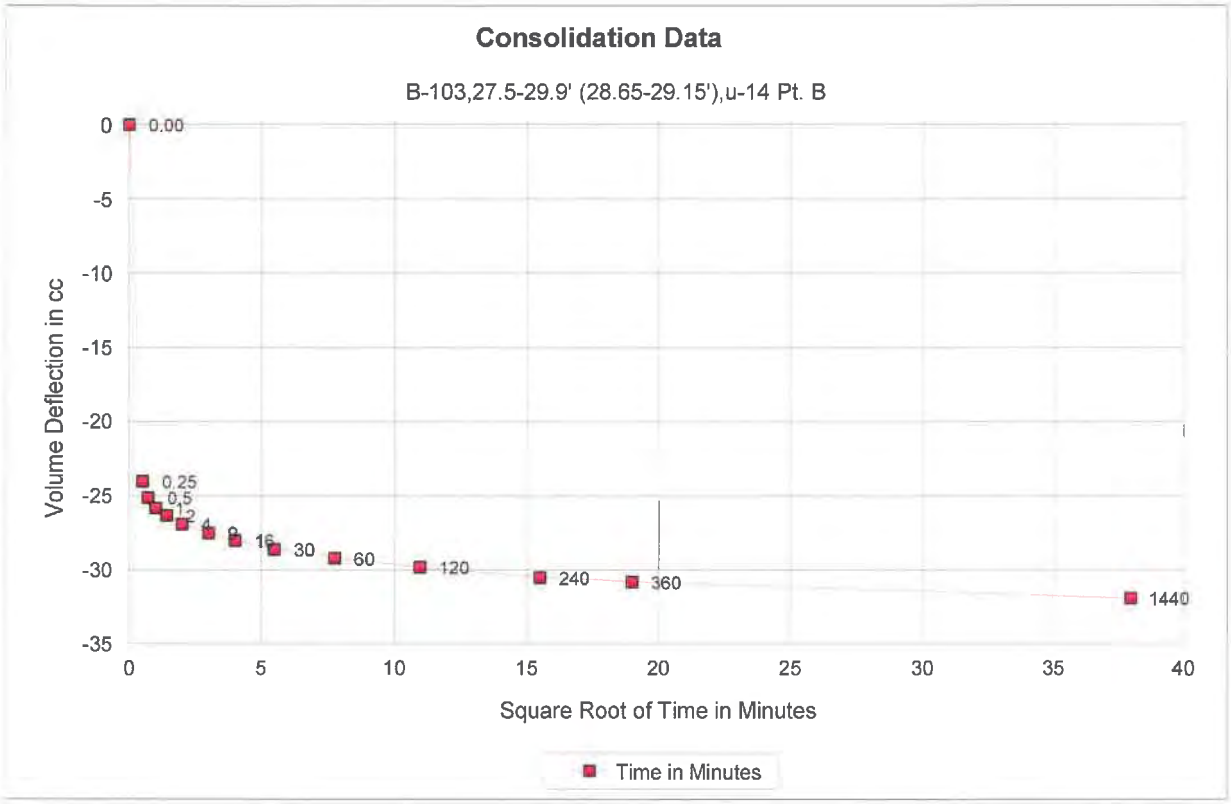
ATT
ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D 4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/22/2018
BORING NO.	B-103	CELL NUMBER	18S
DEPTH	27.5-29.9' (28.65-29.15')	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. B	CONF. PRES. (psf)	6000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

Init. Ht. (in)	5.973	Init. Area (sq in)	6.465
Consol. Ht. (in)	5.857	Consol. Area (sq in)	6.202
Back Pres. (psi)	48.3	Strain Rate (in/min)	0.0023

Axial Load (lbs.)	Axial Load (psf)	Delta Ht. (in)	Axial Strain (%)	Area Final (sq in)	Dev. Stress (psf)	Pore Pres. (psi)	Delta Pres. (psf)	Sigma 3' (psf)	Sigma 1' (psf)	Prin. Stress Ratio
0.0	0	0.000	0.00	6.202	0	48.3	0	6000	6000	1.00
7.6	176	0.005	0.09	6.207	176	48.7	58	5942	6119	1.03
20.5	476	0.010	0.17	6.212	475	49.8	216	5784	6259	1.08
73.2	1700	0.016	0.27	6.219	1695	53.6	763	5237	6932	1.32
113	2624	0.021	0.36	6.224	2614	57.9	1382	4618	7232	1.57
138	3204	0.027	0.46	6.230	3189	61.7	1930	4070	7260	1.78
153	3553	0.033	0.56	6.237	3533	64.7	2362	3638	7171	1.97
164	3808	0.039	0.67	6.243	3783	67.2	2722	3278	7061	2.15
172	3994	0.044	0.75	6.249	3964	69.1	2995	3005	6969	2.32
177	4110	0.050	0.85	6.255	4075	70.6	3211	2789	6864	2.46
182	4226	0.056	0.96	6.262	4186	71.8	3384	2616	6802	2.60
196	4551	0.085	1.45	6.293	4485	75.4	3902	2098	6583	3.14
204	4737	0.115	1.96	6.326	4644	76.9	4118	1882	6525	3.47
211	4899	0.144	2.46	6.358	4779	77.5	4205	1795	6574	3.66
218	5062	0.173	2.95	6.390	4912	77.8	4248	1752	6664	3.80
229	5317	0.231	3.94	6.456	5108	78.2	4306	1694	6802	4.01
240	5573	0.291	4.97	6.526	5296	78.0	4277	1723	7019	4.07
249	5782	0.350	5.98	6.596	5436	77.5	4205	1795	7231	4.03
257	5967	0.409	6.98	6.667	5551	77.2	4162	1838	7389	4.02
263	6107	0.467	7.97	6.739	5620	76.7	4090	1910	7530	3.94
265	6153	0.526	8.98	6.814	5601	76.4	4046	1954	7554	3.87
262	6083	0.585	9.99	6.890	5476	76.2	4018	1982	7458	3.76
255	5921	0.644	11.00	6.968	5270	76.3	4032	1968	7238	3.68
250	5805	0.703	12.00	7.048	5108	76.7	4090	1910	7018	3.67
246	5712	0.761	12.99	7.128	4970	76.9	4118	1882	6851	3.64
242	5619	0.821	14.02	7.213	4831	77.1	4147	1853	6684	3.61
238	5526	0.886	15.13	7.307	4690	77.6	4219	1781	6471	3.63


Consolidation Data
B-103,27.5-29.9' (28.65-29.15'),u-14 Pt. B
Volume Deflection in cc
Square Root of Time in Minutes
Time in Minutes
0.00
0.25
0.5
1
2
4
8
15
30
60
120
240
360
1440

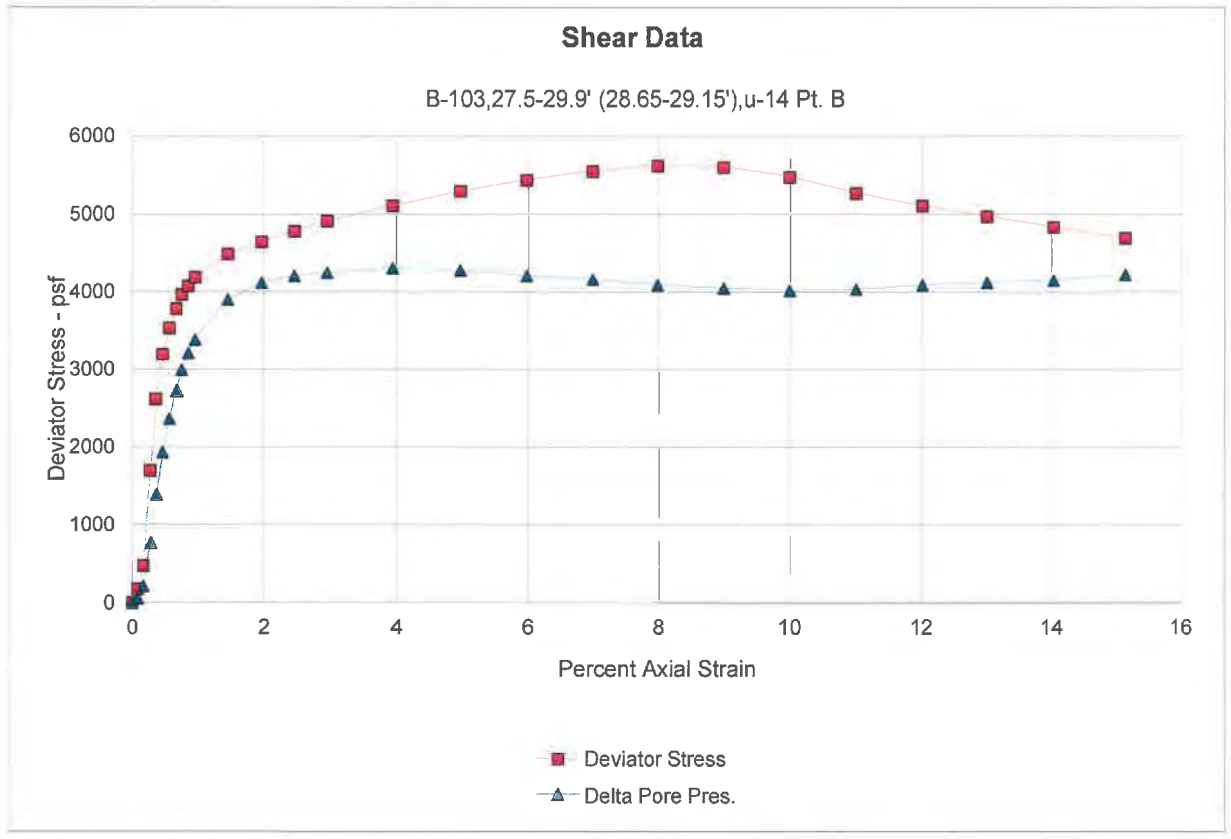
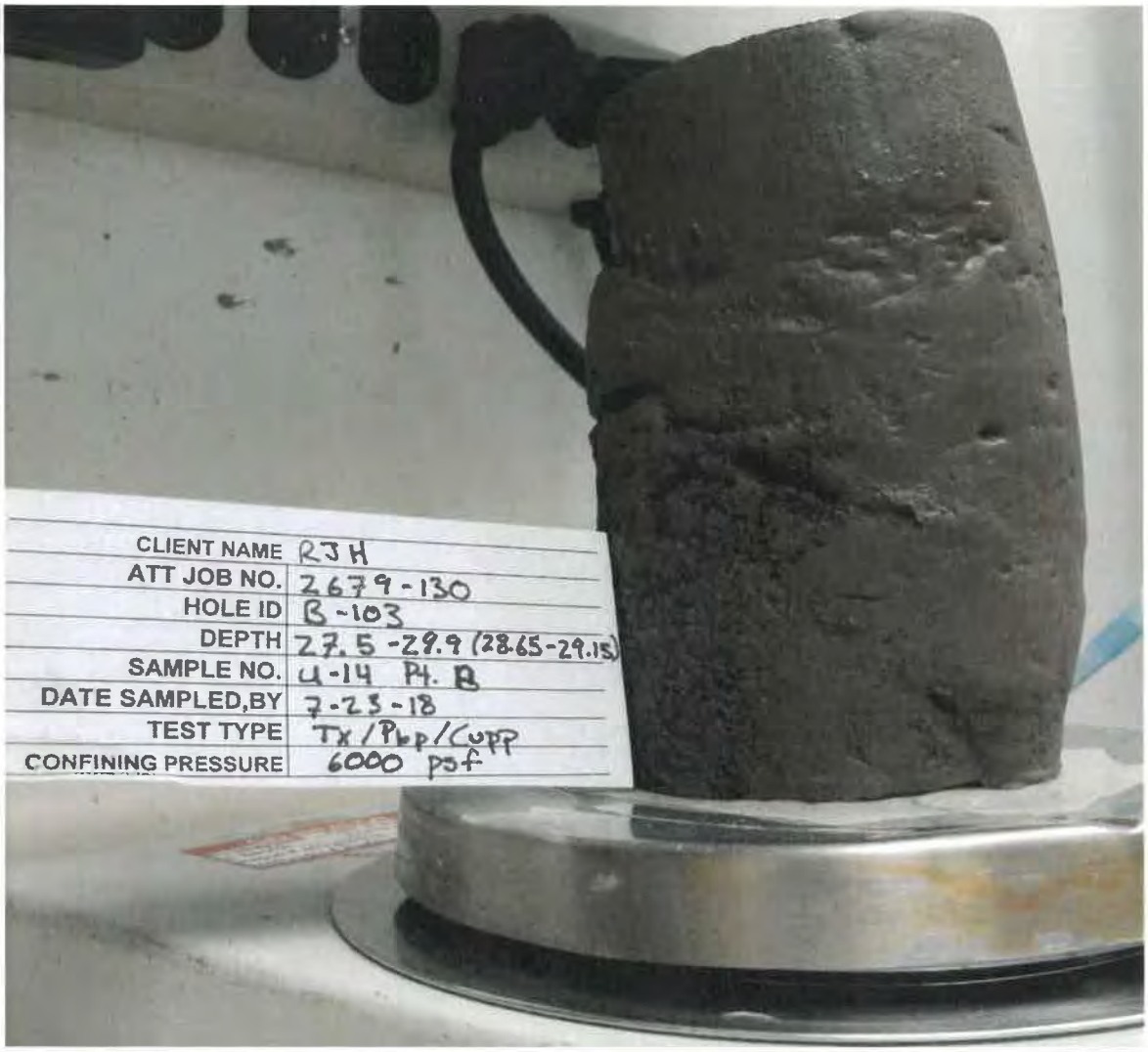

Shear Data
B-103,27.5-29.9' (28.65-29.15'),u-14 Pt. B
Deviator Stress - psf
Percent Axial Strain
Deviator Stress
Delta Pore Pres.

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants
JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation
PROJECT NO.	18115
LOCATION	--



NOTES

File name: 2679130__Image_18_10_24_07_21_35

ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/22/2018
BORING NO.	B-103	CELL NUMBER	16S
DEPTH	28.15-28.65'	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. C	CONF. PRES. (psf)	3000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

MOISTURE/DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture (g)	1321.10	1316.30
Wt. Wet Soil & Pan (g)	1335.31	1330.51
Wt. Dry Soil & Pan (g)	1081.21	1081.21
Wt. Lost Moisture (g)	254.10	249.30
Wt. of Pan Only (g)	14.21	14.21
Wt. of Dry Soil (g)	1067.00	1067.00
Moisture Content %	23.8	23.4
Wet Density (pcf)	127.5	132.5
Dry Density (pcf)	103.0	107.4
Init. Diameter (in)	2.868	
Init. Area (sq in)	6.460	
Init. Height (in)	6.109	
Vol. Bef. Consol. (cu ft)	0.02284	
Vol. After Consol. (cu ft)	0.02191	

Notes & Comments: Did not use slotted filter paper drains due to permeability test prior to shear.

Data entry by: SPH Date: 10/23/2018 Technician CAL
Data checked by: CAL Date: 10/23/2018
FileName: 2679_130_TX_CUpp_ASTMD_4767_R0_0.xls

ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/22/2018
BORING NO.	B-103	CELL NUMBER	16S
DEPTH	28.15-28.65'	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. C	CONF. PRES. (psf)	3000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

SATURATION DATA

Cell Pressure (psi)	Back Pressure (psi)	Burette Reading (cc)		Pore Pressure (psi)		Change	B
		Close	Open	Close	Open		
40.0	38.0	4.0	19.8				
50.0	48.0	20.6	21.6	38.9	48.2	9.3	0.93
60.0		21.9	22.0	48.7	58.3	9.6	0.96

CONSOLIDATION DATA

Elapsed Time (min)	SQRT TIME (min)	Burette Reading (cc)	Volume Deflection (cc)
0.00	0.0	0.50	0.00
0.25	0.5	12.45	-11.95
0.5	0.7	13.70	-13.20
1	1.0	14.80	-14.30
2	1.4	15.40	-14.90
4	2.0	15.90	-15.40
9	3.0	16.25	-15.75
16	4.0	16.65	-16.15
30	5.5	17.00	-16.50
60	7.7	17.50	-17.00
120	11.0	17.90	-17.40
240	15.5	18.30	-17.80
360	19.0	18.50	-18.00
1440	37.9	19.30	-18.80

Initial Height (in)	6.109	Initial Vol. (cc)	646.841
Height Change (in)	0.054	Vol. Change (cc)	37.800
Ht. After Cons. (in)	6.055	Cell Exp. (cc)	11.361
Initial Area (sq in)	6.460	Net Change (cc)	26.439
Area After Cons. (sq in)	6.252	Cons. Vol. (cc)	620.401

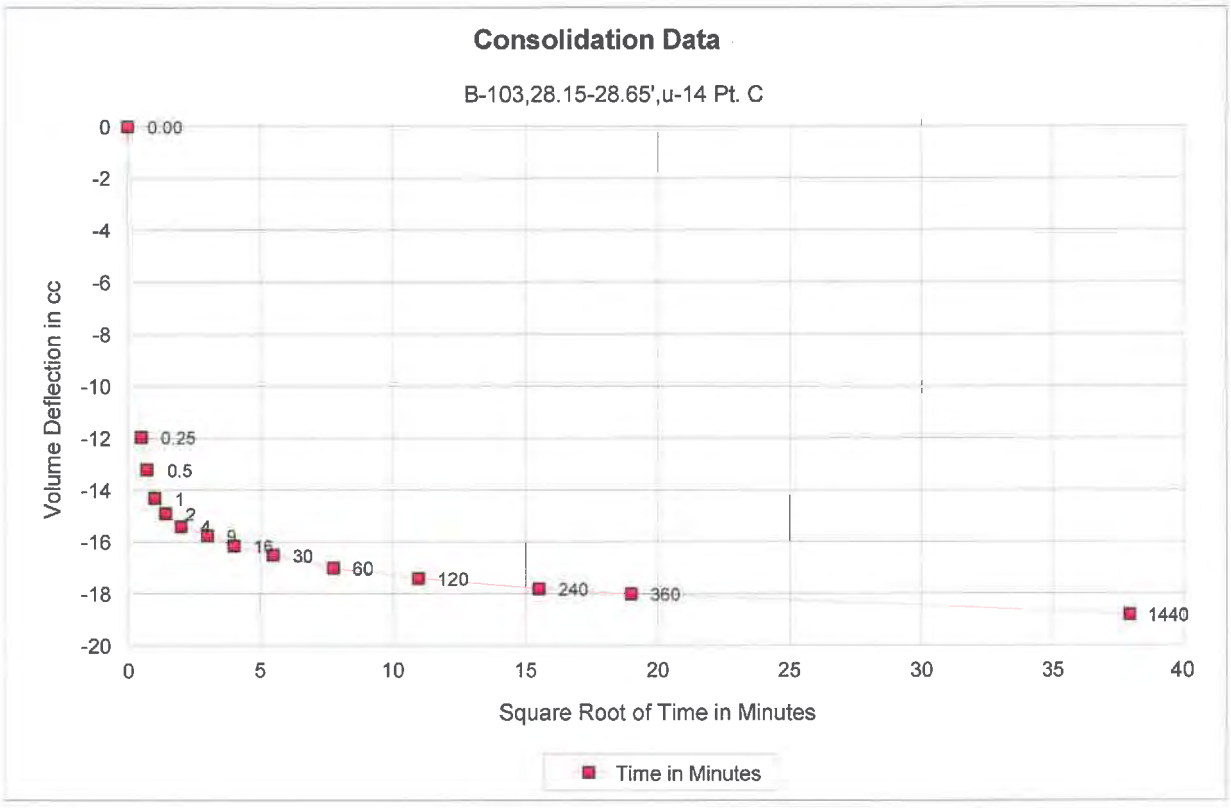
ADVANCED TERRA TESTING

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS
ASTM D 4767

CLIENT	RJH Consultants	JOB NO.	2679-130
PROJECT	Hogchute Dam Safety Evaluation	Date Sampled	7/23/2018
PROJECT NO.	18115	Date Tested	10/22/2018
BORING NO.	B-103	CELL NUMBER	16S
DEPTH	28.15-28.65'	SATURATED TEST	Yes
SAMPLE NO.	u-14 Pt. C	CONF. PRES. (psf)	3000
LOCATION	--		
SAMPLE TYPE	Shelby Tube		

Init. Ht. (in)	6.109	Init. Area (sq in)	6.460
Consol. Ht. (in)	6.055	Consol. Area (sq in)	6.252
Back Pres. (psi)	48.3	Strain Rate (in/min)	0.0036

Axial Load (lbs.)	Axial Load (psf)	Delta Ht. (in)	Axial Strain (%)	Area Final (sq in)	Dev. Stress (psf)	Pore Pres. (psi)	Delta Pres. (psf)	Sigma 3' (psf)	Sigma 1' (psf)	Prin. Stress Ratio
0.0	0	0.000	0.00	6.252	0	48.3	0	3000	3000	1.00
12	276	0.006	0.10	6.258	276	50.7	346	2654	2931	1.10
20	461	0.011	0.19	6.263	460	51.3	432	2568	3028	1.18
47	1083	0.017	0.27	6.269	1080	53.7	778	2222	3302	1.49
69	1589	0.022	0.37	6.275	1583	55.8	1080	1920	3503	1.82
83	1912	0.028	0.45	6.280	1903	57.4	1310	1690	3593	2.13
93	2142	0.033	0.55	6.286	2130	58.5	1469	1531	3662	2.39
100	2303	0.039	0.64	6.292	2289	59.3	1584	1416	3705	2.62
105	2419	0.045	0.74	6.299	2401	59.9	1670	1330	3730	2.81
109	2511	0.052	0.85	6.305	2489	60.4	1742	1258	3747	2.98
112	2580	0.058	0.96	6.312	2555	60.7	1786	1214	3769	3.10
115	2649	0.064	1.05	6.318	2621	61.0	1829	1171	3792	3.24
127	2925	0.094	1.56	6.351	2880	61.9	1958	1042	3921	3.76
135	3110	0.125	2.06	6.384	3045	62.1	1987	1013	4058	4.01
143	3294	0.155	2.56	6.416	3209	62.0	1973	1027	4237	4.12
149	3432	0.186	3.07	6.449	3327	61.8	1944	1056	4383	4.15
159	3662	0.241	3.98	6.511	3517	61.4	1886	1114	4630	4.16
169	3893	0.302	4.99	6.580	3698	61.2	1858	1142	4841	4.24
178	4100	0.364	6.00	6.651	3854	60.7	1786	1214	5068	4.17
185	4261	0.424	7.01	6.723	3963	60.3	1728	1272	5235	4.12
190	4376	0.487	8.04	6.798	4025	59.9	1670	1330	5354	4.03
196	4515	0.547	9.04	6.873	4107	59.7	1642	1358	5465	4.02
200	4607	0.609	10.05	6.950	4144	59.3	1584	1416	5560	3.93
203	4676	0.670	11.06	7.029	4158	59.0	1541	1459	5618	3.85
207	4768	0.731	12.07	7.110	4192	58.8	1512	1488	5680	3.82
209	4814	0.793	13.10	7.194	4183	58.5	1469	1531	5715	3.73
210	4837	0.853	14.09	7.277	4156	58.4	1454	1546	5701	3.69
209	4814	0.915	15.11	7.364	4087	58.3	1440	1560	5647	3.62


Consolidation Data
B-103,28.15-28.65',u-14 Pt. C
Volume Deflection in cc
Square Root of Time in Minutes
0.00
0.25
0.5
1
2
4
8
15
30
60
120
240
360
1440
Time in Minutes

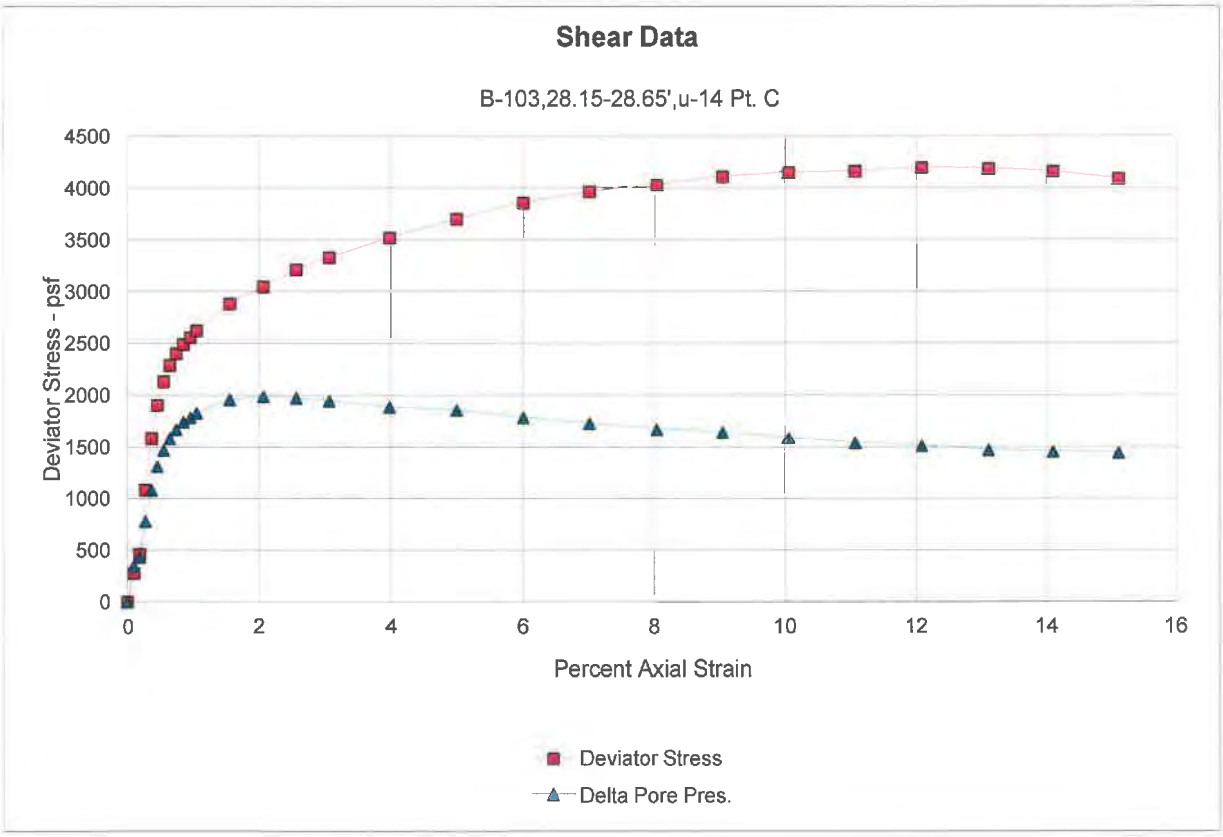
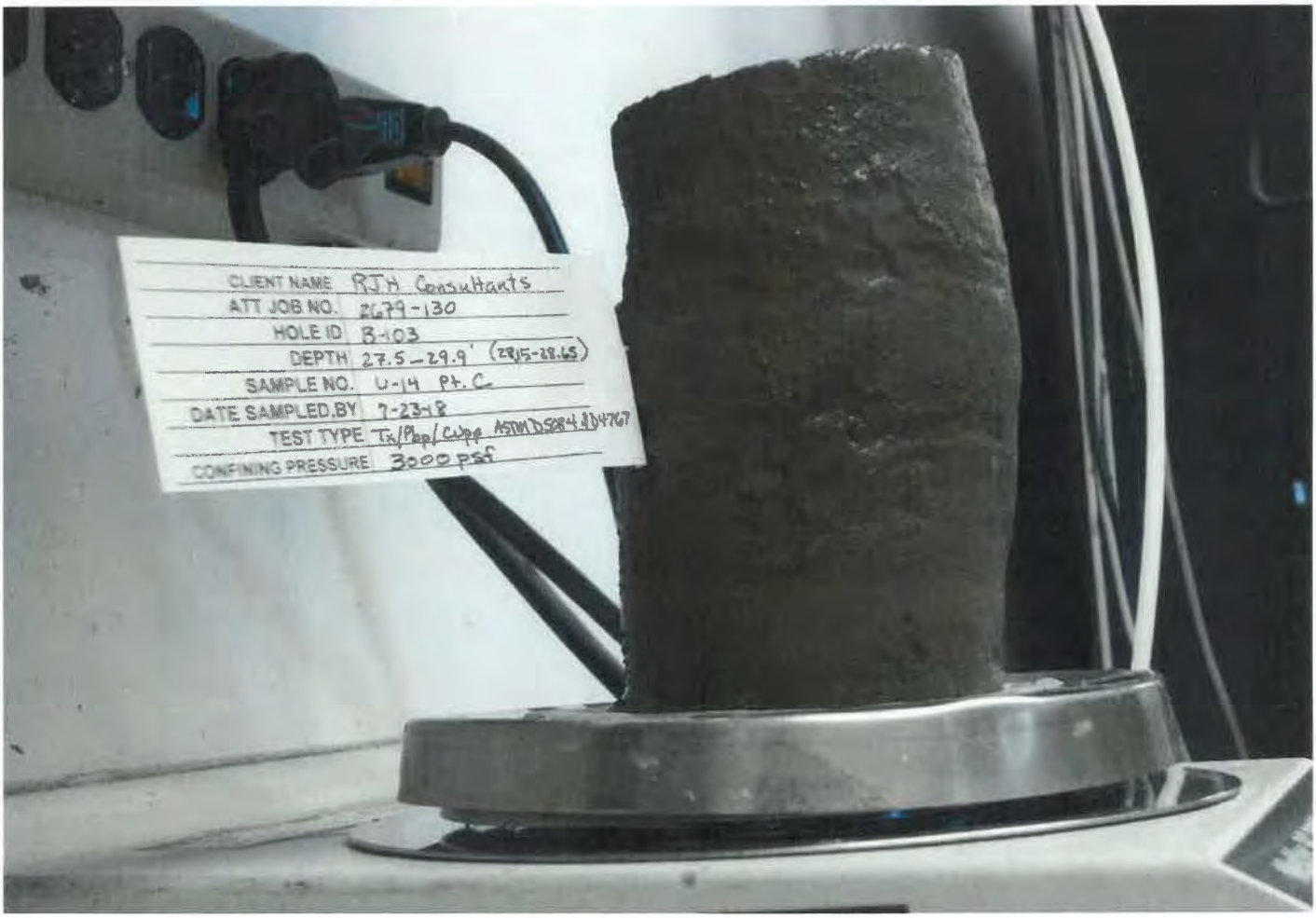

Shear Data
B-103,28.15-28.65',u-14 Pt. C
Deviator Stress - psf
Percent Axial Strain
Deviator Stress
Delta Pore Pres.

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--



NOTES

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Permeability Test
Flow Pump
ASTM D5084, Method D

ATT ADVANCED TERRA TESTING

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD

ASTM D5084 Method D

CLIENT	RJH Consultants	JOB NO.	2679-130

PROJECT	Hogchute Dam Safety Evaluation			
PROJECT NO.	18115	SAMPLED	7/23/2018	By: --
BORING NO.	B-103	TEST STARTED	10/16/2018	By: CAL
DEPTH	27.5-29.9' (29.15-29.65)	TEST FINISHED	10/19/2018	By: CAL
SAMPLE NO.	u-14 Pt. A	CELL NUMBER	24S	
LOCATION	--	PERMEANT	Tap Water	
SAMPLE TYPE	Shelby Tube	CONF. PRES. - (psf)	10000	

MOISTURE / DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture - (g)	1266.40	1226.21
Wt. Wet Soil & Pan - (g)	1280.85	1240.66
Wt. Dry Soil & Pan - (g)	1017.19	1017.19
Wt. Lost Moisture - (g)	263.66	223.47
Wt. of Pan Only - (g)	14.45	14.45
Wt. of Dry Soil - (g)	1002.74	1002.74
Moisture Content - (%)	26.3	22.3
Wet Density - (pcf)	127.3	133.3
Dry Density - (pcf)	100.8	109.0
Init. Diameter - (in)	2.872	
Init. Area - (sq in)	6.478	
Init. Height - (in)	5.852	
Vol. Bef. Consol. - (cu ft)	0.02194	
Vol. After Consol. - (cu ft)	0.02028	
Porosity - (%)	38.91	

FLOW PUMP CALCULATIONS	
Pump Setting (gear number)	7
Percentage of Pump Setting	100
Q - (cc/s)	3.54E-04
Height - (in)	5.701
Diameter - (in)	2.797
Pressure - (psi)	0.107
Area after consol. - (sq cm)	39.650
Gradient	0.520
Permeability k - (cm/s)	1.7E-05
Permeability k - (m/s)	1.7E-07
Back Pressure - (psi)	38.0
Cell Pressure - (psi)	107.4
Ave. Effective Stress - (psi)	69.347
Average temperature degree - (c°)	22.0

Data entry by:	SPH	Date:	10/23/2018
Checked by:	ekr	Date:	10/23/2018

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD

ASTM D5084 Method D

CLIENT	RJH Consultants	JOB NO.	2679-130

PROJECT	Hogchute Dam Safety Evaluation			
PROJECT NO.	18115	SAMPLED	7/23/2018	By: --
BORING NO.	B-103	TEST STARTED	10/16/2018	By: CAL
DEPTH	27.5-29.9' (29.15-29.65)	TEST FINISHED	10/19/2018	By: CAL
SAMPLE NO.	u-14 Pt. A	CELL NUMBER	24S	
LOCATION	--	PERMEANT	Tap Water	
SAMPLE TYPE	Shelby Tube	CONF. PRES. - (psf)	10000	

SATURATION DATA

Cell Press.	Back Press.	Burette Reading (cc)		Pore Press. (psi)			
(psi)	(psi)	Close	Open	Close	Open	Change	B
40.0	38.0	3.2	19.3				
50.0		20.0	20.3	38.7	48.4	9.7	0.97

CONSOLIDATION DATA

Elapsed Time (Min)	SQRT Time (Min)	Burette Reading (cc)	Volume Defl. (cc)
0.00	0.00	0.80	0.00
0.25	0.50	34.60	-33.80
0.5	0.71	36.70	-35.90
1	1.00	37.80	-37.00
2	1.41	38.60	-37.80
4	2.00	39.30	-38.50
9	3.00	40.10	-39.30
16	4.00	40.80	-40.00
30	5.48	41.50	-40.70
88	9.38	42.70	-41.90
120	10.95	43.10	-42.30
240	15.49	43.90	-43.10
360	18.97	44.30	-43.50

Initial Height - (in)	5.852	Init. Vol. - (cc)	621.36
Height Change - (in)	0.151	Vol. Change - (cc)	62.80
Ht. After Cons. - (in)	5.701	Cell Exp. - (cc)	15.70
Initial Area - (sq in)	6.478	Net Change - (cc)	47.10
Area After Cons. - sq in	6.146	Cons. Vol. - (cc)	574.26

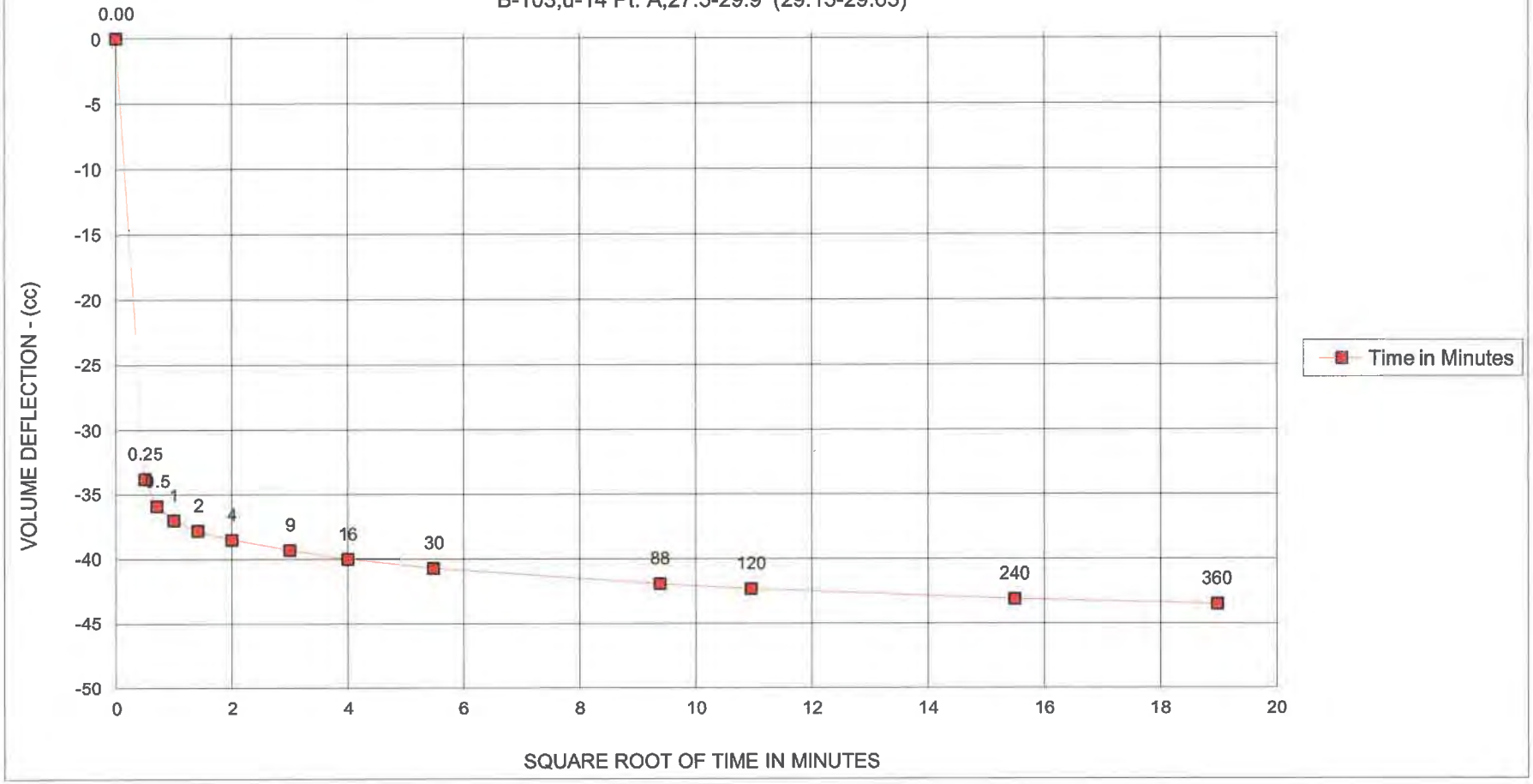
ATT
ADVANCED TERRA TESTING

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD
ASTM D5084 Method D

CLIENT	RJH Consultants		JOB NO.	2679-130

CONSOLIDATION DATA

B-103,u-14 Pt. A,27.5-29.9' (29.15-29.65)

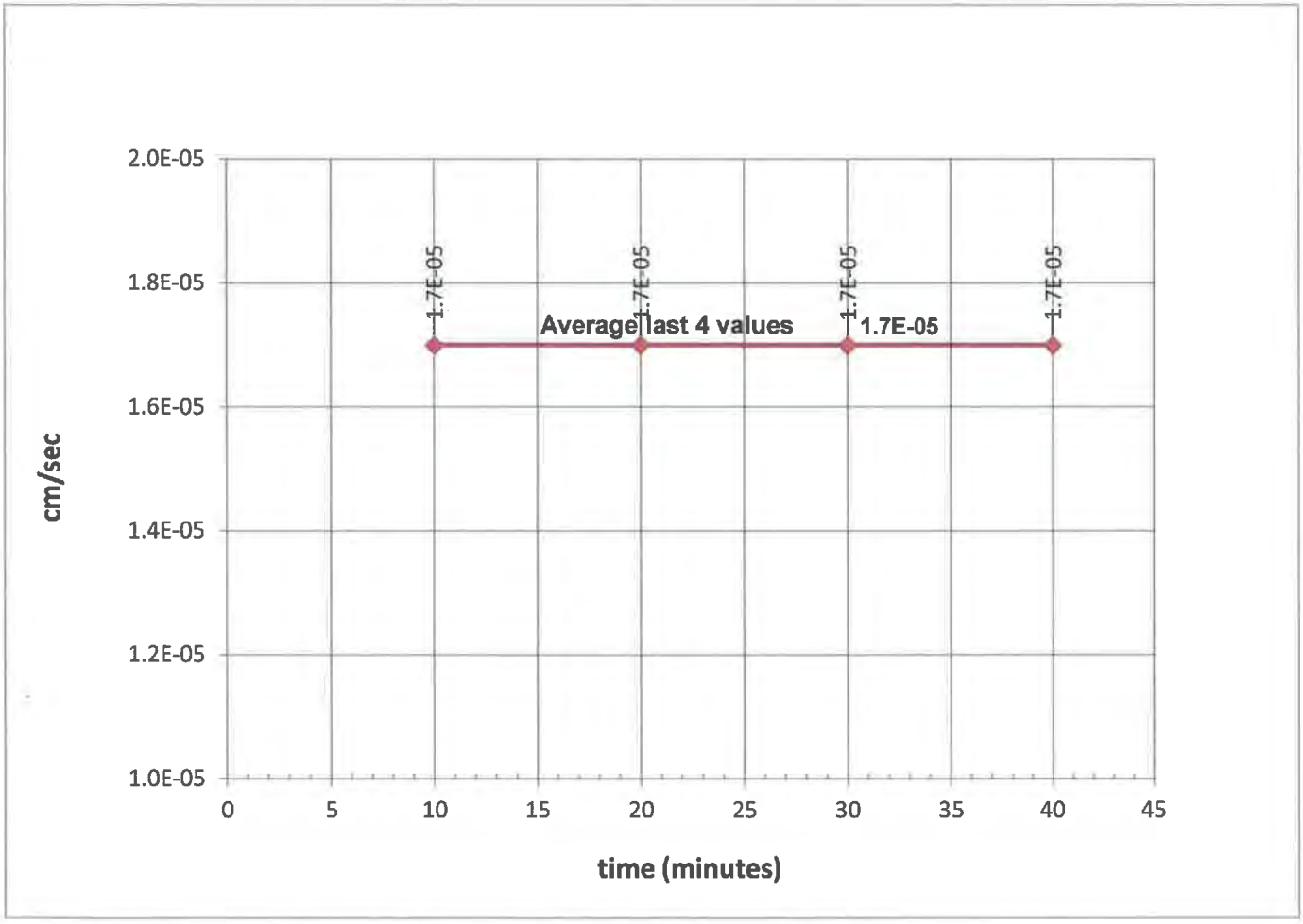


FileName: 2679_130_HarvardFlowPump-Perm-ASTMD-5084-R3_1.xls

ADVANCED TERRA TESTING

Preliminary Flow Pump Test Data
ASTM D5084

Client:	RJH Consultants	Boring Number:	b-103		
Job Number:	2679-130	Depth:	27.5'-29.9'		
Project:	Hogchute Dam Safety Evaluation	Sample Number:	U-14 Pt. A		
Location:	--	Sampled Date:	7/23/2018	Sampled By:	--
Project Number:	18115	Test Date:	10/19/2018	Technician:	CAL



Data Entered By: CAL
Date: 10/19/2018
File Name: 2679_130_PrelimPerm_ASTMD-5084-methodD-R0_0.xls

Checked By: SPH
Date: 10-23-18

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD

ASTM D5084 Method D

CLIENT	RJH Consultants	JOB NO.	2679-130

PROJECT	Hogchute Dam Safety Evaluation			
PROJECT NO.	18115	SAMPLED	7/23/2018	By: --
BORING NO.	B-103	TEST STARTED	10/16/2018	By: SPH/CAL
DEPTH	27.5-29.9' (28.65-29.15')	TEST FINISHED	10/22/2018	By: SPH/CAL
SAMPLE NO.	u-14 Pt. B	CELL NUMBER	18S	
LOCATION	--	PERMEANT	Tap Water	
SAMPLE TYPE	shelby tube	CONF. PRES. - (psf)	6000	

MOISTURE / DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture - (g)	1281.53	1261.98
Wt. Wet Soil & Pan - (g)	1295.34	1275.79
Wt. Dry Soil & Pan - (g)	1037.24	1037.24
Wt. Lost Moisture - (g)	258.10	238.55
Wt. of Pan Only - (g)	13.81	13.81
Wt. of Dry Soil - (g)	1023.43	1023.43
Moisture Content - (%)	25.2	23.3
Wet Density - (pcf)	126.4	132.1
Dry Density - (pcf)	101.0	107.1
Init. Diameter - (in)	2.869	
Init. Area - (sq in)	6.465	
Init. Height - (in)	5.973	
Vol. Bef. Consol. - (cu ft)	0.02235	
Vol. After Consol. - (cu ft)	0.02106	
Porosity - (%)	40.00	

FLOW PUMP CALCULATIONS	
Pump Setting (gear number)	1
Percentage of Pump Setting	100
Q - (cc/s)	3.59E-02
Height - (in)	5.857
Diameter - (in)	2.813
Pressure - (psi)	0.060
Area after consol. - (sq cm)	40.085
Gradient	0.284
Permeability k - (cm/s)	3.2E-03
Permeability k - (m/s)	3.2E-05
Back Pressure - (psi)	48.0
Cell Pressure - (psi)	89.7
Ave. Effective Stress - (psi)	41.670
Average temperature degree - (c°)	22.2

Data entry by:	CAL	Date:	10/24/2018
Checked by:	SPH	Date:	10-24-18

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD

ASTM D5084 Method D

CLIENT	RJH Consultants	JOB NO.	2679-130

PROJECT	Hogchute Dam Safety Evaluation			
PROJECT NO.	18115	SAMPLED	7/23/2018	By: --
BORING NO.	B-103	TEST STARTED	10/16/2018	By: SPH/CAL
DEPTH	27.5-29.9' (28.65-29.15')	TEST FINISHED	10/22/2018	By: SPH/CAL
SAMPLE NO.	u-14 Pt. B	CELL NUMBER	18S	
LOCATION	--	PERMEANT	Tap Water	
SAMPLE TYPE	shelby tube	CONF. PRES. - (psf)	6000	

SATURATION DATA

Cell Press.	Back Press.	Burette Reading (cc)		Pore Press. (psi)			
(psi)	(psi)	Close	Open	Close	Open	Change	B
40.0	38.0	4.0	18.3				
50.0	48.0	19.3	20.1	38.4	47.8	9.4	0.94
60.0		20.4	20.5	48.5	58.0	9.5	0.95

CONSOLIDATION DATA

Elapsed Time (Min)	SQRT Time (Min)	Burette Reading (cc)	Volume Defl. (cc)
0.00	0.00	0.30	0.00
0.25	0.50	24.30	-24.00
0.5	0.71	25.40	-25.10
1	1.00	26.10	-25.80
2	1.41	26.60	-26.30
4	2.00	27.20	-26.90
9	3.00	27.80	-27.50
16	4.00	28.30	-28.00
30	5.48	28.90	-28.60
60	7.75	29.50	-29.20
120	10.95	30.10	-29.80
240	15.49	30.80	-30.50
360	18.97	31.10	-30.80

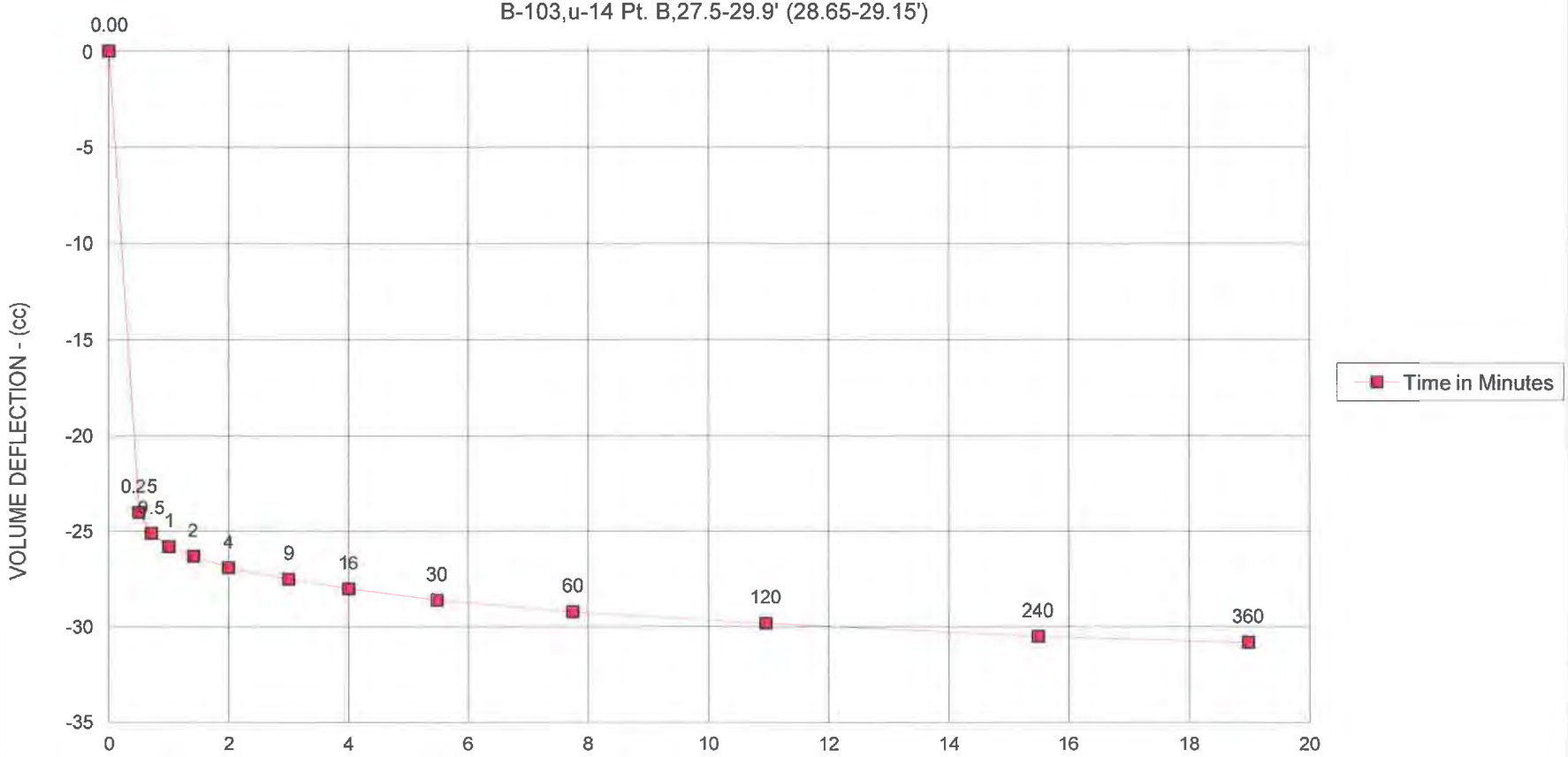
Initial Height - (in)	5.973	Init. Vol. - (cc)	632.88
Height Change - (in)	0.116	Vol. Change - (cc)	48.40
Ht. After Cons. - (in)	5.857	Cell Exp. - (cc)	11.96
Initial Area - (sq in)	6.465	Net Change - (cc)	36.44
Area After Cons. - sq in	6.213	Cons. Vol. - (cc)	596.44

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD
ASTM D5084 Method D

CLIENT	RJH Consultants		JOB NO.	2679-130

CONSOLIDATION DATA

B-103,u-14 Pt. B,27.5-29.9' (28.65-29.15')

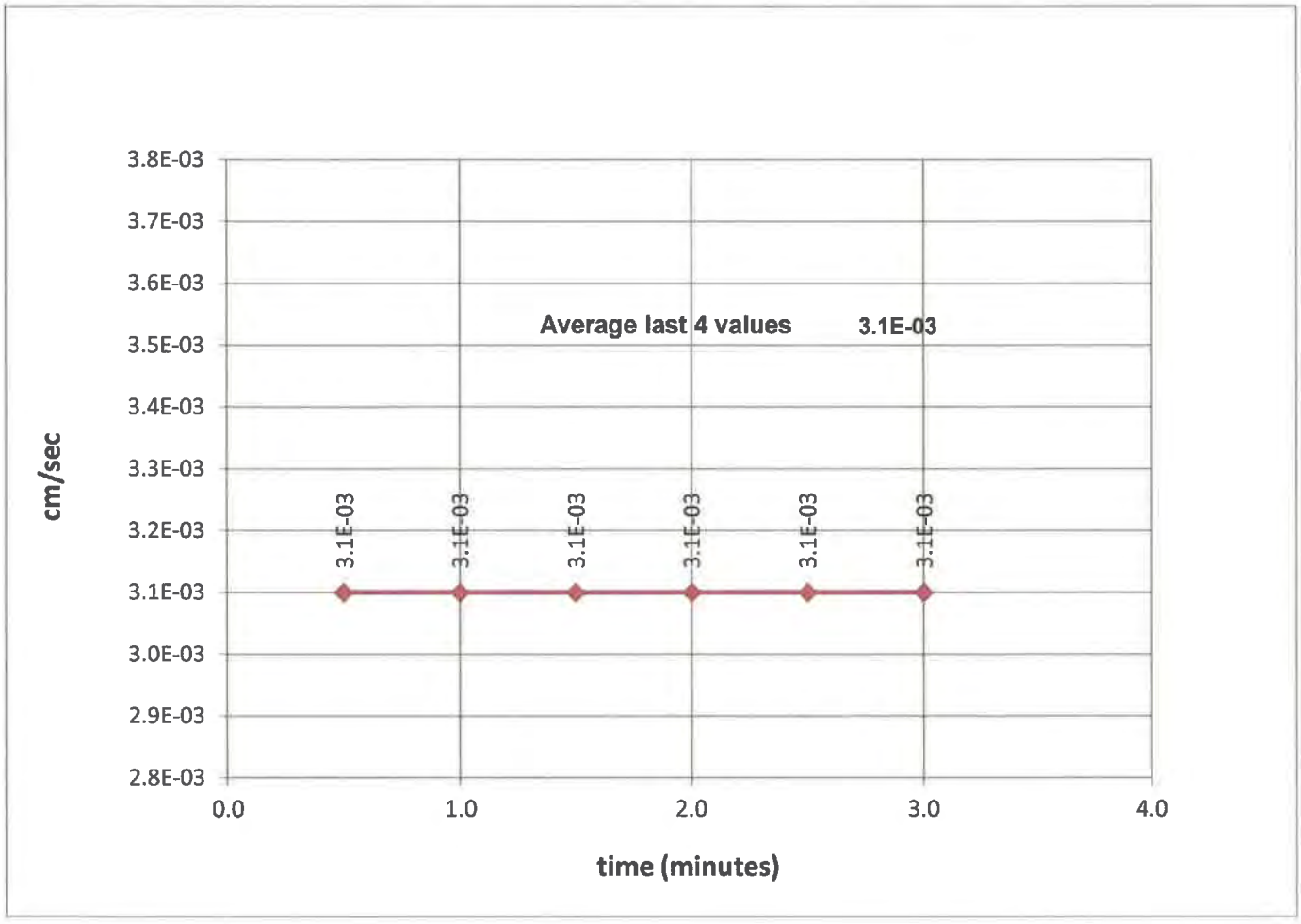


FileName: 2679_130_HarvardFlowPump-Perm-ASTMD-5084-R3_2.xls

ADVANCED TERRA TESTING

Preliminary Flow Pump Test Data
ASTM D5084

Client:	RJH Consultants	Boring Number:	B-103		
Job Number:	2679-130	Depth:	27.5-29.9'		
Project:	Hogchute Dam Safety Evaluation	Sample Number:	U-14 Pt. B		
Location:	--	Sampled Date:	7/23/2018	Sampled By:	--
Project Number:	18115	Test Date:	10/19/2018	Technician:	CAL



Data Entered By: CAL
Date: 10/19/2018
File Name: 2679_130_PrelimPerm_ASTMD-5084-methodD-R0_1.xls

Checked By: SPH
Date: 10-24-18

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD

ASTM D5084 Method D

CLIENT	RJH Consultants	JOB NO.	2679-130

PROJECT	Hogchute Dam Safety Evaluation			
PROJECT NO.	18115	SAMPLED	7/23/2018	By: --
BORING NO.	B-103	TEST STARTED	10/16/2018	By: SPH/CAL
DEPTH	28.15-25.65'	TEST FINISHED	10/19/2018	By: SPH/CAL
SAMPLE NO.	u-14 Pt. C	CELL NUMBER	16S	
LOCATION	--	PERMEANT	Tap Water	
SAMPLE TYPE	Shelby Tube	CONF. PRES. - (psf)	3000	

MOISTURE / DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture - (g)	1321.10	1316.30
Wt. Wet Soil & Pan - (g)	1335.31	1330.51
Wt. Dry Soil & Pan - (g)	1081.21	1081.21
Wt. Lost Moisture - (g)	254.10	249.30
Wt. of Pan Only - (g)	14.21	14.21
Wt. of Dry Soil - (g)	1067.00	1067.00
Moisture Content - (%)	23.8	23.4
Wet Density - (pcf)	127.5	132.3
Dry Density - (pcf)	103.0	107.2
Init. Diameter - (in)	2.868	
Init. Area - (sq in)	6.460	
Init. Height - (in)	6.109	
Vol. Bef. Consol. - (cu ft)	0.02284	
Vol. After Consol. - (cu ft)	0.02194	
Porosity - (%)	40.12	

FLOW PUMP CALCULATIONS	
Pump Setting (gear number)	6
Percentage of Pump Setting	100
Q - (cc/s)	7.07E-04
Height - (in)	6.055
Diameter - (in)	2.824
Pressure - (psi)	0.111
Area after consol. - (sq cm)	40.399
Gradient	0.507
Permeability k - (cm/s)	3.4E-05
Permeability k - (m/s)	3.4E-07
Back Pressure - (psi)	48.0
Cell Pressure - (psi)	68.8
Ave. Effective Stress - (psi)	20.745
Average temperature degree - (c°)	22.1

Data entry by:	SPH	Date:	10/23/2018
Checked by:	CAL	Date:	10/23/2018

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD

ASTM D5084 Method D

CLIENT	RJH Consultants	JOB NO.	2679-130

PROJECT	Hogchute Dam Safety Evaluation			
PROJECT NO.	18115	SAMPLED	7/23/2018	By: --
BORING NO.	B-103	TEST STARTED	10/16/2018	By: SPH/CAL
DEPTH	28.15-25.65'	TEST FINISHED	10/19/2018	By: SPH/CAL
SAMPLE NO.	u-14 Pt. C	CELL NUMBER	16S	
LOCATION	--	PERMEANT	Tap Water	
SAMPLE TYPE	Shelby Tube	CONF. PRES. - (psf)	3000	

SATURATION DATA

Cell Press.	Back Press.	Burette Reading (cc)		Pore Press. (psi)			
(psi)	(psi)	Close	Open	Close	Open	Change	B
40.0	38.0	4.0	19.8				
50.0	48.0	20.6	21.6	38.9	48.2	9.3	0.93
60.0		21.9	22.0	48.7	58.3	9.6	0.96

CONSOLIDATION DATA

Elapsed Time (Min)	SQRT Time (Min)	Burette Reading (cc)	Volume Defl. (cc)
0.00	0.00	0.50	0.00
0.25	0.50	12.45	-11.95
0.5	0.71	13.70	-13.20
1	1.00	14.80	-14.30
2	1.41	15.40	-14.90
4	2.00	15.90	-15.40
9	3.00	16.25	-15.75
16	4.00	16.65	-16.15
30	5.48	17.00	-16.50
60	7.75	17.50	-17.00
120	10.95	17.90	-17.40
240	15.49	18.30	-17.80
360	18.97	18.50	-18.00

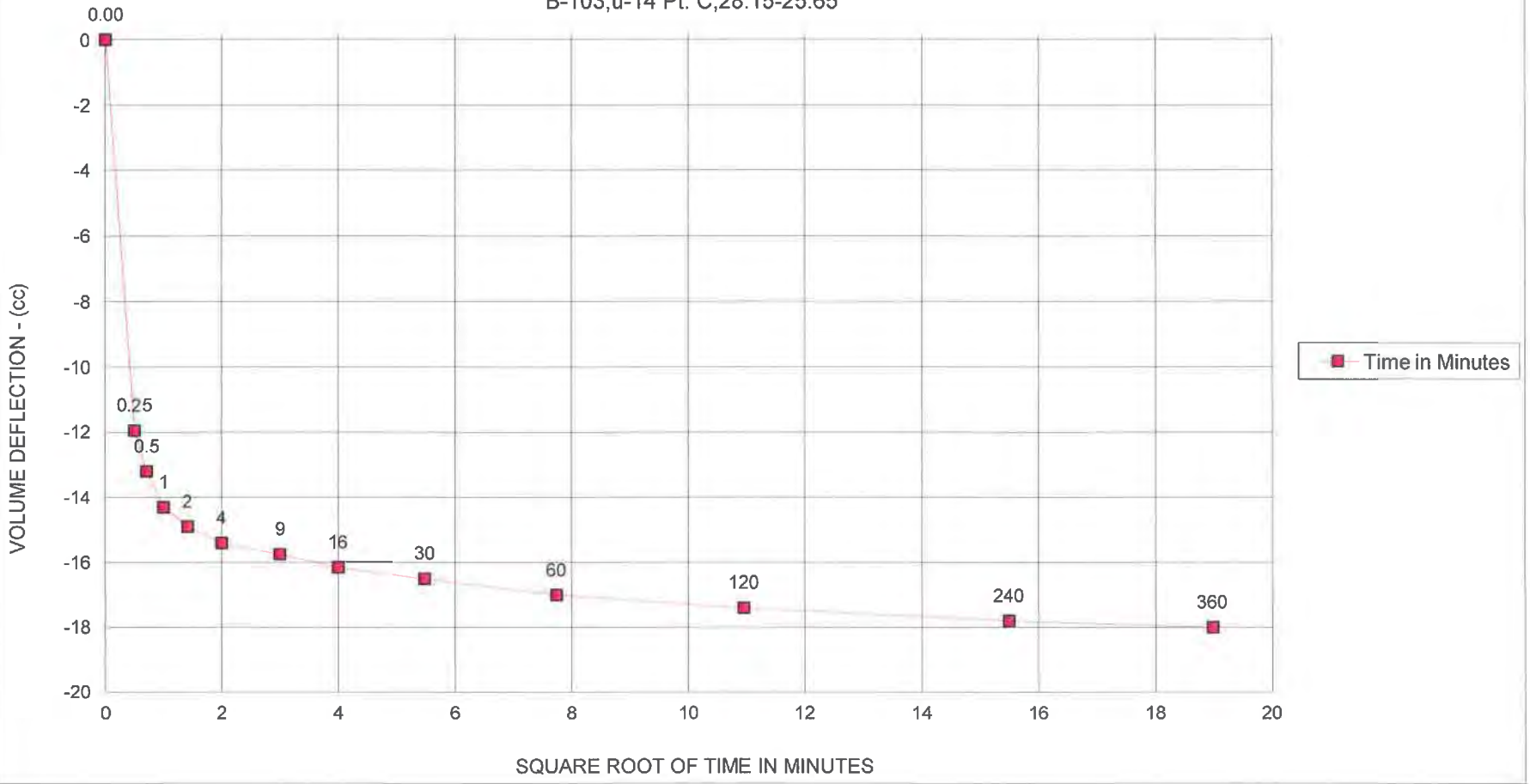
Initial Height - (in)	6.109	Init. Vol. - (cc)	646.84
Height Change - (in)	0.054	Vol. Change - (cc)	36.80
Ht. After Cons. - (in)	6.055	Cell Exp. - (cc)	11.36
Initial Area - (sq in)	6.460	Net Change - (cc)	25.44
Area After Cons. - sq in	6.262	Cons. Vol. - (cc)	621.40

PERMEABILITY TEST - BACK PRESSURE SATURATED - FLOW PUMP METHOD
ASTM D5084 Method D

CLIENT	RJH Consultants		JOB NO.	2679-130

CONSOLIDATION DATA

B-103,u-14 Pt. C,28.15-25.65'

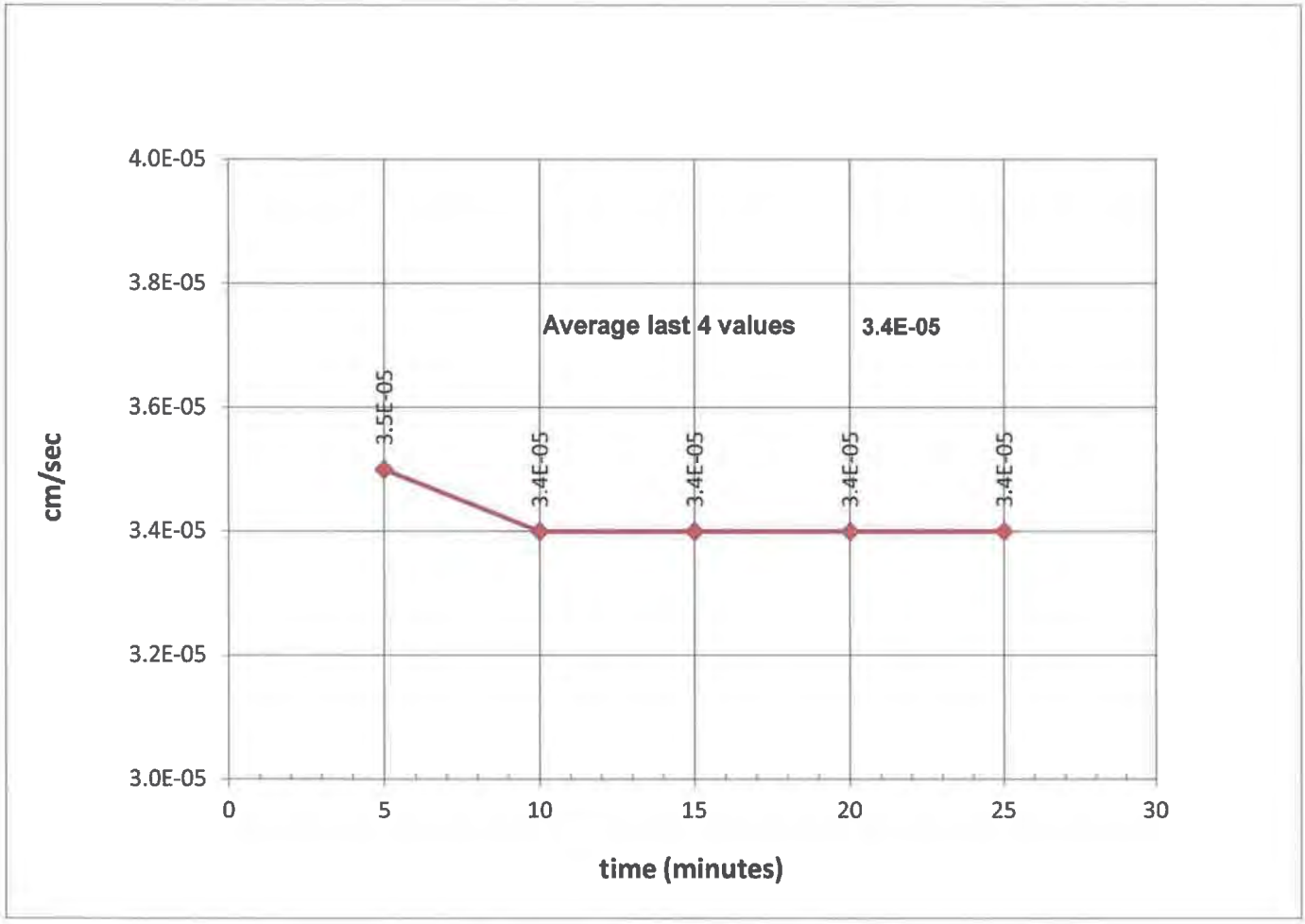


FileName: 2679_130_HarvardFlowPump-Perm-ASTMD-5084-R3_0.xls

ADVANCED TERRA TESTING

Preliminary Flow Pump Test Data
ASTM D5084

Client:	RJH Consultants	Boring Number:	B-103		
Job Number:	2679-130	Depth:	27.5-29.9'		
Project:	Hogchute Dam Safety Evaluation	Sample Number:	U-14 Pt. C		
Location:	--	Sampled Date:	7/23/2018	Sampled By:	--
Project Number:	18115	Test Date:	10/19/2018	Technician:	CAL



Data Entered By: CAL
Date: 10/19/2018
File Name: 2679_130_PrelimPerm_ASTMD-5084-methodD-R0_2.xls

Checked By: SPH
Date: 10-23-18

Unconfined Compressive Strength
ASTM D2166

ADVANCED TERRA TESTING

Unconfined Compressive Strength
ASTM D 2166

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Test Parameters

Strain Rate (in/min):	0.037
Strain Rate (cm/min):	0.09398

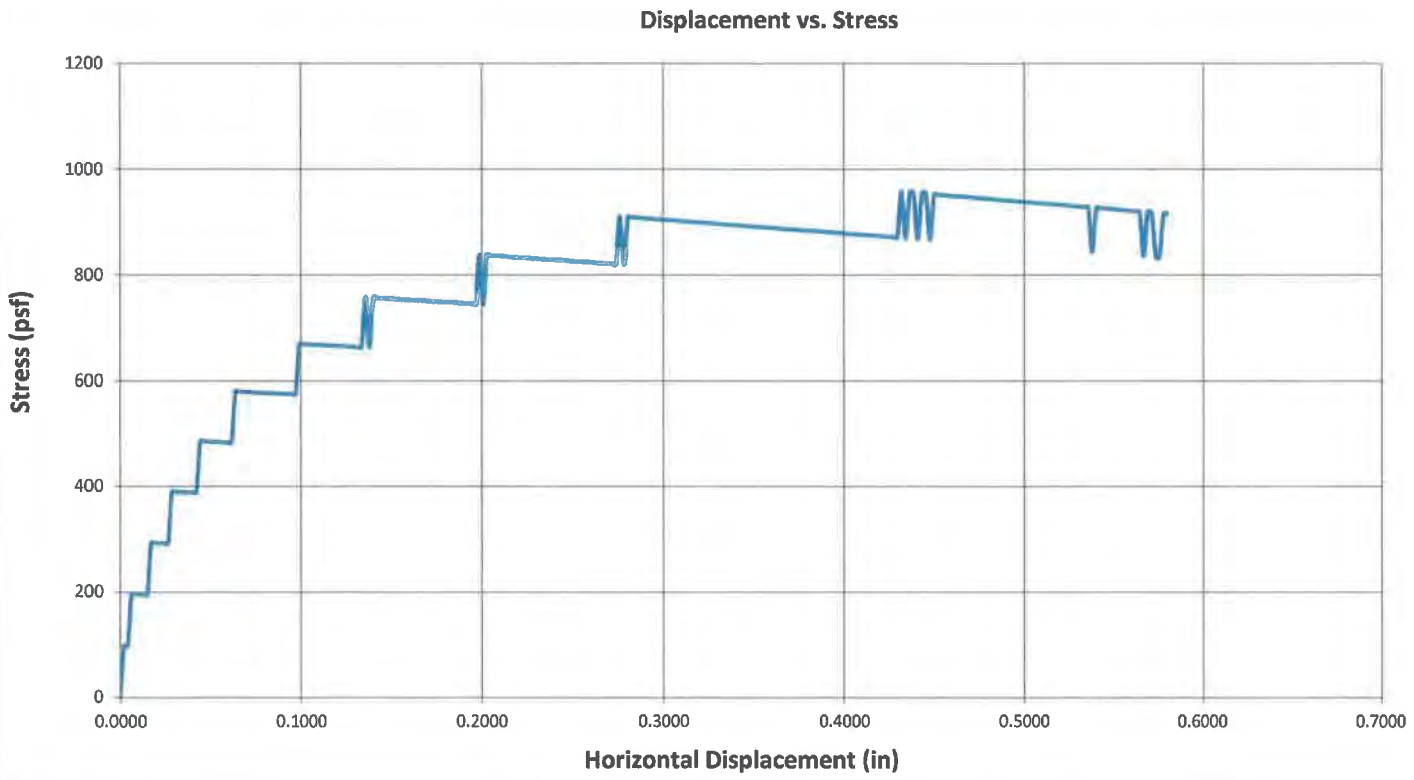
Raw Data Files: RJH_2679_130_CA-9_B-104.txt

Moisture & Density Data

Mass of Wet Soil and Pan (g):	531.30	Initial Wet Density (pcf):	133.9
Mass of Dry Soil and Pan (g):	466.78	Initial Dry Density (pcf):	111.8
Mass of Pan (g):	139.65	Initial Wet Density (kg/m³):	2144
Mass of Wet Soil (g):	391.7	Initial Dry Density (kg/m³):	1791
Initial Diameter (in):	1.93	Initial Moisture (%):	19.7
Initial Height (in):	3.80		

Test Results

Peak Stress (psf):	958	Axial Strain at Peak Stress(%):	11.4
Peak Stress (kPa):	46	Height to Diameter Ratio:	2.0:1



NOTES:	Short Sample! Filling required due to gravel.

Data entry by:	CAL	Date:	10/18/2018
Checked by:	SPH	Date:	10-19-18
File name:	2679130__UCS ASTM D2166_0.xlsm		

Unconfined Compressive Strength
ASTM D2166

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Displacement (in)	Displacement (cm)	Strain (%)	Average Cross Sectional Area (in²)	Load (lbs)	Load (N)	Stress (psf)	Stress (kPa)
0.0000	0.000	0.00	2.93	0	0	0	0
0.0022	0.006	0.06	2.93	2	9	98	5
0.0043	0.011	0.11	2.93	2	9	98	5
0.0064	0.016	0.17	2.94	4	18	196	9
0.0086	0.022	0.23	2.94	4	18	196	9
0.0106	0.027	0.28	2.94	4	18	196	9
0.0127	0.032	0.33	2.94	4	18	196	9
0.0150	0.038	0.39	2.94	4	18	196	9
0.0170	0.043	0.45	2.94	6	27	293	14
0.0192	0.049	0.50	2.95	6	27	293	14
0.0218	0.055	0.57	2.95	6	27	293	14
0.0240	0.061	0.63	2.95	6	27	293	14
0.0263	0.067	0.69	2.95	6	27	293	14
0.0284	0.072	0.75	2.95	8	36	390	19
0.0307	0.078	0.81	2.96	8	36	390	19
0.0328	0.083	0.86	2.96	8	36	390	19
0.0352	0.089	0.93	2.96	8	36	389	19
0.0376	0.096	0.99	2.96	8	36	389	19
0.0399	0.101	1.05	2.96	8	36	389	19
0.0419	0.106	1.10	2.96	8	36	389	19
0.0441	0.112	1.16	2.97	10	44	486	23
0.0464	0.118	1.22	2.97	10	44	485	23
0.0486	0.123	1.28	2.97	10	44	485	23
0.0507	0.129	1.33	2.97	10	44	485	23
0.0531	0.135	1.40	2.97	10	44	484	23
0.0551	0.140	1.45	2.97	10	44	484	23
0.0572	0.145	1.50	2.98	10	44	484	23
0.0592	0.150	1.56	2.98	10	44	484	23
0.0614	0.156	1.61	2.98	10	44	483	23
0.0636	0.162	1.67	2.98	12	53	580	28
0.0656	0.167	1.73	2.98	12	53	579	28
0.0679	0.172	1.79	2.98	12	53	579	28
0.0700	0.178	1.84	2.99	12	53	579	28
0.0723	0.184	1.90	2.99	12	53	578	28
0.0745	0.189	1.96	2.99	12	53	578	28
0.0768	0.195	2.02	2.99	12	53	578	28
0.0791	0.201	2.08	2.99	12	53	577	28
0.0813	0.207	2.14	3.00	12	53	577	28
0.0836	0.212	2.20	3.00	12	53	576	28
0.0859	0.218	2.26	3.00	12	53	576	28
0.0880	0.224	2.31	3.00	12	53	576	28
0.0903	0.229	2.38	3.00	12	53	575	28
0.0924	0.235	2.43	3.00	12	53	575	28

Unconfined Compressive Strength
ASTM D2166

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Displacement (in)	Displacement (cm)	Strain (%)	Average Cross Sectional Area (in²)	Load (lbs)	Load (N)	Stress (psf)	Stress (kPa)
0.0947	0.241	2.49	3.01	12	53	575	28
0.0968	0.246	2.55	3.01	12	53	574	28
0.0991	0.252	2.61	3.01	14	62	670	32
0.1012	0.257	2.66	3.01	14	62	669	32
0.1035	0.263	2.72	3.01	14	62	669	32
0.1057	0.268	2.78	3.02	14	62	669	32
0.1079	0.274	2.84	3.02	14	62	668	32
0.1100	0.279	2.89	3.02	14	62	668	32
0.1120	0.284	2.95	3.02	14	62	667	32
0.1142	0.290	3.00	3.02	14	62	667	32
0.1162	0.295	3.06	3.02	14	62	667	32
0.1183	0.300	3.11	3.03	14	62	666	32
0.1204	0.306	3.17	3.03	14	62	666	32
0.1224	0.311	3.22	3.03	14	62	666	32
0.1247	0.317	3.28	3.03	14	62	665	32
0.1268	0.322	3.34	3.03	14	62	665	32
0.1291	0.328	3.40	3.03	14	62	664	32
0.1312	0.333	3.45	3.04	14	62	664	32
0.1335	0.339	3.51	3.04	14	62	664	32
0.1356	0.344	3.57	3.04	16	71	758	36
0.1379	0.350	3.63	3.04	14	62	663	32
0.1399	0.355	3.68	3.04	16	71	757	36
0.1420	0.361	3.73	3.05	16	71	757	36
0.1443	0.367	3.80	3.05	16	71	756	36
0.1464	0.372	3.85	3.05	16	71	756	36
0.1487	0.378	3.91	3.05	16	71	755	36
0.1508	0.383	3.97	3.05	16	71	755	36
0.1531	0.389	4.03	3.05	16	71	754	36
0.1552	0.394	4.08	3.06	16	71	754	36
0.1572	0.399	4.13	3.06	16	71	753	36
0.1594	0.405	4.19	3.06	16	71	753	36
0.1616	0.410	4.25	3.06	16	71	753	36
0.1638	0.416	4.31	3.06	16	71	752	36
0.1660	0.422	4.37	3.07	16	71	752	36
0.1682	0.427	4.42	3.07	16	71	751	36
0.1704	0.433	4.48	3.07	16	71	751	36
0.1726	0.438	4.54	3.07	16	71	750	36
0.1748	0.444	4.60	3.07	16	71	750	36
0.1771	0.450	4.66	3.07	16	71	749	36
0.1792	0.455	4.71	3.08	16	71	749	36
0.1815	0.461	4.77	3.08	16	71	748	36
0.1836	0.466	4.83	3.08	16	71	748	36
0.1859	0.472	4.89	3.08	16	71	747	36

Unconfined Compressive Strength
ASTM D2166

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Displacement (in)	Displacement (cm)	Strain (%)	Average Cross Sectional Area (in^2)	Load (lbs)	Load (N)	Stress (psf)	Stress (kPa)
0.1880	0.478	4.94	3.08	16	71	747	36
0.1900	0.483	5.00	3.09	16	71	747	36
0.1922	0.488	5.06	3.09	16	71	746	36
0.1943	0.494	5.11	3.09	16	71	746	36
0.1964	0.499	5.17	3.09	16	71	745	36
0.1987	0.505	5.23	3.09	18	80	838	40
0.2007	0.510	5.28	3.09	16	71	744	36
0.2027	0.515	5.33	3.10	18	80	837	40
0.2048	0.520	5.39	3.10	18	80	837	40
0.2069	0.526	5.44	3.10	18	80	836	40
0.2091	0.531	5.50	3.10	18	80	836	40
0.2112	0.536	5.55	3.10	18	80	835	40
0.2134	0.542	5.61	3.11	18	80	835	40
0.2155	0.547	5.67	3.11	18	80	834	40
0.2176	0.553	5.72	3.11	18	80	834	40
0.2196	0.558	5.78	3.11	18	80	833	40
0.2219	0.564	5.84	3.11	18	80	833	40
0.2239	0.569	5.89	3.12	18	80	832	40
0.2259	0.574	5.94	3.12	18	80	832	40
0.2280	0.579	6.00	3.12	18	80	831	40
0.2300	0.584	6.05	3.12	18	80	831	40
0.2320	0.589	6.10	3.12	18	80	830	40
0.2341	0.595	6.16	3.12	18	80	830	40
0.2363	0.600	6.22	3.13	18	80	829	40
0.2385	0.606	6.27	3.13	18	80	829	40
0.2407	0.611	6.33	3.13	18	80	828	40
0.2427	0.616	6.38	3.13	18	80	828	40
0.2449	0.622	6.44	3.13	18	80	827	40
0.2472	0.628	6.50	3.14	18	80	827	40
0.2495	0.634	6.56	3.14	18	80	826	40
0.2517	0.639	6.62	3.14	18	80	826	40
0.2540	0.645	6.68	3.14	18	80	825	40
0.2563	0.651	6.74	3.14	18	80	825	39
0.2586	0.657	6.80	3.15	18	80	824	39
0.2608	0.662	6.86	3.15	18	80	824	39
0.2628	0.668	6.91	3.15	18	80	823	39
0.2651	0.673	6.97	3.15	18	80	823	39
0.2673	0.679	7.03	3.15	18	80	822	39
0.2695	0.685	7.09	3.16	18	80	821	39
0.2715	0.690	7.14	3.16	18	80	821	39
0.2736	0.695	7.20	3.16	18	80	821	39
0.2759	0.701	7.26	3.16	20	89	911	44
0.2781	0.706	7.31	3.16	18	80	819	39

Unconfined Compressive Strength
ASTM D2166

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Displacement (in)	Displacement (cm)	Strain (%)	Average Cross Sectional Area (in²)	Load (lbs)	Load (N)	Stress (psf)	Stress (kPa)
0.2804	0.712	7.38	3.17	20	89	910	44
0.2825	0.718	7.43	3.17	20	89	909	44
0.2847	0.723	7.49	3.17	20	89	909	44
0.2867	0.728	7.54	3.17	20	89	908	43
0.2888	0.734	7.60	3.17	20	89	908	43
0.2911	0.739	7.66	3.17	20	89	907	43
0.2932	0.745	7.71	3.18	20	89	907	43
0.2955	0.751	7.77	3.18	20	89	906	43
0.2978	0.756	7.83	3.18	20	89	905	43
0.2999	0.762	7.89	3.18	20	89	905	43
0.3019	0.767	7.94	3.18	20	89	904	43
0.3040	0.772	8.00	3.19	20	89	904	43
0.3062	0.778	8.05	3.19	20	89	903	43
0.3084	0.783	8.11	3.19	20	89	903	43
0.3106	0.789	8.17	3.19	20	89	902	43
0.3127	0.794	8.22	3.19	20	89	902	43
0.3149	0.800	8.28	3.20	20	89	901	43
0.3171	0.805	8.34	3.20	20	89	900	43
0.3192	0.811	8.40	3.20	20	89	900	43
0.3212	0.816	8.45	3.20	20	89	899	43
0.3235	0.822	8.51	3.20	20	89	899	43
0.3256	0.827	8.56	3.21	20	89	898	43
0.3277	0.832	8.62	3.21	20	89	898	43
0.3300	0.838	8.68	3.21	20	89	897	43
0.3321	0.844	8.73	3.21	20	89	897	43
0.3343	0.849	8.79	3.21	20	89	896	43
0.3364	0.854	8.85	3.22	20	89	895	43
0.3387	0.860	8.91	3.22	20	89	895	43
0.3407	0.865	8.96	3.22	20	89	894	43
0.3427	0.870	9.01	3.22	20	89	894	43
0.3448	0.876	9.07	3.22	20	89	893	43
0.3471	0.882	9.13	3.23	20	89	893	43
0.3492	0.887	9.18	3.23	20	89	892	43
0.3514	0.893	9.24	3.23	20	89	892	43
0.3535	0.898	9.30	3.23	20	89	891	43
0.3557	0.903	9.36	3.23	20	89	890	43
0.3579	0.909	9.41	3.24	20	89	890	43
0.3600	0.914	9.47	3.24	20	89	889	43
0.3623	0.920	9.53	3.24	20	89	889	43
0.3644	0.926	9.58	3.24	20	89	888	43
0.3666	0.931	9.64	3.24	20	89	888	43
0.3687	0.936	9.70	3.25	20	89	887	42
0.3708	0.942	9.75	3.25	20	89	887	42

Unconfined Compressive Strength
ASTM D2166

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Displacement (in)	Displacement (cm)	Strain (%)	Average Cross Sectional Area (in²)	Load (lbs)	Load (N)	Stress (psf)	Stress (kPa)
0.3728	0.947	9.81	3.25	20	89	886	42
0.3748	0.952	9.86	3.25	20	89	886	42
0.3771	0.958	9.92	3.25	20	89	885	42
0.3794	0.964	9.98	3.26	20	89	884	42
0.3816	0.969	10.04	3.26	20	89	884	42
0.3838	0.975	10.09	3.26	20	89	883	42
0.3860	0.980	10.15	3.26	20	89	883	42
0.3883	0.986	10.21	3.27	20	89	882	42
0.3905	0.992	10.27	3.27	20	89	881	42
0.3928	0.998	10.33	3.27	20	89	881	42
0.3951	1.004	10.39	3.27	20	89	880	42
0.3973	1.009	10.45	3.27	20	89	880	42
0.3996	1.015	10.51	3.28	20	89	879	42
0.4019	1.021	10.57	3.28	20	89	879	42
0.4040	1.026	10.63	3.28	20	89	878	42
0.4063	1.032	10.69	3.28	20	89	877	42
0.4084	1.037	10.74	3.28	20	89	877	42
0.4104	1.042	10.79	3.29	20	89	876	42
0.4126	1.048	10.85	3.29	20	89	876	42
0.4148	1.054	10.91	3.29	20	89	875	42
0.4168	1.059	10.96	3.29	20	89	875	42
0.4190	1.064	11.02	3.29	20	89	874	42
0.4212	1.070	11.08	3.30	20	89	874	42
0.4235	1.076	11.14	3.30	20	89	873	42
0.4255	1.081	11.19	3.30	20	89	872	42
0.4275	1.086	11.24	3.30	20	89	872	42
0.4297	1.091	11.30	3.31	20	89	871	42
0.4320	1.097	11.36	3.31	22	98	958	46
0.4343	1.103	11.42	3.31	20	89	870	42
0.4366	1.109	11.48	3.31	22	98	957	46
0.4388	1.115	11.54	3.31	22	98	956	46
0.4411	1.120	11.60	3.32	20	89	868	42
0.4435	1.126	11.66	3.32	22	98	955	46
0.4457	1.132	11.72	3.32	22	98	954	46
0.4479	1.138	11.78	3.32	20	89	867	41
0.4502	1.144	11.84	3.33	22	98	953	46
0.4524	1.149	11.90	3.33	22	98	952	46
0.4547	1.155	11.96	3.33	22	98	951	46
0.4568	1.160	12.01	3.33	22	98	951	46
0.4591	1.166	12.08	3.33	22	98	950	45
0.4614	1.172	12.14	3.34	22	98	949	45
0.4636	1.178	12.19	3.34	22	98	949	45
0.4659	1.183	12.25	3.34	22	98	948	45

Unconfined Compressive Strength
ASTM D2166

CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Displacement (in)	Displacement (cm)	Strain (%)	Average Cross Sectional Area (in²)	Load (lbs)	Load (N)	Stress (psf)	Stress (kPa)
0.4680	1.189	12.31	3.34	22	98	948	45
0.4703	1.195	12.37	3.35	22	98	947	45
0.4724	1.200	12.43	3.35	22	98	946	45
0.4744	1.205	12.48	3.35	22	98	946	45
0.4766	1.211	12.54	3.35	22	98	945	45
0.4789	1.216	12.60	3.35	22	98	945	45
0.4811	1.222	12.65	3.36	22	98	944	45
0.4834	1.228	12.71	3.36	22	98	943	45
0.4856	1.233	12.77	3.36	22	98	943	45
0.4879	1.239	12.83	3.36	22	98	942	45
0.4900	1.245	12.89	3.37	22	98	941	45
0.4923	1.250	12.95	3.37	22	98	941	45
0.4944	1.256	13.00	3.37	22	98	940	45
0.4964	1.261	13.06	3.37	22	98	940	45
0.4986	1.266	13.11	3.37	22	98	939	45
0.5008	1.272	13.17	3.38	22	98	938	45
0.5030	1.278	13.23	3.38	22	98	938	45
0.5067	1.287	13.33	3.38	22	98	937	45
0.5090	1.293	13.39	3.38	22	98	936	45
0.5112	1.298	13.45	3.39	22	98	935	45
0.5135	1.304	13.51	3.39	22	98	935	45
0.5184	1.317	13.63	3.39	22	98	933	45
0.5207	1.323	13.70	3.40	22	98	933	45
0.5228	1.328	13.75	3.40	22	98	932	45
0.5251	1.334	13.81	3.40	22	98	931	45
0.5273	1.339	13.87	3.40	22	98	931	45
0.5295	1.345	13.93	3.41	22	98	930	45
0.5315	1.350	13.98	3.41	22	98	930	45
0.5336	1.355	14.03	3.41	22	98	929	44
0.5358	1.361	14.09	3.41	22	98	928	44
0.5380	1.367	14.15	3.41	20	89	843	40
0.5403	1.372	14.21	3.42	22	98	927	44
0.5424	1.378	14.27	3.42	22	98	926	44
0.5447	1.384	14.33	3.42	22	98	926	44
0.5468	1.389	14.38	3.42	22	98	925	44
0.5491	1.395	14.44	3.43	22	98	925	44
0.5512	1.400	14.50	3.43	22	98	924	44
0.5533	1.405	14.55	3.43	22	98	923	44
0.5555	1.411	14.61	3.43	22	98	923	44
0.5578	1.417	14.67	3.44	22	98	922	44
0.5600	1.422	14.73	3.44	22	98	921	44
0.5623	1.428	14.79	3.44	22	98	921	44
0.5644	1.434	14.84	3.44	22	98	920	44

Unconfined Compressive Strength
ASTM D2166

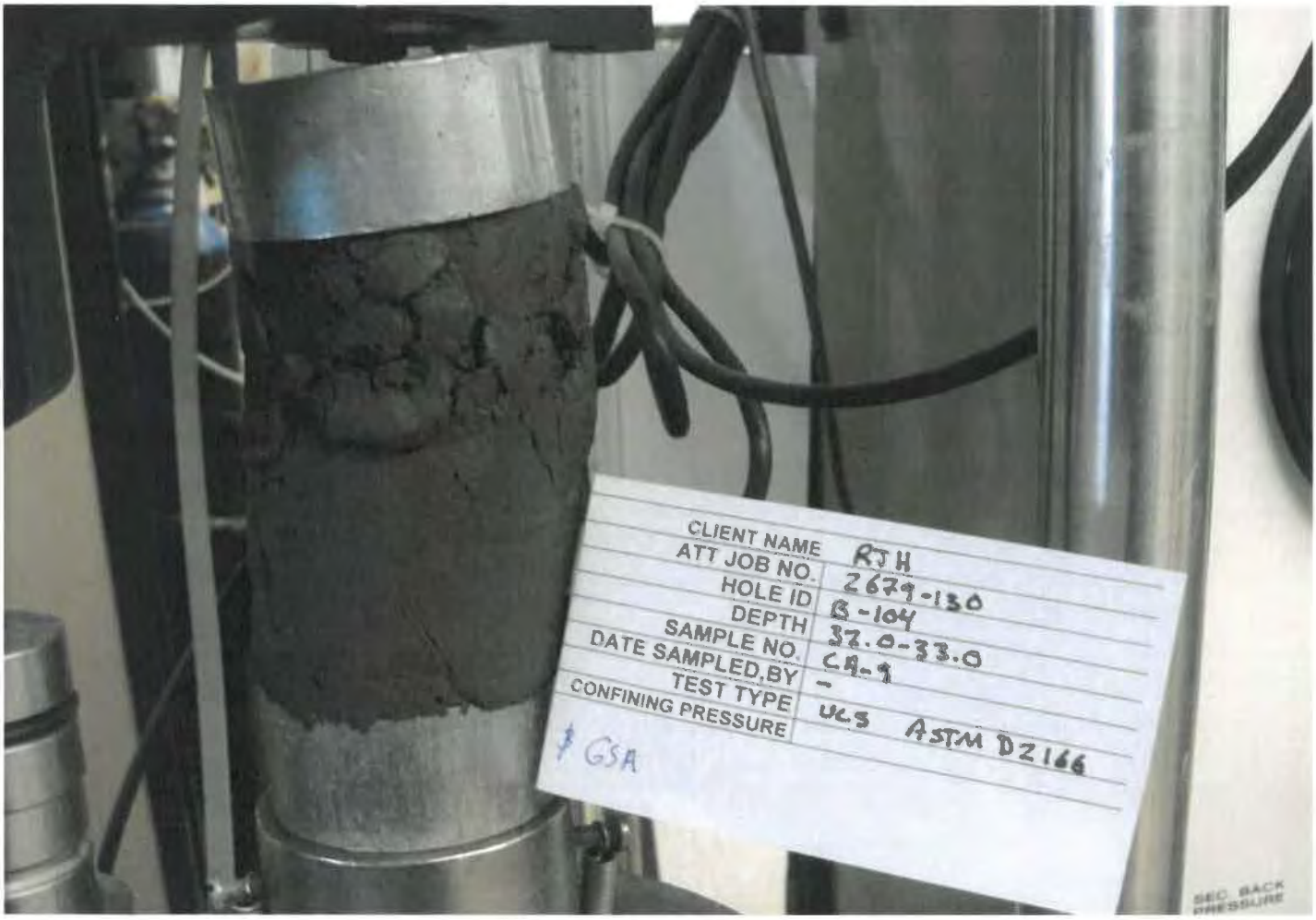
CLIENT	RJH Consultants	BORING NO.	B-104
JOB NO.	2679-130	DEPTH	32-33.0'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-9
PROJECT NO.	18115	DATE SAMPLED	9/20/2018
LOCATION	--	DESCRIPTION	lean clay w/gravel
DATE TESTED	10/17/18		
TECHNICIAN	SPH/CAL		

Displacement (in)	Displacement (cm)	Strain (%)	Average Cross Sectional Area (in²)	Load (lbs)	Load (N)	Stress (psf)	Stress (kPa)
0.5666	1.439	14.90	3.44	20	89	836	40
0.5687	1.444	14.96	3.45	22	98	919	44
0.5709	1.450	15.02	3.45	22	98	918	44
0.5732	1.456	15.08	3.45	20	89	834	40
0.5753	1.461	15.13	3.45	20	89	834	40
0.5775	1.467	15.19	3.46	22	98	916	44
0.5797	1.472	15.25	3.46	22	98	916	44

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--



NOTES

File name: 2679130__Image_18_10_18_07_47_43

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--

NOTES

File name: 2679130__Image_18_10_18_07_48_14

Corrosion Suite
ASTM C1580, D4972, D1411, G187

ADVANCED TERRA TESTING

Corrosion Suite
ASTM C1580 D4972 D1411 G187

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	45-50'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-15
PROJECT NO.	18115	DATE SAMPLED	07/26/18
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/29/18		
TECHNICIAN	SKS		

Measured Sulfate Concentration (ppm):	23
Dilution:	1.00:1
Sulfate Concentration (ppm):	23
Measured Chloride Concentration (ppm):	7.7
Dilution:	5.00:1
Chloride Concentration (ppm):	38.5
pH:	6.8
Minimum Measured Resistivity (Ω):	1200
Box Correction Factor (cm):	2.00
Minimum Resistivity (Ω·cm):	2400
Measured Sulfide Concentration (ppm):	0.104
Dilution:	1.00:1
Sulfide Concentration (ppm):	0.104

NOTES

Data entry by: SKS Date: 11/7/2018

Checked by: WAT Date: 11/7/18

File name: 2679130__Corrosion Test ASTM C1580 D4972 D1411 G187_2.xlsm

ADVANCED TERRA TESTING

Corrosion Suite
ASTM C1580 D4972 D1411 G187

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	50.6-51.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	S-19
PROJECT NO.	18115	DATE SAMPLED	07/26/18
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/27/18		
TECHNICIAN	SKS		

Measured Sulfate Concentration (ppm):	11
Dilution:	10.00:1
Sulfate Concentration (ppm):	107
Measured Chloride Concentration (ppm):	14.4
Dilution:	10.00:1
Chloride Concentration (ppm):	144.0
pH:	7.3
Minimum Measured Resistivity (Ω):	1300
Box Correction Factor (cm):	2.00
Minimum Resistivity (Ω·cm):	2600
Measured Sulfide Concentration (ppm):	0.092
Dilution:	2.00:1
Sulfide Concentration (ppm):	0.184

NOTES

Data entry by: SKS Date: 11/7/2018
Checked by: MME Date: 11/7/18
File name: 2679130__Corrosion Test ASTM C1580 D4972 D1411 G187_0.xlsm

ADVANCED TERRA TESTING

Corrosion Suite
ASTM C1580 D4972 D1411 G187

CLIENT	RJH Consultants	BORING NO.	B-102A
JOB NO.	2679-130	DEPTH	44-45'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	CA-20
PROJECT NO.	18115	DATE SAMPLED	07/24/18
LOCATION	--	DESCRIPTION	--
DATE TESTED	10/28/18		
TECHNICIAN	SKS		

Measured Sulfate Concentration (ppm):	5
Dilution:	1.00:1
Sulfate Concentration (ppm):	5
Measured Chloride Concentration (ppm):	10.8
Dilution:	10.00:1
Chloride Concentration (ppm):	108.0
pH:	7.4
Minimum Measured Resistivity (Ω):	770
Box Correction Factor (cm):	2.00
Minimum Resistivity (Ω·cm):	1540
Measured Sulfide Concentration (ppm):	0.01
Dilution:	1.00:1
Sulfide Concentration (ppm):	0.01

NOTES

Data entry by: SKS Date: 11/7/2018
Checked by: WAL Date: 11/7/18
File name: 2679130__Corrosion Test ASTM C1580 D4972 D1411 G187_1.xlsm

Pinhole Dispersion Test
ASTM D4647, Method A

Identification and Classification of Dispersive Clay Soils by the Pinhole Test
ASTM D 4647 Method A

Client: RJH Consultants
Job No.: 2679-130
Project: Hogchute Dam Safety Evaluation
Location: --
Project No.: 18115

Boring No.: B-101
Depth: 30-31'
Sample No.: CA-11
Date Tested: 11/6/2018 By: BDF

Before Test Moisture Content

Wet Weight Soil & Dish: 29.84g
Dry Weight Soil & Dish: 25.99g
Weight Of Water: 3.86g
Dish Weight: 3.14g
Dry Weight Soil: 22.84g
Moisture Content: 16.89%

Classification: ND1
Original Pinhole Diameter: 0.04in, 0.0010m
Final Pinhole Diameter: 0.04in, 0.0010m
Sample Type: Remolded

Before Test Density

Height: 1.50in, 0.0381m
Diameter: 1.94in, 0.0492m
Weight: 0.3297lbs, 149.57g
Wet Density: 128.78lbs/ft³, 2063kg/m³
Dry Density: 110.17lbs/ft³, 1765kg/m³

Test Data

Two Inch Head Height	Flow	Elapsed Time	Flow Rate	Turbidity	Observations
	30ml	60s	0.008gal/min, 0.5ml/s	Clear	
	29ml	60s	0.008gal/min, 0.5ml/s	Clear	
	28ml	60s	0.007gal/min, 0.5ml/s	Clear	
	28ml	60s	0.007gal/min, 0.5ml/s	Clear	
	27ml	60s	0.007gal/min, 0.5ml/s	Clear	
	27ml	60s	0.007gal/min, 0.5ml/s	Clear	
	27ml	60s	0.007gal/min, 0.5ml/s	Clear	
	27ml	60s	0.007gal/min, 0.5ml/s	Clear	
	25ml	60s	0.007gal/min, 0.4ml/s	Clear	
	26ml	60s	0.007gal/min, 0.4ml/s	Clear	
Seven Inch Head Height	**Flow**	**Elapsed Time**	**Flow Rate**	**Turbidity**	**Observations**
	48ml	60s	0.013gal/min, 0.8ml/s	Barely Visible	Particles at bottom
	48ml	60s	0.013gal/min, 0.8ml/s	Clear	
	47ml	60s	0.012gal/min, 0.8ml/s	Clear	
	47ml	60s	0.012gal/min, 0.8ml/s	Clear	
	46ml	60s	0.012gal/min, 0.8ml/s	Clear	
Fifteen Inch Head Height	**Flow**	**Elapsed Time**	**Flow Rate**	**Turbidity**	**Observations**
	79ml	60s	0.021gal/min, 1.3ml/s	Barely Visible	
	79ml	60s	0.021gal/min, 1.3ml/s	Clear	
	78ml	60s	0.021gal/min, 1.3ml/s	Clear	
	78ml	60s	0.021gal/min, 1.3ml/s	Clear	
	78ml	60s	0.021gal/min, 1.3ml/s	Clear	
Forty Inch Head Height	**Flow**	**Elapsed Time**	**Flow Rate**	**Turbidity**	**Observations**
	69ml	30s	0.036gal/min, 2.3ml/s	Barely Visible	
	70ml	30s	0.037gal/min, 2.3ml/s	Clear	
	72ml	30s	0.038gal/min, 2.4ml/s	Clear	
	70ml	30s	0.037gal/min, 2.3ml/s	Clear	Particles at bottom
	70ml	30s	0.037gal/min, 2.3ml/s	Clear	

File Name: 2679_130_pinhole-ASTMD-4647-R3_0.xls
Entered By: SPH Date: 11/7/2018

Data Checked By: KMS
Date: 11/8/18

ATT
ADVANCED TERRA TESTING

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	B-101
JOB NO.	2679-130	30-31'
PROJECT	Hogchute Dam Safety Evaluation	CA-11
PROJECT NO.	18115	
LOCATION	--	AFTER TEST

NOTES

File name: 2679130__Image_18_11_07_16_16_39

Identification and Classification of Dispersive Clay Soils by the Pinhole Test
ASTM D 4647 Method A

Client: RJH Consultants
Job No.: 2679-130
Project: Hogchute Dam Safety Evaluation
Location: --
Project No.: 18115

Boring No.: B-103
Depth: 15-16'
Sample No.: CA-8
Date Tested: 11/6/2018 By: BDF

Before Test Moisture Content

Wet Weight Soil & Dish: 39.71g
Dry Weight Soil & Dish: 35.33g
Weight Of Water: 4.38g
Dish Weight: 3.13g
Dry Weight Soil: 32.20g
Moisture Content: 13.60%

Classification: ND1
Original Pinhole Diameter: 0.04in, 0.0010m
Final Pinhole Diameter: 0.04in, 0.0010m
Sample Type: Remolded

Before Test Density

Height: 1.50in, 0.0381m
Diameter: 1.94in, 0.0492m
Weight: 0.3068lbs, 139.16g
Wet Density: 119.81lbs/ft³, 1919kg/m³
Dry Density: 105.47lbs/ft³, 1689kg/m³

Test Data

Two Inch Head Height	Flow	Elapsed Time	Flow Rate	Turbidity	Observations
	10ml	30s	0.005gal/min, 0.3ml/s	Clear	Particles on bottom
	8ml	30s	0.004gal/min, 0.3ml/s	Clear	Particles on bottom
	8ml	30s	0.004gal/min, 0.3ml/s	Clear	Particles on bottom
	9ml	30s	0.005gal/min, 0.3ml/s	Clear	
	9ml	30s	0.005gal/min, 0.3ml/s	Clear	
	8ml	30s	0.004gal/min, 0.3ml/s	Clear	
	12ml	30s	0.006gal/min, 0.4ml/s	Clear	
	7ml	30s	0.004gal/min, 0.2ml/s	Clear	
	8ml	30s	0.004gal/min, 0.3ml/s	Clear	
	9ml	30s	0.005gal/min, 0.3ml/s	Clear	
Seven Inch Head Height	**Flow**	**Elapsed Time**	**Flow Rate**	**Turbidity**	**Observations**
	42ml	60s	0.011gal/min, 0.7ml/s	Barely Visible	Particles on bottom
	41ml	60s	0.011gal/min, 0.7ml/s	Barely Visible	
	42ml	60s	0.011gal/min, 0.7ml/s	Clear	
	42ml	60s	0.011gal/min, 0.7ml/s	Clear	
	40ml	60s	0.011gal/min, 0.7ml/s	Clear	
Fifteen Inch Head Height	**Flow**	**Elapsed Time**	**Flow Rate**	**Turbidity**	**Observations**
	78ml	60s	0.021gal/min, 1.3ml/s	Barely Visible	Particles on bottom
	89ml	60s	0.024gal/min, 1.5ml/s	Clear	Particles on bottom
	90ml	60s	0.024gal/min, 1.5ml/s	Clear	
	88ml	60s	0.023gal/min, 1.5ml/s	Clear	
	87ml	60s	0.023gal/min, 1.5ml/s	Clear	
Forty Inch Head Height	**Flow**	**Elapsed Time**	**Flow Rate**	**Turbidity**	**Observations**
	74ml	30s	0.039gal/min, 2.5ml/s	Barely Visible	Particles on bottom
	75ml	30s	0.040gal/min, 2.5ml/s	Clear	Particles on bottom
	74ml	30s	0.039gal/min, 2.5ml/s	Clear	
	73ml	30s	0.039gal/min, 2.4ml/s	Clear	
	74ml	30s	0.039gal/min, 2.5ml/s	Clear	

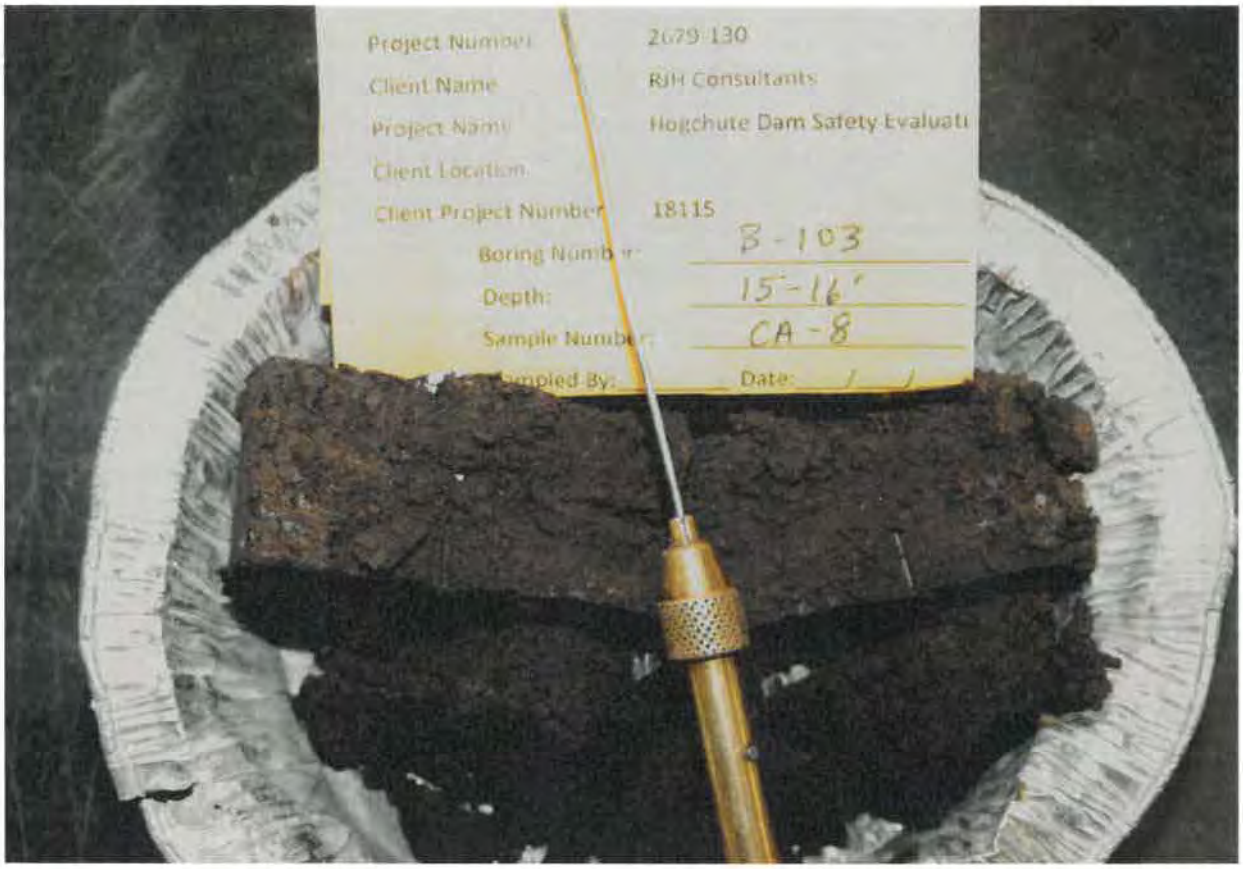
File Name: 2679_130_pinhole-ASTMD-4647-R3_1.xls
Entered By: SPH Date: 11/7/2018

Data Checked By: KMS
Date: 11/8/18

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	B-103
JOB NO.	2679-130	15-16'
PROJECT	Hogchute Dam Safety Evaluation	CA-8
PROJECT NO.	18115	
LOCATION	--	AFTER TEST



NOTES

File name: 2679130__Image_18_11_07_16_19_51

Other - Sample Photographs

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--

NOTES

File name: 2679130__Image_18_10_18_07_42_26

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--

NOTES

File name: 2679130__Image_18_10_18_07_43_12

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--

NOTES

File name: 2679130__Image_18_10_18_07_44_01

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--

NOTES	

File name: 2679130__Image_18_10_18_07_44_41

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--

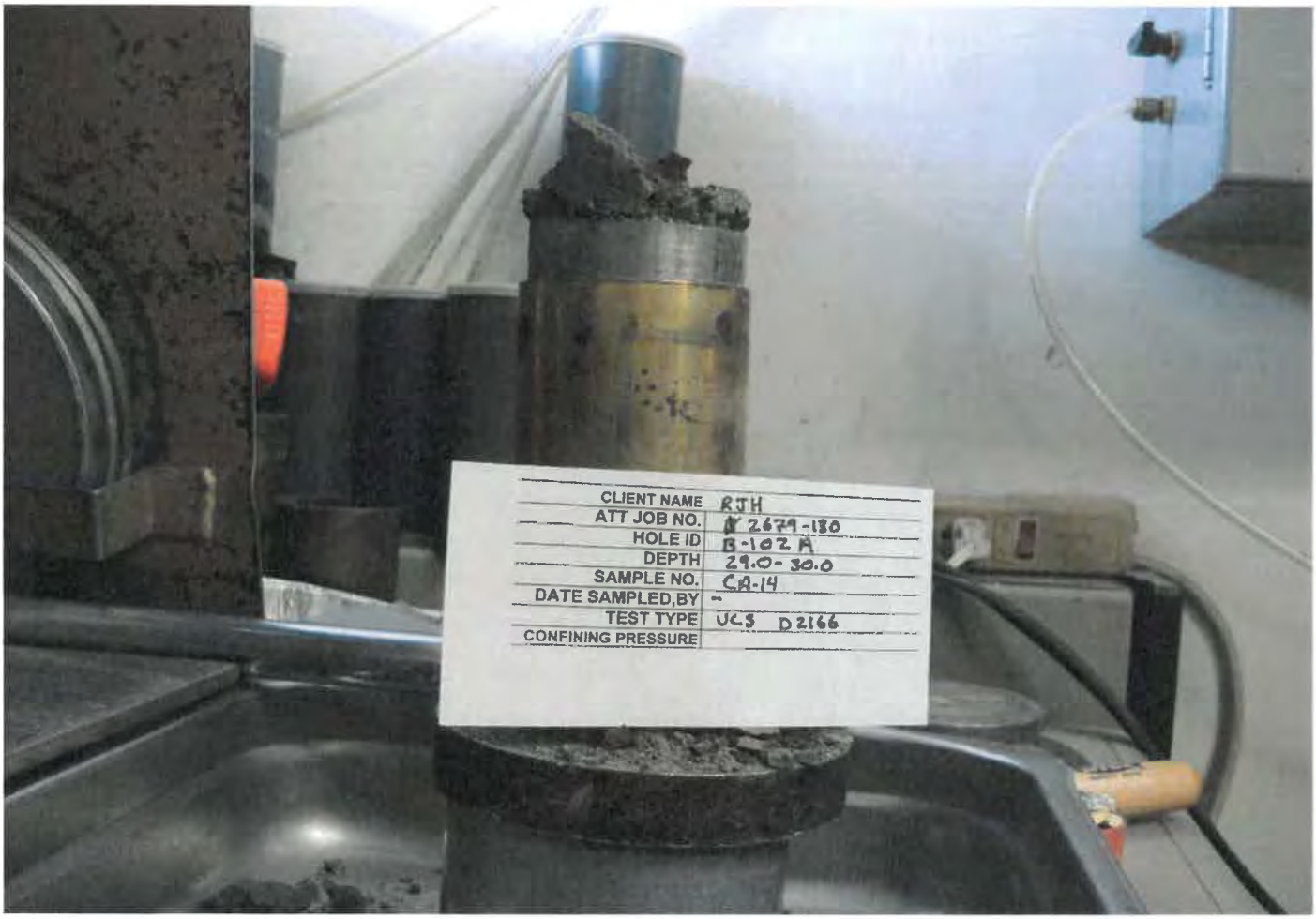
NOTES

File name: 2679130__Image_18_10_18_07_45_19

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--



NOTES

File name: 2679130__Image_18_10_18_07_46_14

Image Attachment

ADVANCED TERRA TESTING

CLIENT	RJH Consultants	--
JOB NO.	2679-130	--
PROJECT	Hogchute Dam Safety Evaluation	--
PROJECT NO.	18115	--
LOCATION	--	--

NOTES

File name: 2679130__Image_18_10_18_07_46_52

APPENDIX E

SEEPAGE INVESTIGATION DAILY FIELD REPORTS

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 001-GOJ**
Task 1002 Seepage Investigation **Date: Wednesday, August 8, 2018**

Prepared By: Garrett Jackson (GOJ)
Weather: low 80's, mostly sunny, light-moderate breeze

People on Site (arrival/departure time)

- RJH:
  - GOJ (09:45/18:00)
- Sorter:
  - Bill Ogle (10:25/17:45)
- Slade Connell, Ron Key from City of Grand Junction (~11:30-12:00) observing from dam crest

Equipment on Site

Mobile
- GOJ Chevy Silverado
- Sorter Ford F250

At Dam Site Overnight
- Cat 320N backhoe

Excavation Progress Summary

- Sorter mobilized to the site, unloaded the backhoe from the low-boy at the intersection of the Hogchute road and Land's End Road, walked the hoe down to the dam.
- The backhoe was used to move rocks on the right side of the downstream dam slope and over the outlet conduit to construct a working pad from which to reach the left (south) side of the conduit. The backhoe moved rocks from this area to expose the native soil under the riprap to a level below the top of the outlet conduit encasement. No wetness was encountered in the area left of the outlet conduit.
- The backhoe moved slightly and excavated rock to expose the top of the concrete-encased outlet conduit and the area right (north) of the conduit. Water has historically been observed right of the outlet conduit, pooled under the riprap behind (upstream of) the remains of the original outlet control structure at the downstream end of the conduit. Water was encountered during excavation along the right of the outlet about 12"-16" below the top of the encasement. Water appeared to be entering the excavation from along the conduit, but a specific location could not be identified. The backhoe continued removing rock along the conduit to follow the ponded water back under the riprap to about 25' upstream of the concrete structure headwall.
- The backhoe repositioned to the top of the outlet channel bank above the right end of the headwall to finish cleaning the excavation. Sorter used the backhoe and a hand

shovel to clear the excavation bottom and drain standing water around the right end of the headwall. To drain the water from the deeper portions of the excavation, a trench was excavated around the right end of the headwall, which bypassed the existing seepage measuring sump used for monitoring the flow. The sump and old drain pipe are now dry, and water from the excavated area drains under the outlet channel riprap into the channel downstream of the headwall.

Seepage Observations

- At the beginning of the day, water was standing in the riprap behind the headwall and draining to the sump as usual. No flow measurement was taken, but visually the discharge appeared to be about 4-5 gpm, and was clear.
- Minor wetness was observed in a backhoe track on the native ground above the riprap on the right side of the dam toe. Wetness persisted through the day but dried slightly in the sun and breeze.
- At the end of the day, after the excavation had drained, very minor seepage was observed coming from under the thin layer of riprap at the north corner of the upstream end of the excavation. It is unknown at this time if this is a remnant of ponded water draining to the excavation, or if this is potentially groundwater discharging from the excavated natural hillside at the toe of the dam.
- Water was observed seeping from under the headwall slab when the rock and soil were excavated along the right side of the conduit. Discharge was from the gravel beneath the slab, was somewhat episodic, and was clear. Maximum discharge was estimated to be about 1 gpm.
- Minor wetness (no flow) was observed high on the slope in the dense willows above the two historic seepage channels through the willows.

Plan for Next Work Day

- GOJ will call USFS and USACE first thing in the morning to discuss the work to date and invite them to observe conditions at the site.
- GOJ and Sorter to meet at the site tomorrow at 8:30 am.
- GOJ and Sorter will inspect the excavation to identify any sources of seepage from the outlet conduit, the dam toe, or the natural hillside.
- The outlet conduit concrete encasement will be cleaned with hand shovels to expose any seepage discharge location(s).
- All seepage locations will be documented with photos and measurements.
- SEO will be onsite to observe the excavation and seepage conditions.

Site Coordination Activities

- Did not have a gate key or combination, so GOJ cut the chain on the gate to the reservoir. Slade Connell (City) placed a padlock on the gate later in the morning, combination is 1564. Gate is locked.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project
Task 1002 Seepage Investigation

Report No.: 001-GOJ
Date: Wednesday, August 8, 2018

Photographs

Downstream toe of slope above the remains of the old outlet control structure and headwall prior to excavation for seepage investigation.

Standing water has been observed for many years under the rocks in the area behind the headwall and along the right side of the conduit.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project
Task 1002 Seepage Investigation

Report No.: 001-GOJ
Date: Wednesday, August 8, 2018

Excavating the right (north) side of the outlet conduit, where water has been observed pooled behind the headwall for many years.

Note the left side of the conduit is dry.

Top of concrete-encased outlet conduit.

Water pooled right of the conduit behind the headwall.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project
Task 1002 Seepage Investigation

Report No.: 001-GOJ
Date: Wednesday, August 8, 2018

Pooled seepage water draining around right end of headwall.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project
Task 1002 Seepage Investigation

Report No.: 001-GOJ
Date: Wednesday, August 8, 2018

Backhoe cleaning channel for draining excavation along right side of conduit.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project **Report No.: 002-GOJ**
Task 1002 Seepage Investigation **Date: Wednesday, August 9, 2018**

Prepared By: Garrett Jackson (GOJ)
Weather: mid 80's, sunny, light-moderate breeze

People on Site (arrival/departure time)

- RJH:
  - GOJ (08:00-14:45)
- Sorter:
  - Bill Ogle (08:50-14:45)
- Jackie Blumberg (JB) and Jason Ward (JW) (SEO) (~08:50-13:30)

Equipment on Site

Mobile
- GOJ Chevy Silverado
- Sorter Ford F250
- SEO Chevy Blazer and Ford F150

At Dam Site Overnight
- All equipment demobilized from site

Excavation Progress Summary

- After draining overnight, the excavated area right of the outlet conduit was essentially dry, except for water standing in several puddles. The seepage entering the excavation from under the rocks at the upstream end is a steady but barely-noticeable trickle. The only measurable water entering the excavation came from under the old control structure foundation slab at the downstream end of the conduit. The seepage flow was visually estimated to be about 4-5 gpm and was clear.
- After clearing the top of the concrete encasement under the slab, water was observed bubbling from an old broken approximately 3/4" inch diameter pipe in the top of the encasement. Based on a review of the 1947 design drawings, this pipe is believed to be the remnant of the reservoir level measurement line extending from the upstream end of the conduit to the old control structure at the downstream end. The line is shown as passing along the conduit within the concrete encasement. The pipe is located inside the old structure, about18" downstream of the upstream structure wall. An earlier patch is visible in the floor of the structure remains over the broken pipe, and the file mentions a previous repair to the reservoir level pressure line.
- With the excavation drained, I observed that the outlet conduit is founded on clayey soils that are very similar to the materials encountered in the piezometer borings drilled on the crest of the dam.

- Sorter backfilled the investigation excavation with the rock removed yesterday. Generally, and as feasible, Sorter placed smaller (< ~12") rocks in the bottom of the excavation at the concrete structure where the flowing water enters the excavation. Larger rocks were placed above these smaller rocks, taking care to not drop large rocks on top of the outlet conduit.
- Since the bottom of the new trench draining the excavation is lower than the existing drain pipe and sump, the drain was reconstructed by laying the old 4" PVC pipe in the bottom of the new trench. The old plastic sump was relocated to a new hole excavated in the riprap on the right side of the outlet channel downstream of the headwall, and the 4" pipe was anchored to drain into the relocated sump.

SEO Discussion

- GOJ, JB, and JW discussed the seepage investigation. All agreed that the major source of the water historically pooled behind the headwall has been identified, and this leakage from the broken pipe is not an immediate threat to the dam's safety. The broken reservoir level measurement pipe will be repaired or properly abandoned during rehabilitation of the outlet works.
- Other evidences of seepage (in the upstream corner of the excavation and in the dense willows at the top of the hill right of the outlet channel) will need to be evaluated and addressed, but there is no evidence at this time indicating this seepage is a dam safety concern.
- The City is considering not draining the reservoir for the outlet works assessment task of this dam safety evaluation. They will likely skip the assessment and proceed directly to design of the outlet works rehabilitation, which will include eliminating the twin 20-inch lines and extending the 30-inch conduit upstream to a new intake structure. In this case, there would be no need to drain the reservoir until it is required for construction. JB agreed that, if this is the City's plan, there will be no need to inspect the existing conduit this year. The post-construction inspection will restart the 10-year inspection cycle.

Seepage Observations

- Minor wetness (no flow) was observed in the two historic seepage channels through the willows.
- The shallow wetness exposed yesterday by the backhoe track on top of the hill above the excavation is dry.
- The ground in the dense willows at the top of the hill above the two historic seepage channels is very wet where it has been disturbed by the backhoe.

Plan for Next Work Day

Seepage investigation is complete.

DAILY SITE REPORT

Site Coordination Activities

- Sorter tracked the backhoe back up to the parking lot at about 12:45. Bill Ogle drove out to the observatory and called for the truck to come pick up the hoe. GOJ drove Bill back down to the parking lot, and Bill started driving the hoe up to Land's End Road for loading.
- 14:15 GOJ departed site.

DAILY SITE REPORT

18115 Hogchute Dam Safety Evaluation Project
Task 1002 Seepage Investigation

Report No.: 002-GOJ
Date: Wednesday, August 9, 2018

Photographs

	Seepage emerging from under the old control structure foundation slab is the only measurable water entering the excavation.
	Water is bubbling from a broken pipe in the top of the outlet conduit encasement. The pipe is believed to extend to the upstream end of the conduit, passing along the conduit within the concrete encasement.

	Water from the broken pipe emerges from under the slab and drains to the right side of the conduit, where it is collected in the excavation and routed around the right end of the headwall for measurement.
	Backhoe replacing excavated rock in the investigation excavation.

Appendix D
Drawings and Specifications

CONSTRUCTION DRAWINGS FOR

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION

DAM ID NO. 420127 CONSTRUCTION NO. C-0454A

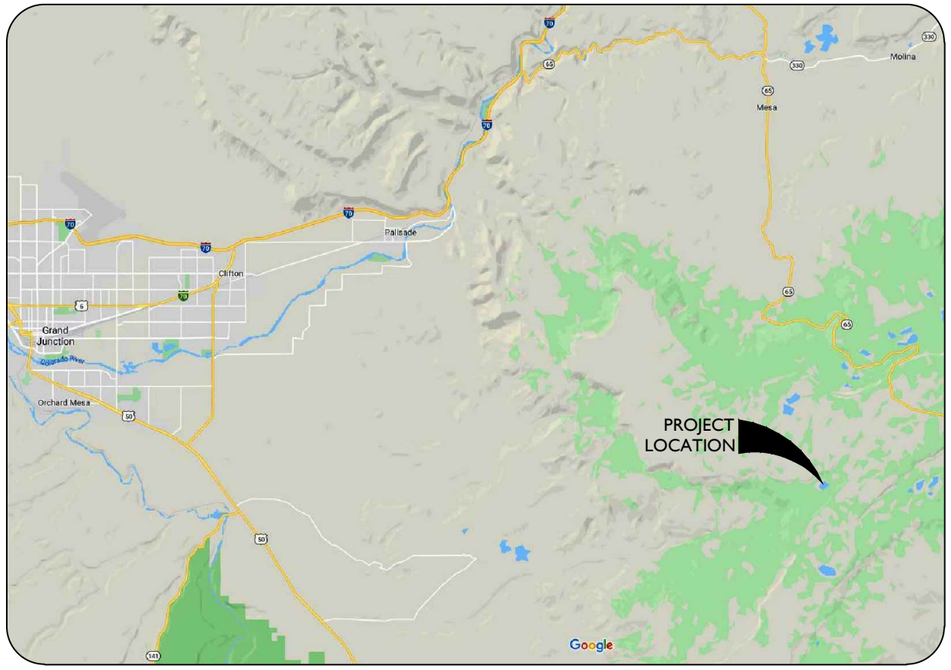
PREPARED FOR THE

CITY OF GRAND JUNCTION, MESA COUNTY, COLORADO
WATER DIVISION 4, WATER DISTRICT 42

FEBRUARY 2021

VICINITY MAP

NOT TO SCALE



CONTACT INFORMATION

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Eau Claire, Wisconsin 54701-7698
(715) 834-3161

Ayres Associates
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3665 JFK Parkway, Building 2, Suite 100
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(970) 223-5556

COLORADO DIVISION OF WATER RESOURCES
Department of Natural Resources
District 5 Dam Safety Engineer
Jackie Blumberg, PE
2768 Compass Drive, #102
Grand Junction, Colorado 81506
(303) 505-6469

UNITED STATES FOREST SERVICE
Grand Valley Ranger District
Loren Paulson, Realty Specialist
2777 Crossroads Blvd.
Grand Junction, Colorado 81506
(970) 242-8211 x4112

THESE PLANS HAVE BEEN PREPARED BY ME OR UNDER MY DIRECT SUPERVISION.

TODD M. RUDOLPH
COLORADO P.E. NO. 0058436

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER 2/26/2021

APPROVED ON THE _____ DAY OF _____ 2021

STATE ENGINEER

BILL McCORMICK, CHIEF, COLORADO DAM SAFETY
COLORADO P.E. NO. 0029127

THESE PLANS REPRESENT THE AS-CONSTRUCTED CONDITIONS OF HOGCHUTE DAM REPAIRS TO THE BEST OF MY KNOWLEDGE AND JUDGMENT, BASED IN PART ON INFORMATION FURNISHED BY OTHERS, AS OF THE _____ DAY OF _____ 2021

TODD M. RUDOLPH
COLORADO P.E. NO. 0058436

F:\AAC2017CUI\GenBlocks\Colorado 811 Logo.JPG

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

TITLE SHEET

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr: CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
G01

SHEET 1 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 COVER.DWG Friday, February 26, 2021 6:54 PM By: WAYNE, ROBERT

SHEET INDEX

SHEET	DRAWING	DESCRIPTION
1	G01	TITLE SHEET
2	G02	INDEX TO DRAWINGS, SURVEY NOTES, AND GENERAL NOTES
3	G03	SITE ACCESS AND BORROW AREAS PLAN
4	G04	EXISTING SITE, STAGING, AND STOCKPILE AREAS PLAN
5	G05	GEOTECHNICAL INFORMATION
6	G06	RATING CURVES
7	C01	EXISTING CONDITIONS AND DEMOLITION PLAN - 1 OF 2
8	C02	EXISTING CONDITIONS AND DEMOLITION PLAN - 2 OF 2
9	C03	DEWATERING PLAN - PHASE 1
10	C04	DEWATERING PLAN - PHASE 2
11	C05	EROSION CONTROL PLAN
12	C06	EROSION CONTROL DETAILS
13	C07	EROSION CONTROL DETAILS
14	C08	EROSION CONTROL DETAILS
15	C09	EROSION CONTROL DETAILS
16	C10	EROSION CONTROL DETAILS
17	C11	DAM PROPOSED CONDITIONS AND COFFERDAM
18	C12	DAM PROPOSED CONDITIONS ENLARGED PLAN
19	C13	DAM PROFILE AND DETAIL
20	C14	DAM DRAIN PIPE PROFILES
21	C15	DAM DETAILS 1 OF 2
22	C16	DAM DETAILS 2 OF 2
23	C17	DAM SECTIONS 1 OF 3
24	C18	DAM SECTIONS 2 OF 3
25	C19	DAM SECTIONS 3 OF 3
26	C20	SPILLWAY PLAN 1 OF 3
27	C21	SPILLWAY PLAN 2 OF 3
28	C22	SPILLWAY PLAN 3 OF 3
29	C23	SPILLWAY ROCK BERM PROFILES 1 OF 2
30	C24	SPILLWAY ROCK BERM PROFILES 2 OF 2
31	C25	SPILLWAY DETAILS AND SECTIONS
32	C26	SPILLWAY OVERFLOW WIER PLAN AND DETAILS
33	C27	SPILLWAY PRIMARY AND SECONDARY CHANNEL PLAN AND WATTLE DETAIL
34	C28	SPILLWAY PLANTING PLAN
35	C29	DAM INSTRUMENTATION
36	C30	INSTRUMENTATION DETAILS
37	C31	WETLAND IMPACT
38	C32	CONCRETE BUTTRESS REINFORCEMENT
39	C33	STEEL PIPE DETAILS
40	C34	TRASH RACK DETAILS
41	C35	TRASH RACK DETAILS
42	C36	IMPACT BASIN DETAILS
43	C37	MISCELLANEOUS DETAILS

GENERAL NOTES:

1. THESE CONSTRUCTION DRAWINGS ARE SUPPLEMENTED WITH THE TECHNICAL SPECIFICATIONS FOR HOGCHUTE DAM REHABILITATION. AN ENGINEER SEALED COPY OF BOTH DOCUMENTS MUST BE MAINTAINED AT THE JOBSITE DURING ALL PHASES OF CONSTRUCTION.
2. THIS PROJECT FALLS UNDER THE COLORADO STATE ENGINEER'S OFFICE RULE 5 FOR DAM SAFETY RULES AND REGULATIONS FOR JURISDICTIONAL EMBANKMENT DAMS. THE CONTRACTOR, BY STARTING THIS PROJECT, ACKNOWLEDGES THAT THEY ARE FULLY AWARE OF THE CURRENTLY ADOPTED RULES AND REGULATIONS AND HOW THEY PERTAIN TO THEIR CONSTRUCTION PRACTICES AND SCHEDULING.
3. NO CHANGES TO THE DRAWINGS OR SPECIFICATIONS CAN BE MADE WITHOUT PRIOR APPROVAL FROM THE COLORADO STATE ENGINEER'S OFFICE.
4. ALL CONSTRUCTION CHANGE ORDERS MUST BE APPROVED BY THE STATE ENGINEER'S OFFICE OF COLORADO AS WELL AS THE PROJECT ENGINEER. THIS WILL NECESSITATE A LONGER PERIOD FOR REVIEW AND APPROVALS.

SURVEY NOTES:

BENCHMARK:

THE HORIZONTAL AND VERTICAL CONTROL SHOWN IS BASED ON UNADJUSTED GPS STATIC OBSERVATIONS TIED TO THE GRAND MESA AREA (GMA) GEOID ESTABLISHED BY MESA COUNTY. THE NORTHERLY CONTROL POINT (POINT 2) IS A PK NAIL AND WASHER EMBEDDED IN ROCK AND IS THE BENCHMARK FOR THIS PROJECT. THE SOUTH POINT (POINT 3) IS A ROUND HEAD BOLT EMBEDDED IN ROCK.

SURVEY CONTROL POINT TABLE

POINT NUMBER	DESCRIPTION	ELEVATION	NORTHING	EASTING
1	CP /PK-W/	9902.462	15722.80	45752.48
2	CP /PK-W/	9903.882	16393.64	45673.48
3	CP /BOLT/	9905.415	15477.14	45781.19
10	CP /BOLT/	9901.937	16157.18	45763.76
11	CP /BOLT/	9902.010	15831.42	45734.61
12	CP /BOLT/	9902.091	15606.47	45772.61

SEE DRAWING G04 FOR LOCATION OF SURVEY CONTROL POINTS.

LEGEND:

- LIMITS OF DISTURBANCE
- BORROW, STOCKPILE, OR STAGING AREA
- EXISTING WETLANDS
- APPROXIMATE LIMITS OF EXISTING SPILLWAY
- 9870 EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- PRIMARY FLOW PATH
- SECONDARY FLOW PATH
- EXISTING CHANNEL OR STREAM
- SURVEY CONTROL POINT
- GEOTECHNICAL BORE LOCATION (ORIGINAL 1947 DESIGN)
- GEOTECHNICAL BORE LOCATION (2018)
- TEST PIT
- EMBANKMENT FILL
- EXISTING COBBLE/ROCK SHELL
- RIPRAP
- TYPE 1 FILTER MATERIAL
- TYPE 2 FILTER MATERIAL
- EXISTING GROUND
- EXISTING CHANNEL FILL
- EXISTING CHANNEL CUT
- APPROXIMATE LIMITS OF CLEARING AND GRUBBING

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER 2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

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INDEX TO DRAWINGS, SURVEY NOTES, AND GENERAL NOTES

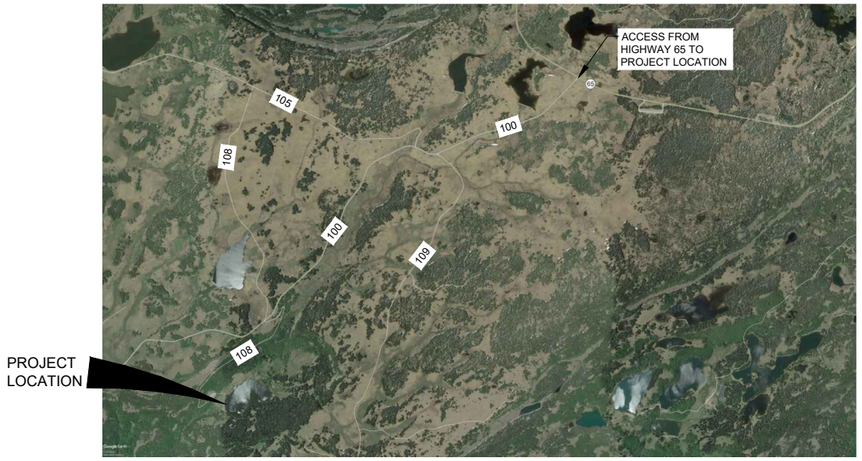
HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Date | Revisions

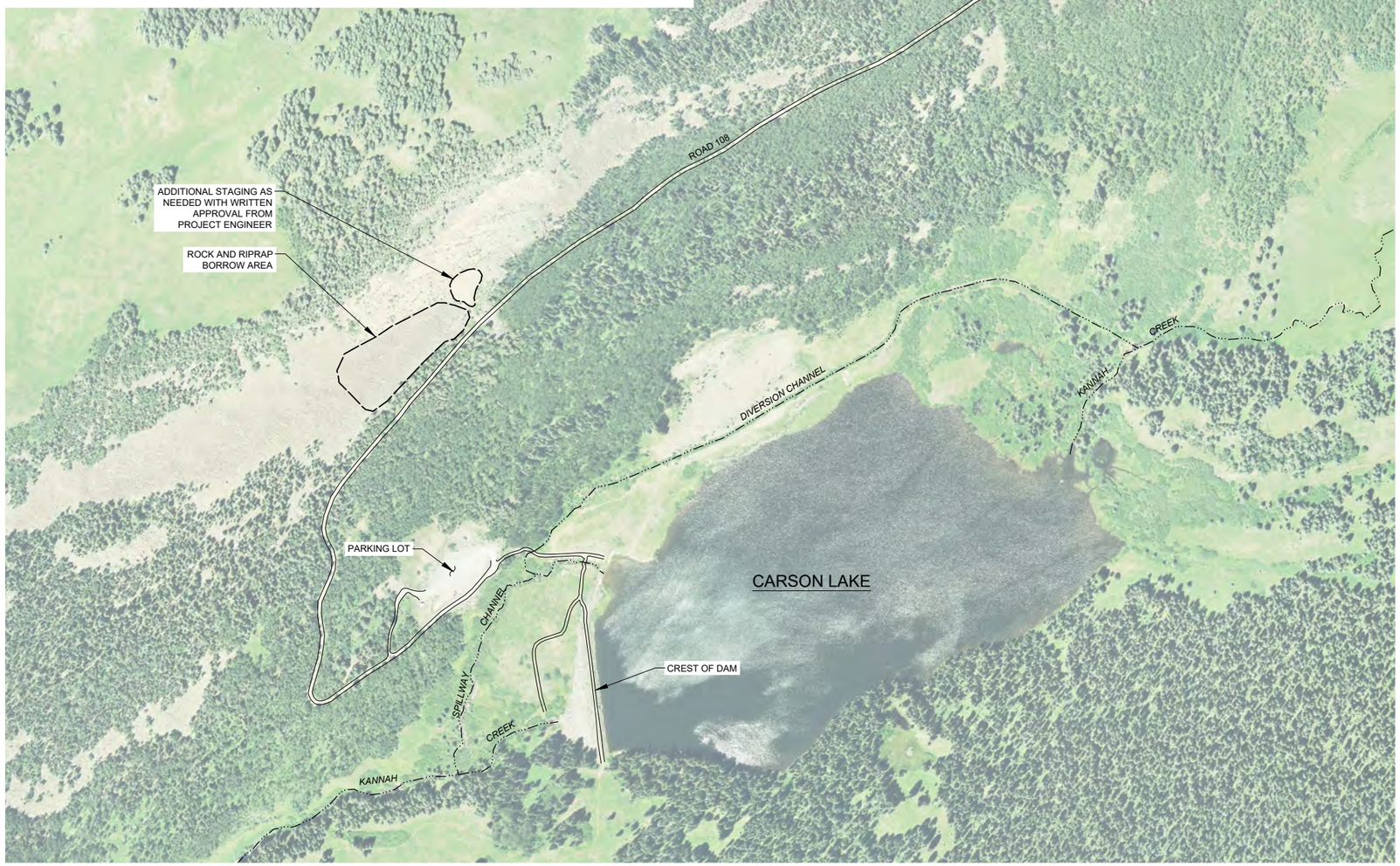
Project Mgr.: CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

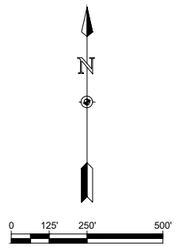
PROJECT NO. 26-1144.00
DRAWING NO. G02
SHEET 2 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 GENERAL NOTES.DWG Friday, February 26, 2021 6:54 PM By: WAYNE, ROBERT



INSET MAP
NOT TO SCALE





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SITE ACCESS AND BORROW AREAS PLAN

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr. CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
G03

SHEET 3 OF 43

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TODD M. RUDQUIST
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PROFESSIONAL ENGINEER
2/26/2021

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Drawing Name: i:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SITE ACCESS.DWG Friday, February 26, 2021 6:37 PM By: WAYNE, ROBERT

N

0 25' 50' 100'

SCALE IN FEET

SUGGESTED WORK FLOW SEQUENCE:

CONTRACTOR MAY MODIFY SEQUENCE TO COMPLETE PROJECT

1. THE RESERVOIR IS CURRENTLY DRAINED AND WILL REMAIN DRAINED TO THE EXTENT FEASIBLE BY THE OWNER DURING CONSTRUCTION.
2. DURING CONSTRUCTION OF THE OUTLET WORKS, UP TO 50 CFS OF FLOW CAN BE DIVERTED AROUND THE RESERVOIR THROUGH THE DIVERSION CHANNEL. THE OWNER WILL OPERATE THE UPSTREAM STRUCTURE TO DIVERT FLOW.
3. WHEN CONSTRUCTION OF THE OUTLET WORKS IS COMPLETE, FLOW WILL BE DIVERTED TO THE OUTLET WORKS TO ALLOW WORK ON THE AUXILIARY SPILLWAY.
   - IF THE EMBANKMENT WORK IS COMPLETE, INCLUDING PLACEMENT OF THE ROCK/COBBLE SHELL, THE REFILLING OF THE RESERVOIR CAN BEGIN.
   - IF EMBANKMENT WORK IS NOT COMPLETE, FLOW CAN BE DIVERTED FROM THE DIVERSION CHANNEL, BUT FILLING OF THE RESERVOIR CANNOT BE INITIATED.
4. CONSTRUCTION OF AUXILIARY SPILLWAY IMPROVEMENTS.

NOTE:

CONTRACTOR TO PROVIDE PORTABLE FACILITIES FOR DURATION OF PROJECT. USE OF EXISTING VAULT SHALL NOT BE PERMISSIBLE.

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EXISTING SITE, STAGING, AND STOCKPILE AREAS PLAN

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

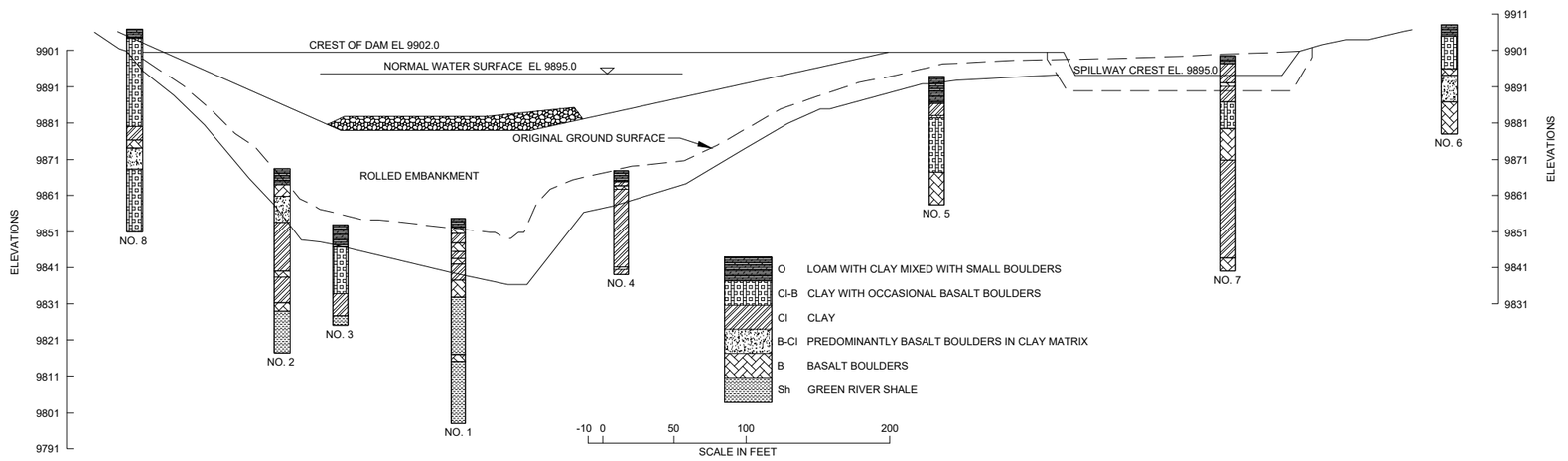
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

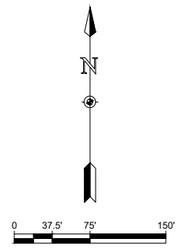
DRAWING NO.
G04

SHEET 4 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 STAGING AND BORROW AREAS.DWG Friday, February 26, 2021 6:37 PM By: WAYNE, ROBERT



PROFILE ON C OF CUTOFF TRENCH AND SPILLWAY - LOG OF DRILL HOLES



NOTES:

1. TEST PITS 1 THROUGH 3 INSTALLED AND LOGGED UNDER DIRECTION OF HUDDLESTON-BERRY ENGINEERING AND TESTING LLC. SEE DESIGN REPORT FOR BORING LOGS.
2. SOIL BORINGS B-101(P) THROUGH B-103(P) INSTALLED AND LOGGED UNDER DIRECTION OF RJH CONSULTANTS INC. SEE DESIGN REPORT FOR BORING LOGS.
3. SOIL BORING IMAGE TAKEN FROM 1947 PLAN SET 'IMPROVEMENTS TO WATER SUPPLY SYSTEM, HOGCHUTE RESERVOIR' PRODUCED BY R.J. TIPTON AND ASSOCIATES, INC., ENGINEERS.

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GEOTECHNICAL INFORMATION

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr.	CTG
Designed By:	AJS
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Approved By:	TMR
Date:	02/26/21

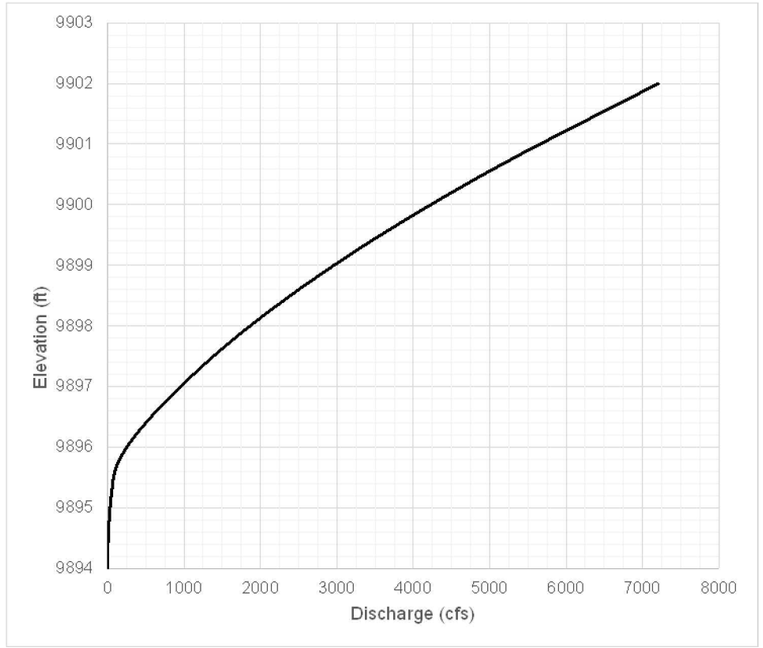
PROJECT NO.
26-1144.00

DRAWING NO.
G05

SHEET 5 OF 43

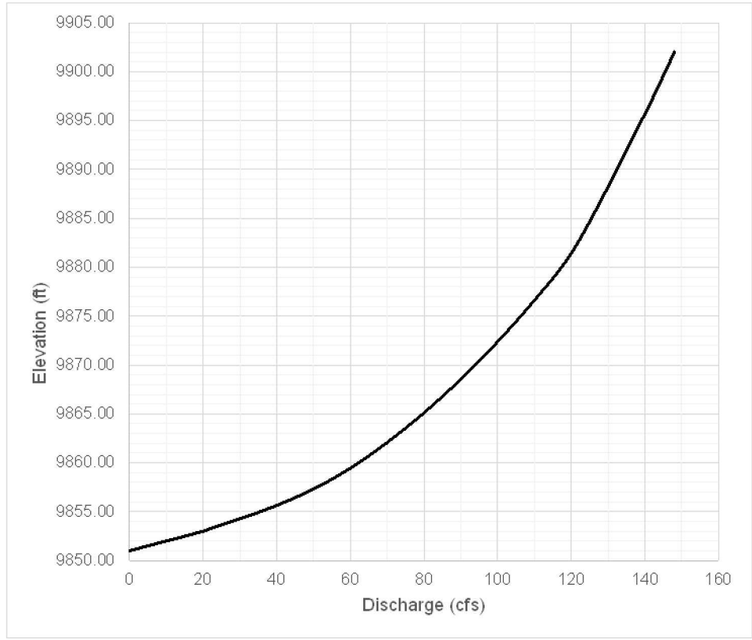
Drawing Name: i:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 GEOTECHNICAL INFORMATION.DWG Friday, February 26, 2021 6:38 PM By: WAYNE, ROBERT

Elevation (ft)	Discharge (cfs)
9851.00	0
9894.00	0
9895.00	31
9896.00	250
9897.50	1382
9899.00	2960
9900.50	4916
9902.00	7207



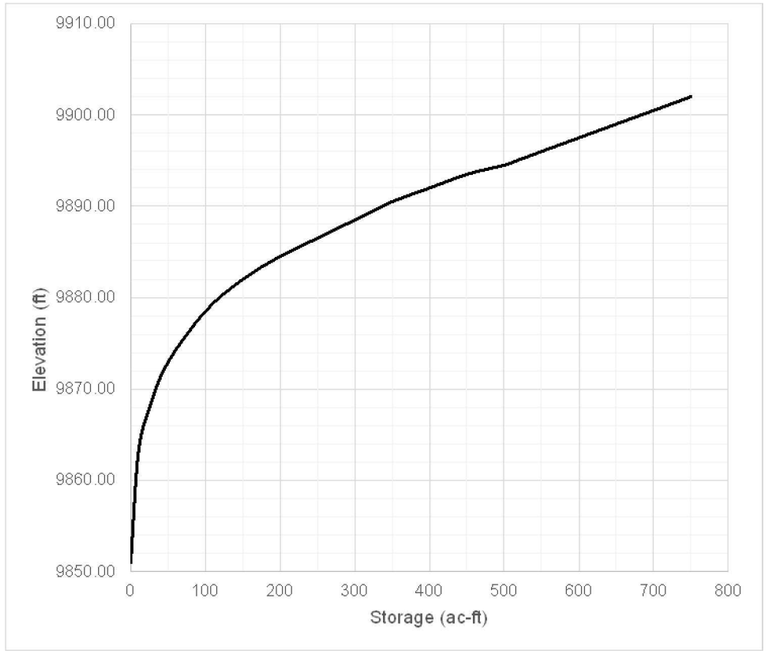
SPILLWAY RATING CURVE

Elevation (ft)	Discharge (cfs)
9851.00	0
9852.00	10
9853.00	20
9854.27	30
9855.63	40
9857.31	50
9859.44	60
9862.07	70
9865.10	80
9868.54	90
9872.38	100
9876.62	110
9881.41	120
9888.23	130
9895.00	139
9895.60	140
9900.40	146
9902.00	148



OUTLET WORKS RATING CURVE

Elevation (ft)	Storage (ac-ft)
9851.00	0
9863.00	10
9868.00	25
9873.00	50
9878.50	100
9882.00	150
9884.50	200
9886.50	250
9888.50	300
9890.50	350
9892.00	400
9893.50	450
9894.50	500
9895.00	517
9896.00	550
9897.50	600
9899.00	650
9900.50	700
9902.00	750



RESERVOIR STAGE-STORAGE CURVE

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PROFESSIONAL ENGINEER
2/26/2021

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RATING CURVES

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr. CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
G06

SHEET 6 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 RATING CURVES.DWG Friday, February 26, 2021 6:39 PM By: WAYNE, ROBERT

EXISTING SITE ACCESS ROAD

EXISTING DIVERSION CHANNEL

PROTECT-IN-PLACE
EXISTING FOOT BRIDGE

ROAD 106

REMOVE EXISTING GROUTED
ROCK SPILLWAY,
APPROXIMATE LIMITS SHOWN

APPROXIMATE LIMITS
OF CLEARING AND
GRUBBING (TYP)

EXISTING SPILLWAY CHANNEL

SEE DRAWING C02

CARSON LAKE

KANNAH CREEK

CREST OF DAM

9925 9920 9915 9910 9905 9900 9895 9890 9885 9880 9875 9870 9865 9860 9855 9850 9845 9840 9835

25+00 24+00 23+00 22+00 21+00

N

0 20' 40' 80'
SCALE IN FEET

ONE INCH - IF NOT,
SCALE ACCORDINGLY

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2/26/2021

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EXISTING CONDITIONS AND DEMOLITION PLAN - 1 OF 2

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

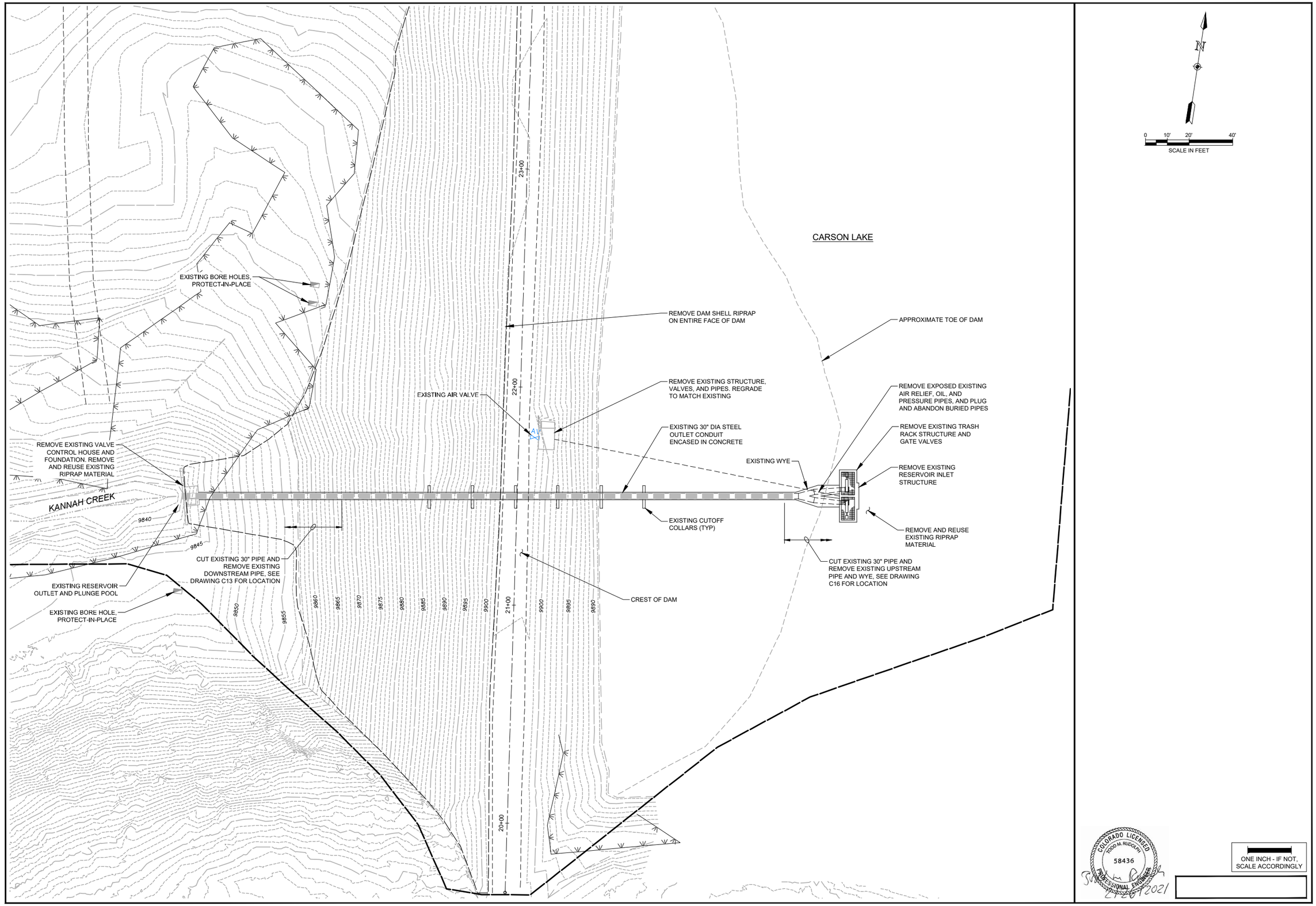
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C01

SHEET 7 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 EXISTING CONDITIONS AND DEMOLITION PLAN 2.DWG Friday, February 26, 2021 6:39 PM By: WAYNE, ROBERT



Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 EXISTING CONDITIONS AND DEMOLITION PLAN.DWG Friday, February 26, 2021 6:39 PM By: WAYNE, ROBERT

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EXISTING CONDITIONS AND DEMOLITION PLAN - 2 OF 2

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION CITY OF GRAND JUNCTION MESA COUNTY, COLORADO

Revisions	Date

Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C02

SHEET 8 OF 43

COLORADO LICENSED
TODD M. RUDOLPH
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PROFESSIONAL ENGINEER
2/26/2021

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PHASE 1 DEWATERING:
FOR CONSTRUCTION ON OUTLET WORKS AND EMBANKMENT

INFLOW FLOOD HYDROLOGY WITH STARTING POOL ELEV 9851.0'				
AE INTERVAL	PEAK INFLOW	PEAK OUTFLOW	TOTAL VOLUME	MAX POOL ELEV
(YR)	(CFS)	(CFS)	(AC-FT)	(FT)
2.0	122.0	54.0	10.4	9858.1
10.0	392.0	92.0	48.0	9869.3
100.0	1752.0	123.0	205.0	9883.2

ASSUMPTIONS: 100 PERCENT OF INFLOWS UP TO 50 CFS ARE BYPASSED AROUND LAKE. WATER RELEASED FROM LAKE DURING EVENTS WITH 30" RELIEF OUTLET AT INVERT ELEV 9851.0'.

N

0 50' 100' 200'
SCALE IN FEET

NOTES:

1. THE COFFERDAM WILL NOT PROTECT THE WORKSITE FROM THE 10-YR FLOOD. CONTRACTOR SHALL MONITOR WEATHER FORECASTS AND EVACUATE PERSONNEL AND EQUIPMENT FROM THE LAKE IF STORM ACTIVITY IS EXPECTED OR IF BYPASS CHANNEL IS APPROACHING CAPACITY.
2. PUMPING FROM LAKE TO MAINTAIN CONSTRUCTION POOL MAY BE REQUIRED IF NORMAL, NON-FLOOD FLOWS EXCEED THE BYPASS CHANNEL'S 50 CFS CAPACITY. AVERAGE MONTHLY INFLOWS PROVIDED ON SHEET C03.
3. CONTRACTOR MUST COORDINATE WITH WATER SUPPLY SUPERVISOR TO DIVERT UPSTREAM INFLOWS TO DIVERSION CHANNEL.

EXPECTED CONSTRUCTION SEQUENCING DURING PHASE 1 DEWATERING:

1. DRAW DOWN LAKE TO CONSTRUCTION POOL ELEVATION.
2. CONSTRUCT COFFERDAM WITH RELIEF OUTLET TIED INTO EXISTING 30" PIPE.
3. COMPLETE CONSTRUCTION ON OUTLET WORKS, EMBANKMENT, IMPACT BASIN, AND PRIMARY STATION INSTRUMENTATION.
4. REMOVE COFFERDAM.
5. COMMENCE PHASE 2 DEWATERING.

COLORADO LICENSED
TODD M. RUDOLPH
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PROFESSIONAL ENGINEER
2/26/2021

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DEWATERING PLAN - PHASE 1

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

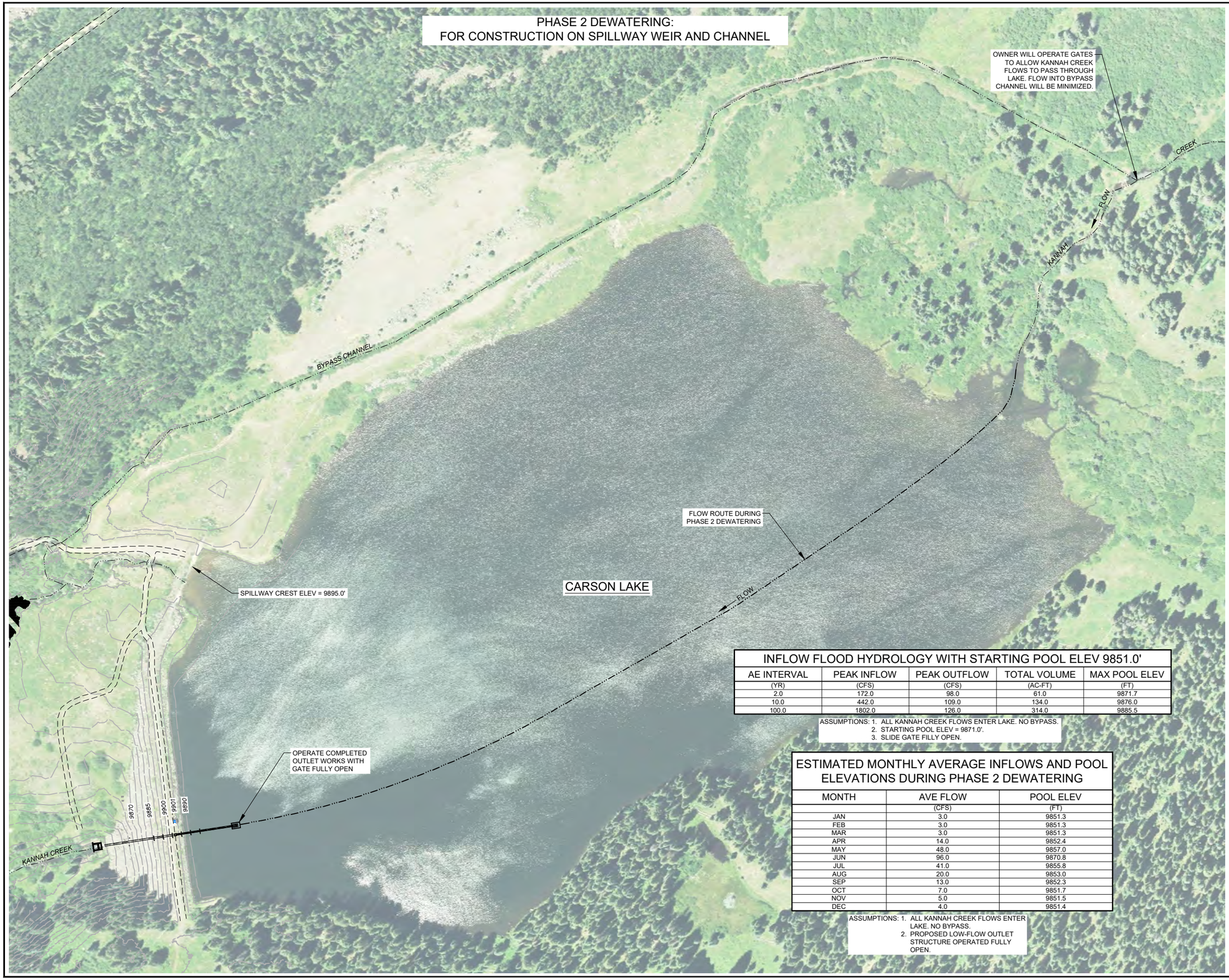
Project Mgr: CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00
DRAWING NO.
C03
SHEET 9 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DEWATERING PLAN PHASE 1.DWG Friday, February 26, 2021 6:40 PM By: WAYNE, ROBERT

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DEWATERING PLAN PHASE 2.DWG Friday, February 26, 2021 6:40 PM By: WAYNE, ROBERT

PHASE 2 DEWATERING: FOR CONSTRUCTION ON SPILLWAY WEIR AND CHANNEL



N

0 50' 100' 200'

SCALE IN FEET

EXPECTED CONSTRUCTION SEQUENCING DURING PHASE 2 DEWATERING:

1. OPERATE COMPLETED OUTLET WORKS WITH SLIDE GATE FULLY OPEN.
2. COMPLETE WORK ON SPILLWAY WEIR AND CHANNEL.

INFLOW FLOOD HYDROLOGY WITH STARTING POOL ELEV 9851.0'

AE INTERVAL	PEAK INFLOW	PEAK OUTFLOW	TOTAL VOLUME	MAX POOL ELEV
(YR)	(CFS)	(CFS)	(AC-FT)	(FT)
2.0	172.0	98.0	61.0	9871.7
10.0	442.0	109.0	134.0	9876.0
100.0	1802.0	126.0	314.0	9885.5

ASSUMPTIONS: 1. ALL KANNAH CREEK FLOWS ENTER LAKE. NO BYPASS.
2. STARTING POOL ELEV = 9871.0'.
3. SLIDE GATE FILLY OPEN.

ESTIMATED MONTHLY AVERAGE INFLOWS AND POOL ELEVATIONS DURING PHASE 2 DEWATERING

MONTH	AVE FLOW	POOL ELEV
	(CFS)	(FT)
JAN	3.0	9851.3
FEB	3.0	9851.3
MAR	3.0	9851.3
APR	14.0	9852.4
MAY	48.0	9857.0
JUN	96.0	9870.8
JUL	41.0	9855.8
AUG	20.0	9853.0
SEP	13.0	9852.3
OCT	7.0	9851.7
NOV	5.0	9851.5
DEC	4.0	9851.4

ASSUMPTIONS: 1. ALL KANNAH CREEK FLOWS ENTER LAKE. NO BYPASS.
2. PROPOSED LOW-FLOW OUTLET STRUCTURE OPERATED FULLY OPEN.

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TODD M. RUDOLPH
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PROFESSIONAL ENGINEER
2/26/2021

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DEWATERING PLAN - PHASE 2

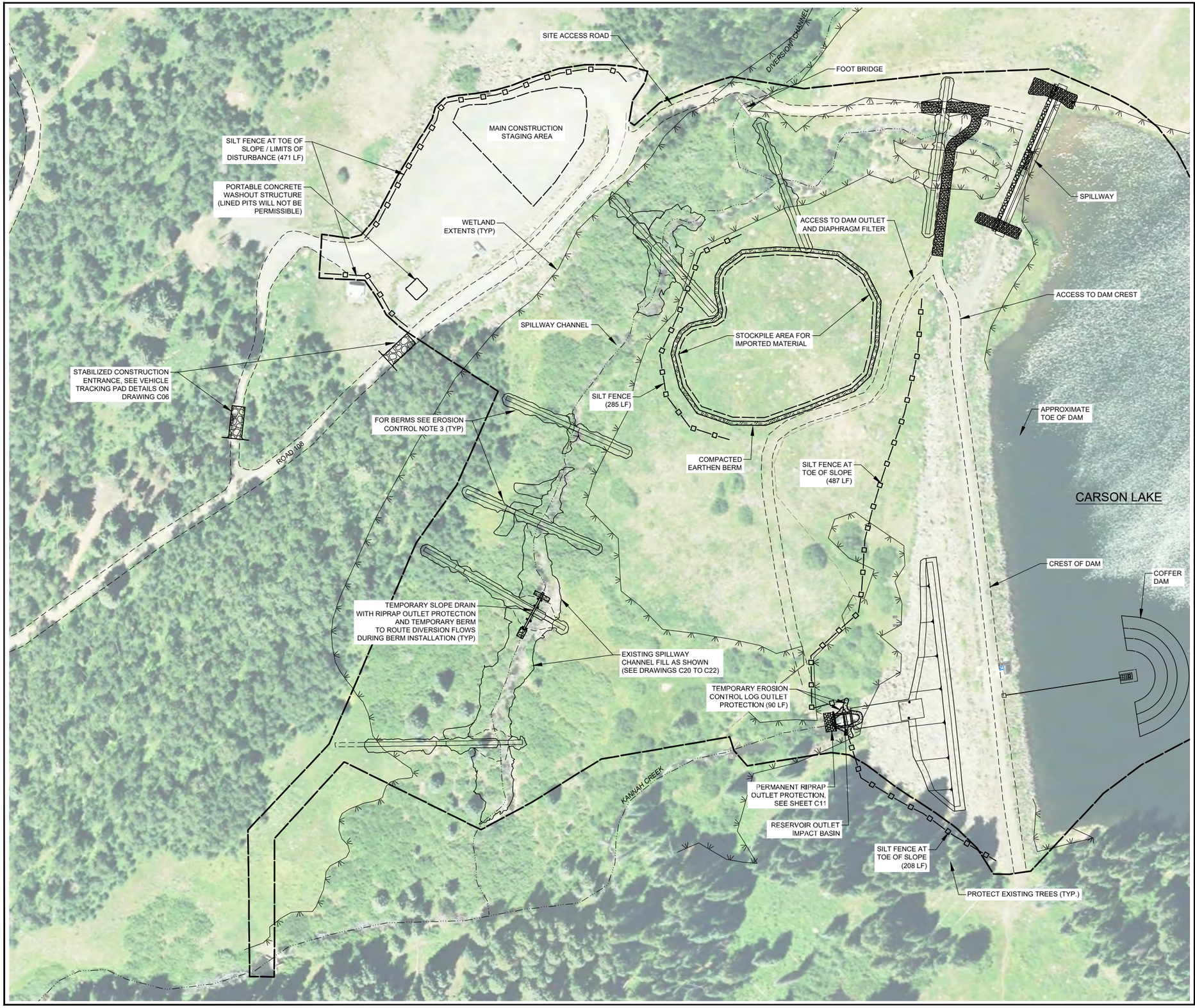
HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

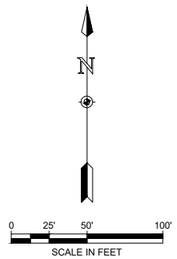
Revisions	Date

Project Mgr: CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C04





NOTES:

1. ALL TEMPORARY AND PERMANENT EROSION CONTROL SHALL COMPLY WITH THE MOST RECENT CDOT M&S STANDARDS.
2. THE CONTRACTOR SHALL STABILIZE DISTURBED AREAS BY SURFACE ROUGHENING, VERTICAL TRACKING, OR A COMBINATION THEREOF.
3. FOR EROSION CONTROL AT THE SPILLWAY CHANNEL BERMS SEE TYPICAL SECTIONS ON SHEET C25.
4. A PALLIATIVE SHALL BE USED TO CONTROL DUST PRODUCED BY CONSTRUCTION ACTIVITIES.
5. THE CONTRACTOR SHALL PROTECT EXISTING VEGETATION AND LIMIT IMPACT TO WETLAND AREAS.
6. THE CONTRACTOR SHALL CHECK EROSION CONTROL MEASURES AFTER EVERY PRECIPITATION EVENT. IF REPAIRS ARE NEEDED, REPAIRS SHALL BE PERFORMED IMMEDIATELY.
7. THE CONTRACTOR IS RESPONSIBLE TO REPAIR ANY DAMAGE TO FS ROAD 108 AT CONTRACTOR'S EXPENSE PRIOR TO ISSUANCE OF FINAL ACCEPTANCE.

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EROSION CONTROL PLAN

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

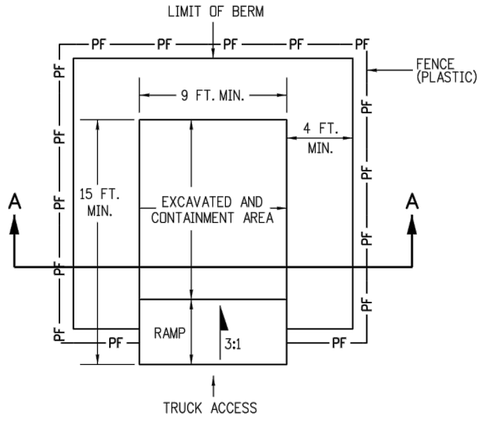
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

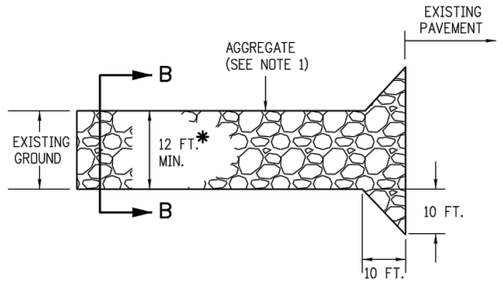
DRAWING NO.
C05

SHEET 11 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 EROSION CONTROL DETAILS.DWG Friday, February 26, 2021 6:41 PM By: WAYNE, ROBERT

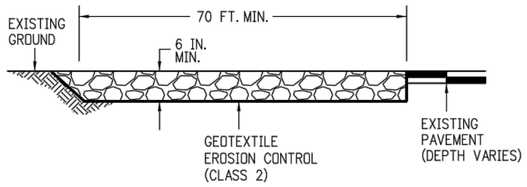


PLAN VIEW

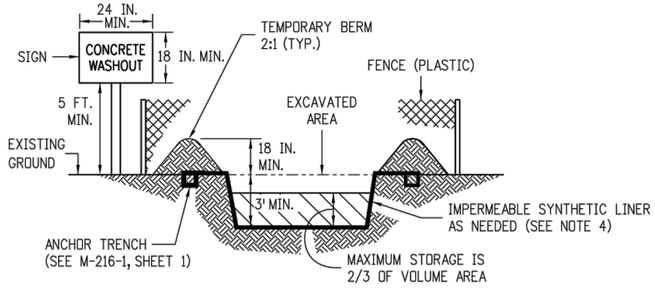


PLAN VIEW

* SHALL EXTEND FULL WIDTH OF INGRESS AND EGRESS OPERATION.



ELEVATION SECTION

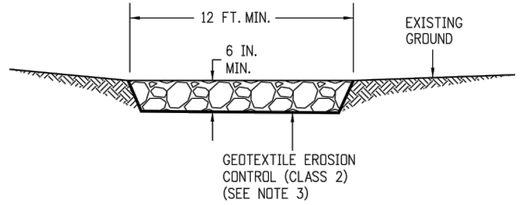


SECTION A-A

NOTES:

1. A FENCE (PLASTIC) CONFORMING TO SECTION 607 SHALL BE INSTALLED AROUND THE CONCRETE WASHOUT AREA, EXCEPT AT THE OPENING.
2. THE CONCRETE WASHOUT SIGN SHALL HAVE LETTERS AT LEAST 3 INCHES HIGH AND CONFORM TO SUBSECTION 630.02.
3. ALL MATERIALS AND LABOR TO COMPLETE THE CONCRETE WASHOUT STRUCTURE SHALL BE INCLUDED IN THE COST OF WORK AND NOT PAID FOR SEPARATELY.
4. THE BOTTOM OF EXCAVATION SHALL BE A MINIMUM OF FIVE FEET ABOVE GROUND WATER. IF NOT, THE BOTTOM OF EXCAVATION SHALL BE IN ACCORDANCE WITH 208.02 (j).
5. THE PAY ITEM NUMBER FOR CONCRETE WASHOUT STRUCTURE (EACH) IS 208-00045.

PORTABLE CONCRETE WASHOUT STRUCTURE



SECTION B-B

NOTES:

1. AGGREGATE SHALL CONFORM TO SUBSECTION 208.02 (I).
2. THE CONTRACTOR SHALL PROTECT CURB AND GUTTER THAT CROSSES THE ENTRANCE FROM DAMAGE, WHILE NOT BLOCKING FLOW OF WATER THRU STRUCTURE. PROTECTION OF THE CURB AND GUTTER SHALL BE INCLUDED IN THE COST OF WORK AND NOT PAID FOR SEPARATELY.
3. GEOTEXTILE SHALL CONFORM TO SUBSECTION 712.08.
4. ALL MATERIALS AND LABOR TO COMPLETE THE VEHICLE TRACKING PAD SHALL BE INCLUDED IN THE COST OF WORK AND NOT PAID FOR SEPARATELY.
5. THE PAY ITEM NUMBER FOR VEHICLE TRACKING PAD (EACH) IS 208-00070.

VEHICLE TRACKING PAD

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Detailer Initials: LTA	(R-X)		
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Project Development Branch JBK

TEMPORARY EROSION CONTROL

Issued by the Project Development Branch: July 31, 2019

STANDARD PLAN NO.
M-208-1
Standard Sheet No. 1 of 11
Project Sheet Number:

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER
2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
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EROSION CONTROL DETAILS

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr. CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO. 26-1144.00
DRAWING NO. C06
SHEET 12 OF 43

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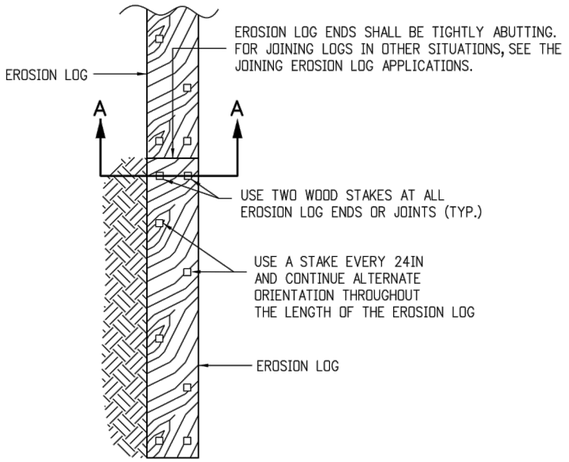
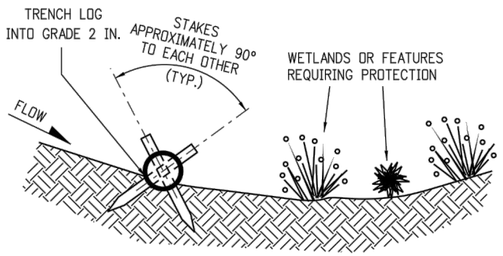


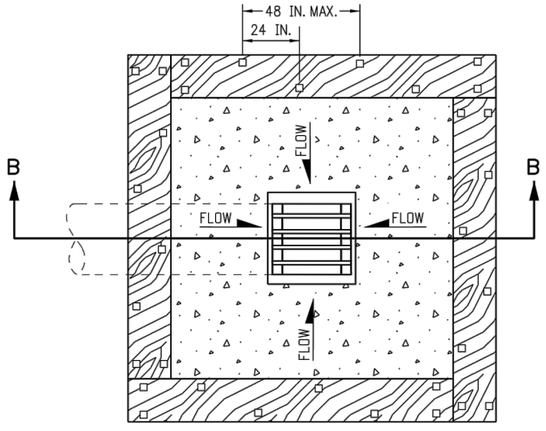
PLAN VIEW



SECTION A-A

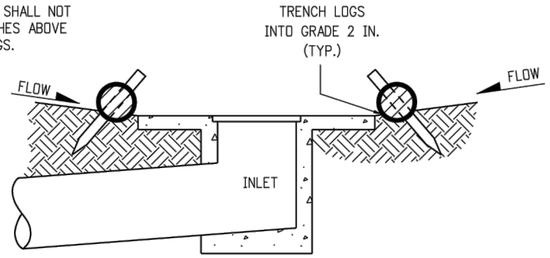
TYPICAL STAKE INSTALLATION

EROSION LOGS PAY ITEMS	
NUMBER	DESCRIPTION
208-00012	TYPE 1 (9 IN.)
208-00002	TYPE 1 (12 IN.)
208-00013	TYPE 1 (20 IN.)
208-00007	TYPE 2 (8 IN.)
208-00008	TYPE 2 (12 IN.)
208-00009	TYPE 2 (18 IN.)
208-00022	TYPE 3 (9 IN.)
208-00023	TYPE 3 (12 IN.)
208-00024	TYPE 3 (20 IN.)



PLAN VIEW

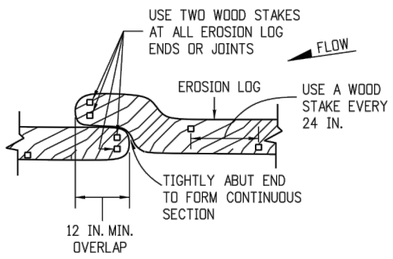
NOTE: THE TOPS OF ALL STAKES SHALL NOT EXTEND MORE THAN 2 INCHES ABOVE THE TOPS OF EROSION LOGS.



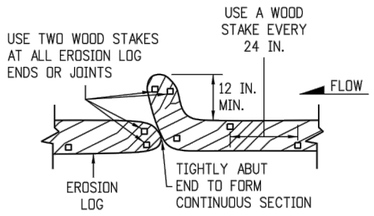
SECTION B-B

NOTE: LOCATE EROSION LOGS AT THE OUTSIDE EDGE OF THE CONCRETE APRON.

EROSION LOG FILTER AT DROP INLET

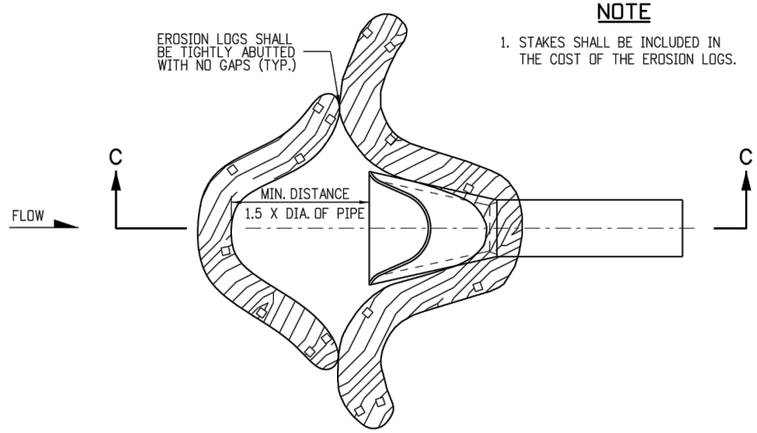


OVERLAP JOINING DETAIL



J-HOOK JOINING DETAIL

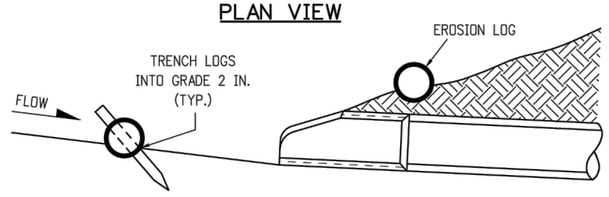
JOINING EROSION LOG APPLICATIONS



NOTE

1. STAKES SHALL BE INCLUDED IN THE COST OF THE EROSION LOGS.

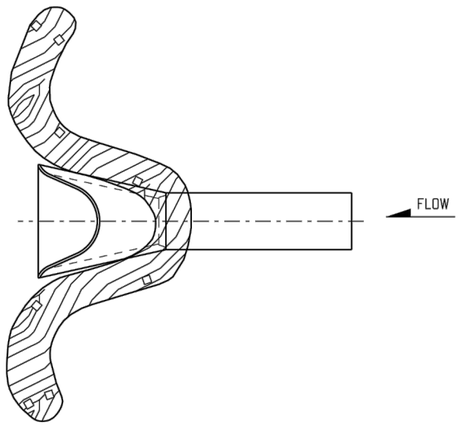
PLAN VIEW



SECTION C-C
(NOT ALL LOGS SHOWN)

NOTE: TOP OF STAKE SHALL NOT EXTEND PAST TOP OF EROSION LOG MORE THAN 2 IN.

EROSION LOG CULVERT INLET PROTECTION



EROSION LOG CULVERT OUTLET PROTECTION

EROSION LOG APPLICATIONS

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TEMPORARY EROSION CONTROL

Issued by the Project Development Branch: July 31, 2019

STANDARD PLAN NO.
M-208-1
Standard Sheet No. 2 of 11
Project Sheet Number:

2/26/2021

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3665 JFK Parkway
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EROSION CONTROL DETAILS

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

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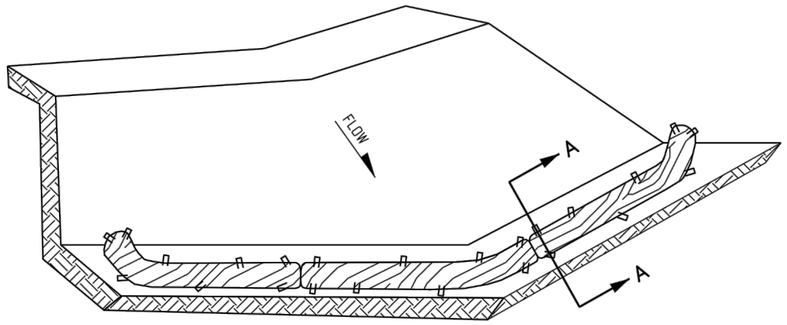
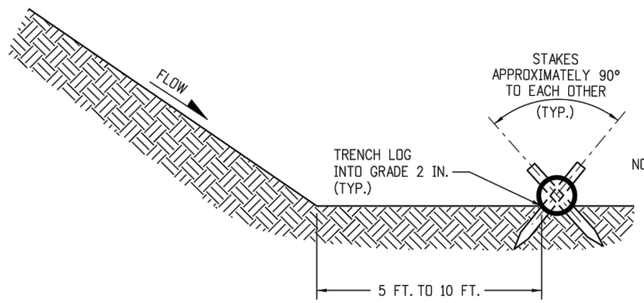
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO. 26-1144.00
DRAWING NO. C07
SHEET 13 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 EROSION CONTROL DETAILS.DWG Friday, February 26, 2021 6:41 PM By: WAYNE, ROBERT



ISOMETRIC VIEW



NOTE: THE TOPS OF ALL STAKES SHALL NOT EXTEND MORE THAN 2 INCHES ABOVE THE TOPS OF EROSION LOGS.

SECTION A-A

NOTES:

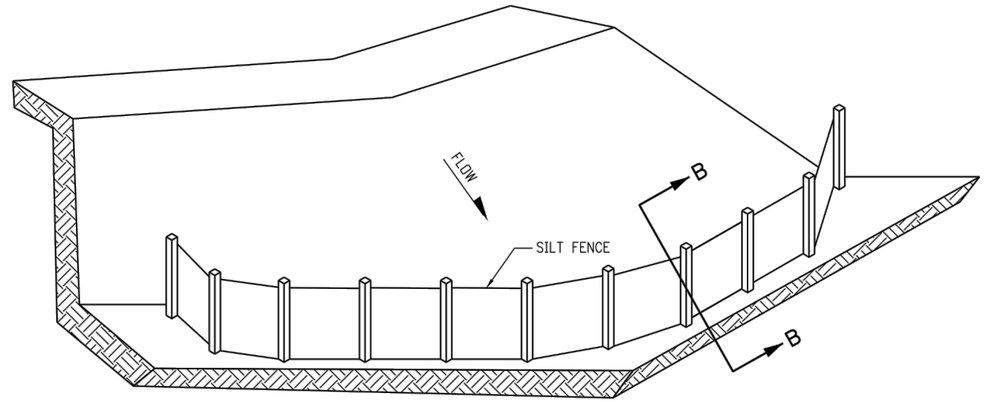
1. EROSION LOGS USED AT TOE OF SLOPE SHALL BE PLACED 5 TO 10 FEET BEYOND TOE OF SLOPE TO PROVIDE STORAGE CAPACITY.
2. EROSION LOGS SHALL BE PLACED ON THE CONTOUR WITH ENDS FLARED UP SLOPE.
3. SEE SHEET 2 OF 11 FOR JOINING LOGS DETAIL.

EROSION LOGS PAY ITEMS	
NUMBER	DESCRIPTION
208-00012	TYPE 1 (9 IN.)
208-00002	TYPE 1 (12 IN.)
208-00013	TYPE 1 (20 IN.)
208-00007	TYPE 2 (8 IN.)
208-00008	TYPE 2 (12 IN.)
208-00009	TYPE 2 (18 IN.)
208-00022	TYPE 3 (9 IN.)
208-00023	TYPE 3 (12 IN.)
208-00024	TYPE 3 (20 IN.)

EROSION LOG TOE OF SLOPE PROTECTION

NOTES

1. SILT FENCE SHALL HAVE A MAXIMUM DRAINAGE AREA OF ONE-QUARTER ACRE PER 100 FEET OF SILT FENCE LENGTH; MAXIMUM SLOPE LENGTH BEHIND BARRIER IS 100 FEET.
2. SILT FENCE USED AT TOE OF SLOPE SHALL BE PLACED 5 TO 10 FEET BEYOND TOE OF SLOPE TO PROVIDE STORAGE CAPACITY.
3. SILT FENCE SHALL BE PLACED PARALLEL TO THE CONTOUR WITH ENDS FLARED UP SLOPE.
4. THE MAXIMUM LENGTH OF EROSION LOGS OR SILT FENCES WITHOUT A FLARED END TURNING UPSLOPE IS 150 FEET.



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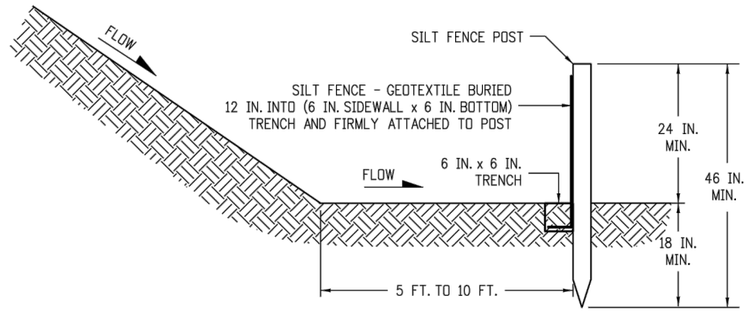

SECTION B-B

SILT FENCE TOE OF SLOPE PROTECTION

NOTE: THE PAY ITEM NUMBER FOR SILT FENCE (LF) IS 208-00020.

TOE OF SLOPE PROTECTION APPLICATIONS

Computer File Information		Sheet Revisions	
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Detailer Initials: LTA	(R-X)		
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TEMPORARY EROSION CONTROL

Issued by the Project Development Branch: July 31, 2019

STANDARD PLAN NO.
M-208-1
Standard Sheet No. 3 of 11
Project Sheet Number:

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

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EROSION CONTROL DETAILS

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Date	Revisions

Project Mgr. CTG
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Drawn By: RBR
Approved By: TMR
Date: 02/26/21

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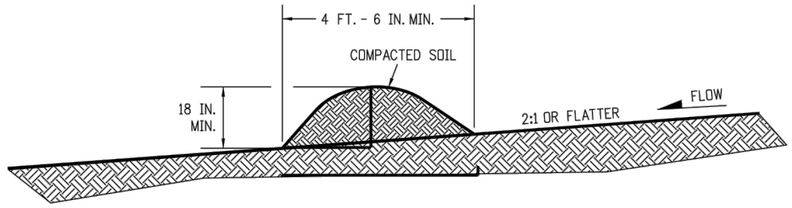
26-1144.00
DRAWING NO.
C08
SHEET 14 OF 43

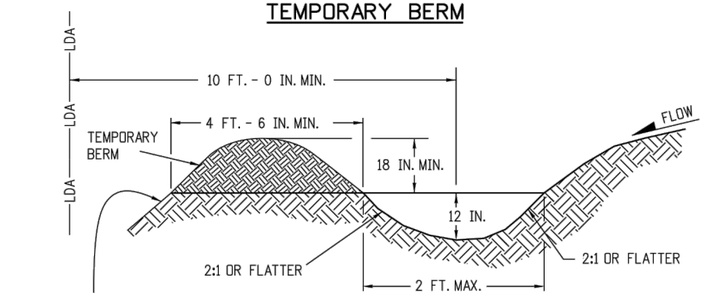
Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 EROSION CONTROL DETAILS.DWG Friday, February 26, 2021 6:41 PM By: WAYNE, ROBERT



NOTES:

1. BERMS SHALL HAVE A HEIGHT OF 18 INCHES, SIDE SLOPES OF 2:1 OR FLATTER AND A MINIMUM BASE WIDTH OF 4 FT. -6 IN.
2. BERMS SHALL BE USED TO INTERCEPT AND DIVERT DRAINAGE TO A DESIGNATED OUTLET.
3. BERMS SHALL NOT BE USED WHERE DRAINAGE AREA EXCEEDS 10 ACRES.
4. BERMS SHALL BE CONSTRUCTED OUT OF ACCEPTABLE MATERIAL THAT CAN BE COMPACTED AND RECEIVE AT A MINIMUM HEAVY EQUIPMENT WHEEL ROLLED COMPACTION.
5. TEMPORARY BERMS SHALL BE CONSTRUCTED OUT OF EMBANKMENT (SUBSOIL) AND IN NO CIRCUMSTANCE CONSTRUCTED OUT OF SALVAGED TOPSOIL.
6. THE PAY ITEM NUMBER FOR TEMPORARY BERM (LF) IS 208-00300.

TEMPORARY BERM

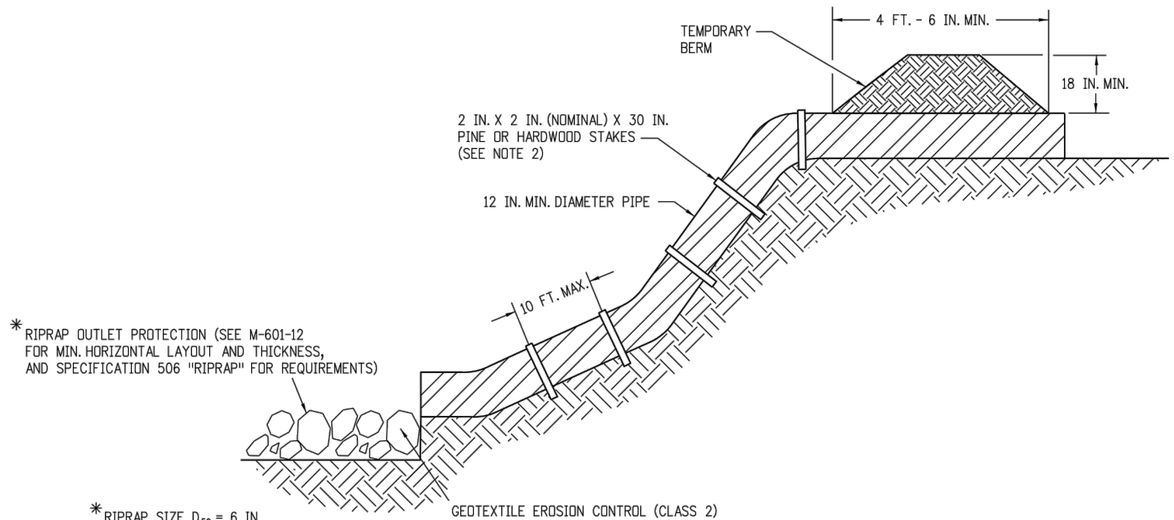


FOR BERMS TALLER THAN 2 FT.,
INSTALL TOE OF SLOPE CONTOL MEASURES.
SEE SHEET 3 OF 11 FOR DETAILS.

NOTES:

1. TEMPORARY DIVERSION DITCHES SHALL BE CONSTRUCTED ACROSS THE SLOPE TO INTERCEPT RUNOFF AND DIRECT IT TO A STABLE OUTLET OR SEDIMENT TRAP.
2. USE THE TEMPORARY DIVERSION DITCH IMMEDIATELY ABOVE A NEW CUT, FILL SLOPE, OR AROUND THE PERIMETER OF A DISTURBED AREA.
3. THE GRADIENT ALONG THE FLOW PATH SHALL HAVE A POSITIVE GRADE TO ASSURE DRAINAGE, BUT SHALL NOT BE SO STEEP AS TO RESULT IN EROSION DUE TO HIGH VELOCITY.
4. THE DIVERSION FLOWLINE SHALL ALWAYS BE LOCATED A MINIMUM 10 FEET FROM THE OUTSIDE LIMITS OF DISTURBED AREA BOUNDARY.
6. DIVERSION BERMS SHALL BE CONSTRUCTED OUT OF EMBANKMENT (SUBSOIL) AND IN NO CIRCUMSTANCE CONSTRUCTED OUT OF SALVAGED TOPSOIL.
5. THE PAY ITEM NUMBER FOR TEMPORARY DIVERSION (LF) IS 208-00301.

TEMPORARY DIVERSION



NOTES:

1. ANCHOR SIZE VARIES ACCORDING TO PIPE SIZE
2. TO SECURE THE PIPE, DRIVE STAKES INTO GROUND, THEN TIE A 12 GUAGE WIRE BETWEEN THEM ABOVE AND ACROSS THE PIPE'S WIDTH.
3. THE OUTLET SHALL BE ALIGNED WITH THE FLOW DIRECTION OF THE EXISTING GRADE. PERPENDICULAR DISCHARGE TO A CHANNEL SHALL NOT BE ACCEPTABLE.
4. THE GRADE AROUND THE INLET TO THE PIPE SHALL BE COMPACTED.
5. THE PAY ITEM NUMBER FOR TEMPORARY SLOPE DRAINS (LF) IS 208-00060.

TEMPORARY SLOPE DRAINS

GRADING APPLICATIONS

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TEMPORARY EROSION CONTROL

Issued by the Project Development Branch: July 31, 2019

STANDARD PLAN NO.
M-208-1
Standard Sheet No. 7 of 11
Project Sheet Number:

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER
2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

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EROSION CONTROL DETAILS

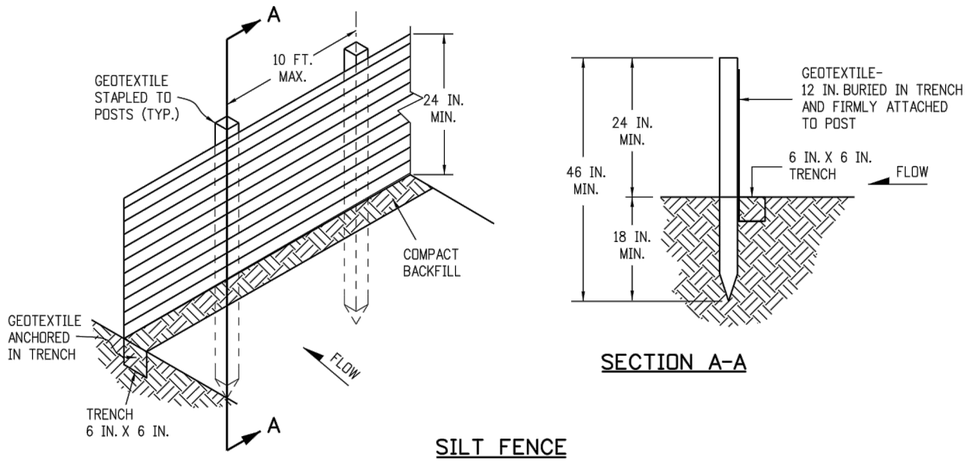
HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr. CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO. 26-1144.00
DRAWING NO. C09
SHEET 15 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 EROSION CONTROL DETAILS.DWG Friday, February 26, 2021 6:41 PM By: WAYNE, ROBERT

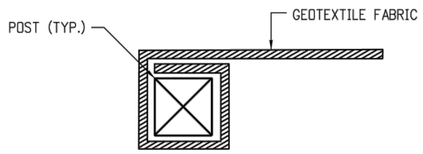


SECTION A-A

SILT FENCE

NOTES:

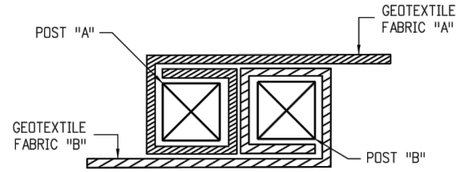
1. GEOTEXTILE SHALL BE ATTACHED TO WOOD POSTS WITH THREE OR MORE STAPLES PER POST. STAPLES SHALL BE HEAVY DUTY WIRE AND AT LEAST 1 INCH LONG.
2. WOOD POST SHALL BE 1 IN. X 1 IN. NOMINAL.
3. THE PAY ITEM NUMBER FOR SILT FENCE (LF) IS 208-00020.
4. THE SILT FENCE SHALL BE PLACED ON THE CONTOUR (AT THE SAME ELEVATION ±6 IN.). THE ENDS SHALL BE FLARED UP SLOPE (MINIMUM ELEVATION GAIN OF 18 IN.).



END SECTION DETAIL (PLAN VIEW)

NOTE:

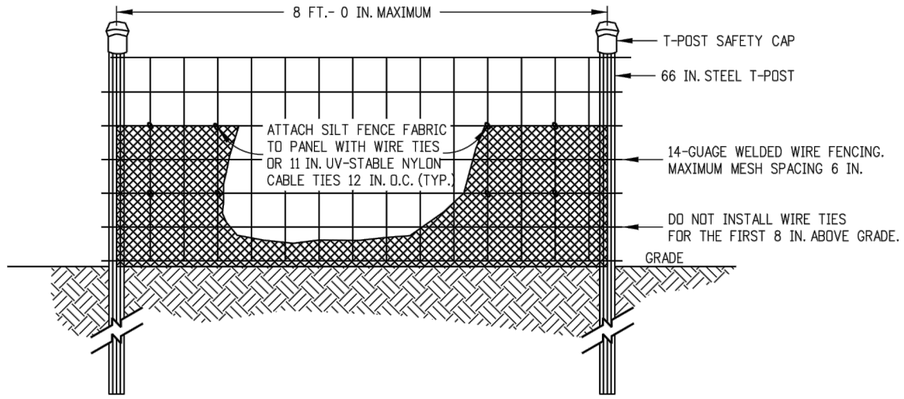
1. THE END OF THE SILT FENCE FABRIC SHALL BE WRAPPED APPROX. 6 INCHES AROUND A WOODEN POST ONE FULL TURN, THEN SECURED ALONG THE POST WITH 6 HEAVY DUTY WIRE STAPLES AT LEAST 1 INCH LONG.



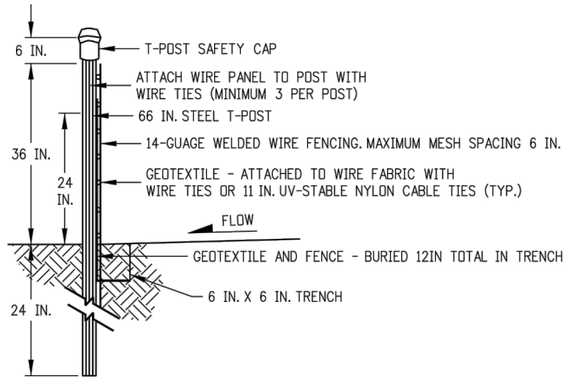
JOINING SECTION DETAIL (PLAN VIEW)

NOTES:

1. THE ENDS OF THE SILT FENCE FABRIC SHALL BE JOINED TOGETHER BY WRAPPING APPROX. 6 INCHES OF EACH END AROUND A WOODEN POST ONE FULL TURN, THEN SECURED ALONG THE POST WITH 6 HEAVY DUTY WIRE STAPLES AT LEAST 1 INCH LONG.
2. POSTS SHALL BE TIGHTLY ABUTTED WITH NO GAPS TO PREVENT POTENTIAL FLOW-THROUGH OF SEDIMENT AT JOINT.



ELEVATION VIEW



SIDE VIEW

NOTES:

1. THE ENDS OF THE SILT FENCE FABRIC SHALL BE JOINED TOGETHER BY WRAPPING APPROX. 6 INCHES OF EACH END AROUND A STEEL T-POST, THEN SECURED ALONG THE POST WITH WIRE TIES (MINIMUM 3 PER POST).
2. POSTS SHALL BE TIGHTLY ABUTTED WITH NO GAPS TO PREVENT POTENTIAL FLOW-THROUGH OF SEDIMENT AT JOINT.
3. SILT FENCES SHALL NOT BE USED FOR CHECK DAMS.
4. THE PAY ITEM NUMBER FOR SILT FENCE (REINFORCED) (LF) IS 208-00021.

SILT FENCE (REINFORCED)

SILT FENCE APPLICATIONS

Computer File Information		Sheet Revisions	
Creation Date: 07/31/19		Date:	Comments
Designer Initials: JBK	(R-X)		
Last Modification Date: 07/31/19	(R-X)		
Detailer Initials: LTA	(R-X)		
CAD Ver.: MicroStation V8 Scale: Not to Scale Units: English	(R-X)		

Colorado Department of Transportation
2829 West Howard Place
CDOT HQ, 3rd Floor
Denver, CO 80204
Phone: 303-757-9021 FAX: 303-757-9868
Project Development Branch JBK

TEMPORARY EROSION CONTROL

Issued by the Project Development Branch: July 31, 2019

STANDARD PLAN NO.
M-208-1
Standard Sheet No. 8 of 11
Project Sheet Number:

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
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(970) 223-5556

EROSION CONTROL DETAILS

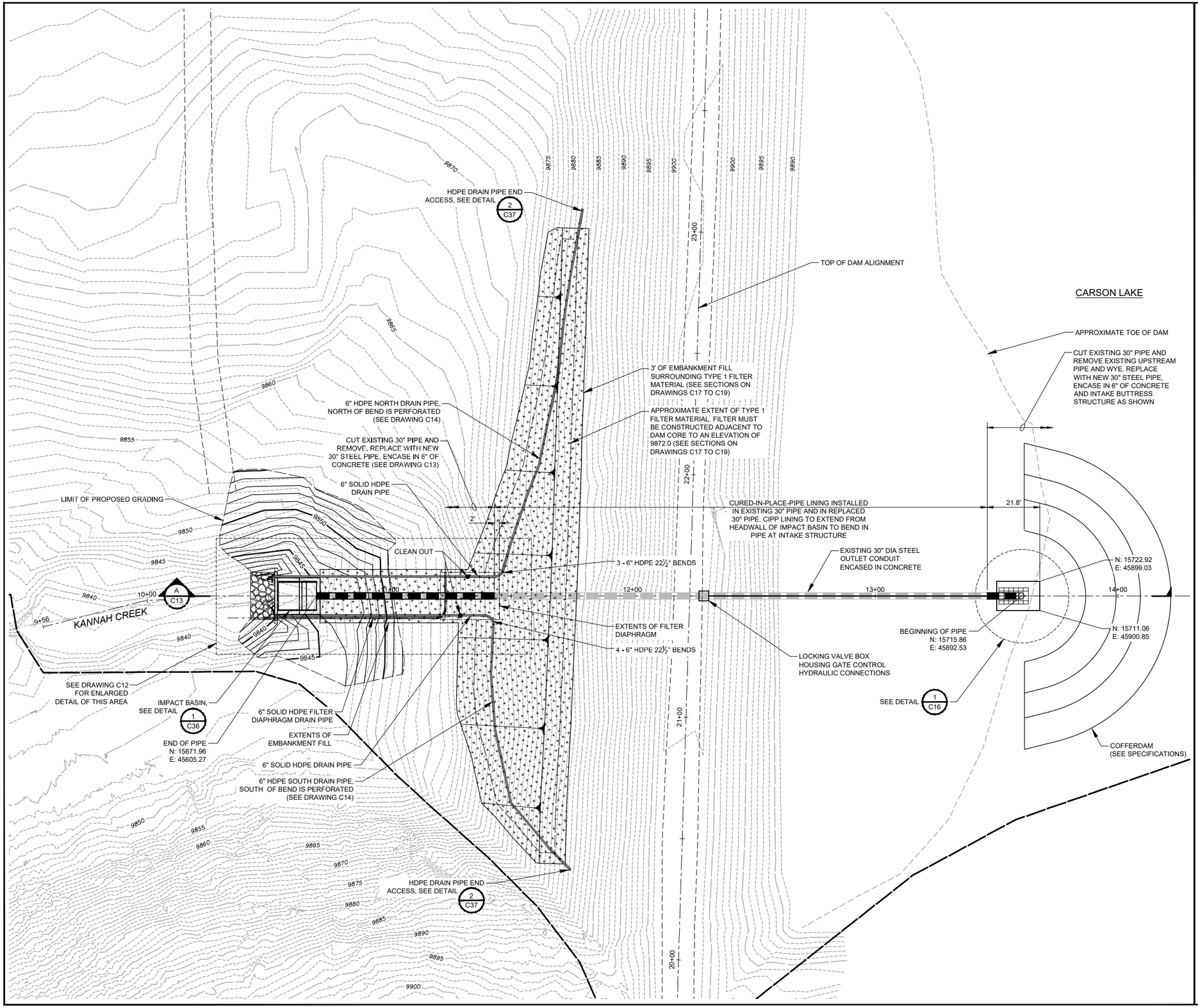
HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr. CTG	Designed By: AJS	Drawn By: RBR	Approved By: TMR	Date: 02/26/21

PROJECT NO. 26-1144.00
DRAWING NO. C10
SHEET 16 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 EROSION CONTROL DETAILS.DWG Friday, February 26, 2021 6:41 PM By: WAYNE, ROBERT



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COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER 2/26/2021

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DAM PROPOSED CONDITIONS AND COFFERDAM

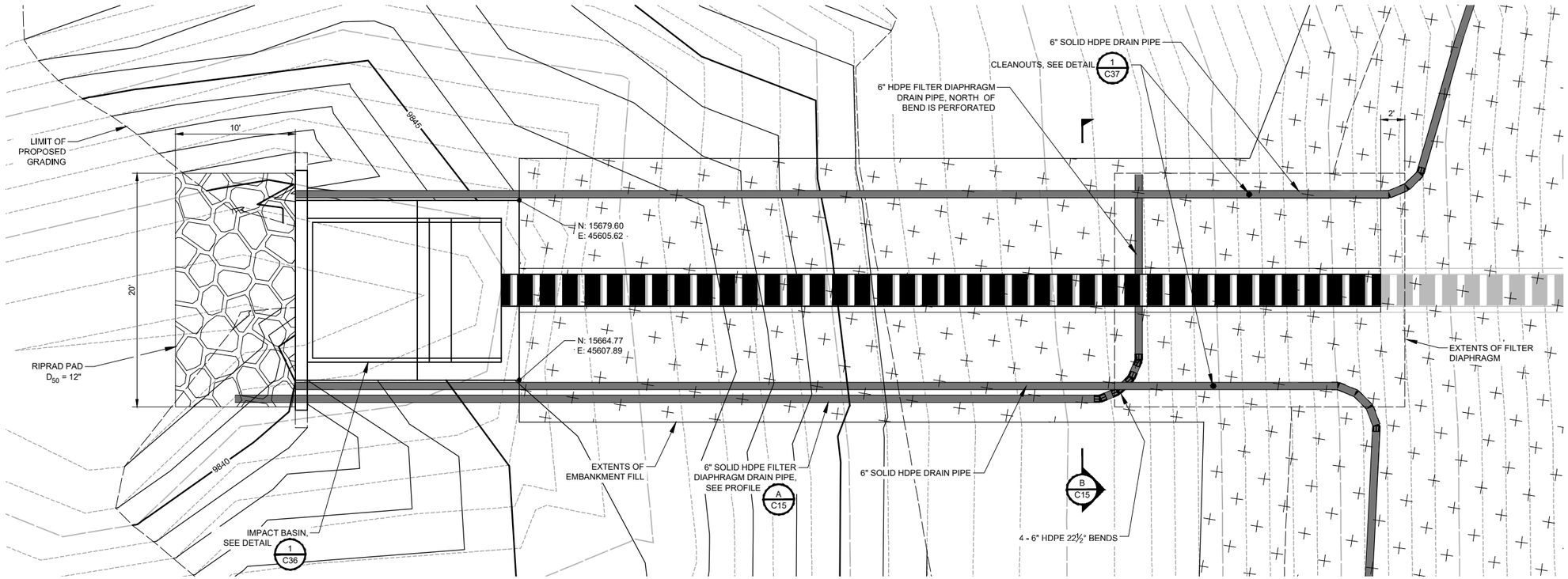
HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C11



FILTER DIAPHRAGM AND IMPACT BASIN AREA ENLARGED PLAN

SCALE: AS SHOWN

Drawing Name: i:\36\26-1144400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM PLAN ENLARGED.DWG Friday, February 26, 2021 6:41 PM By: WAYNE, ROBERT

COLORADO LICENSED TODD M. RUDQUIST 58436 PROFESSIONAL ENGINEER 2/26/2021

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DAM PROPOSED CONDITIONS ENLARGED PLAN

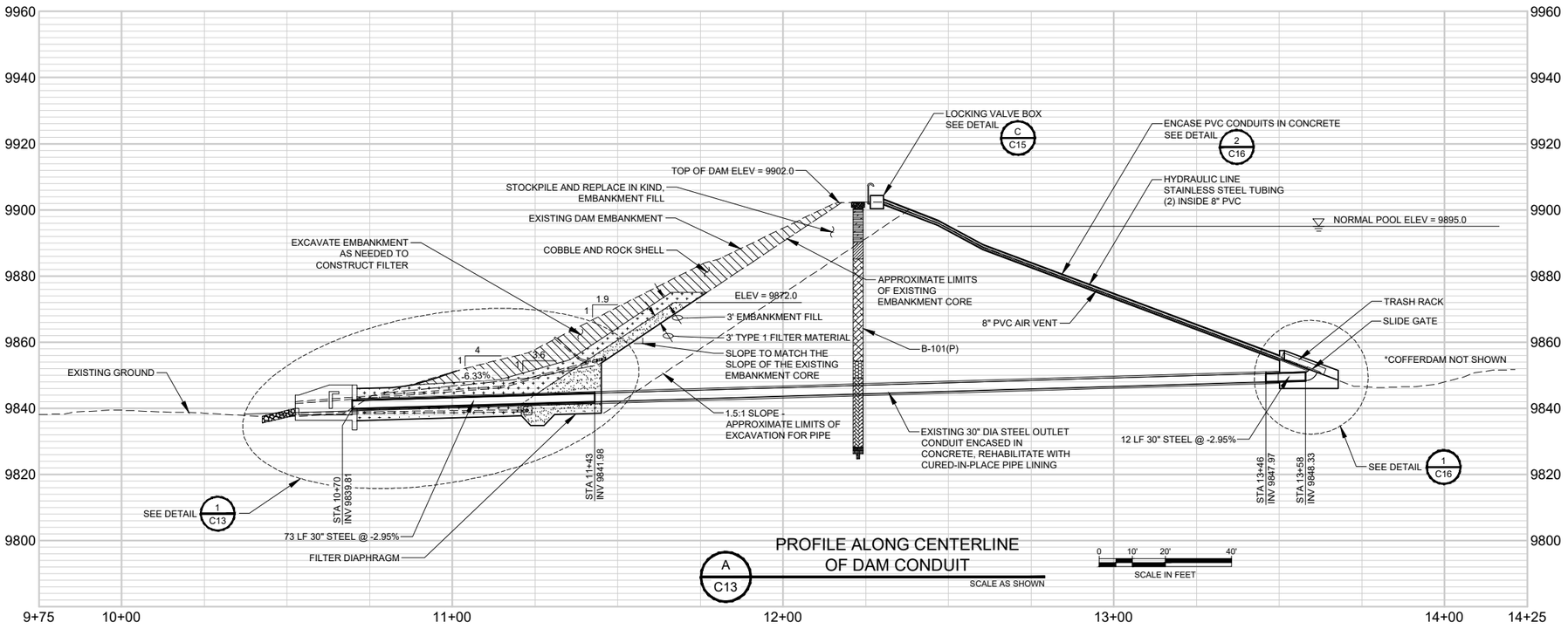
HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr. CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C12

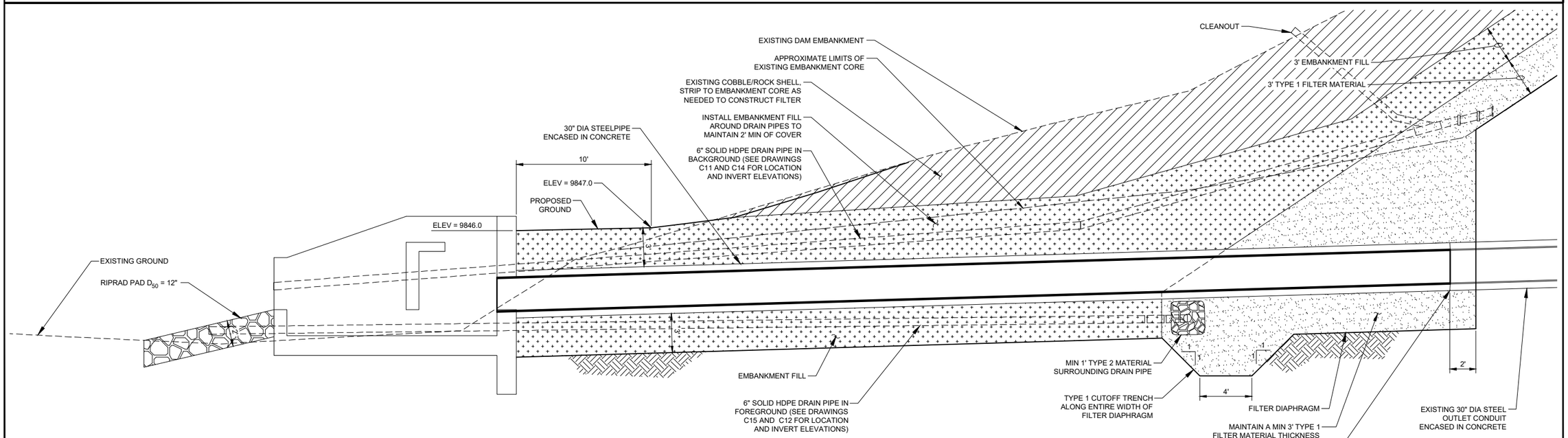


A C13 PROFILE ALONG CENTERLINE OF DAM CONDUIT

SCALE AS SHOWN

0 10' 20' 40'

SCALE IN FEET



1 C13 DIAPHRAGM FILTER AND OUTLET DETAIL

SCALE AS SHOWN

0 2' 4' 8'

SCALE IN FEET

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2021

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DAM PROFILE AND DETAIL

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

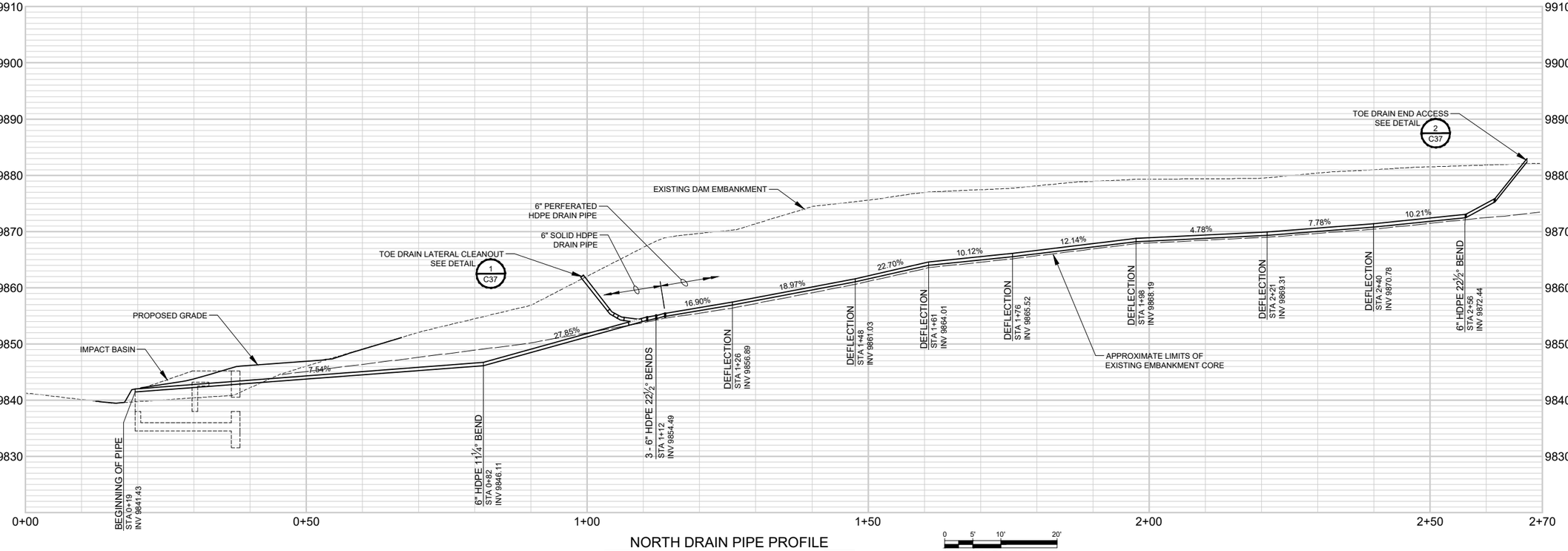
Project Mgr. CTG	
Designed By: AJS	
Drawn By: RBR	
Approved By: TMR	
Date: 02/26/21	

PROJECT NO.
26-1144.00

DRAWING NO.
C13

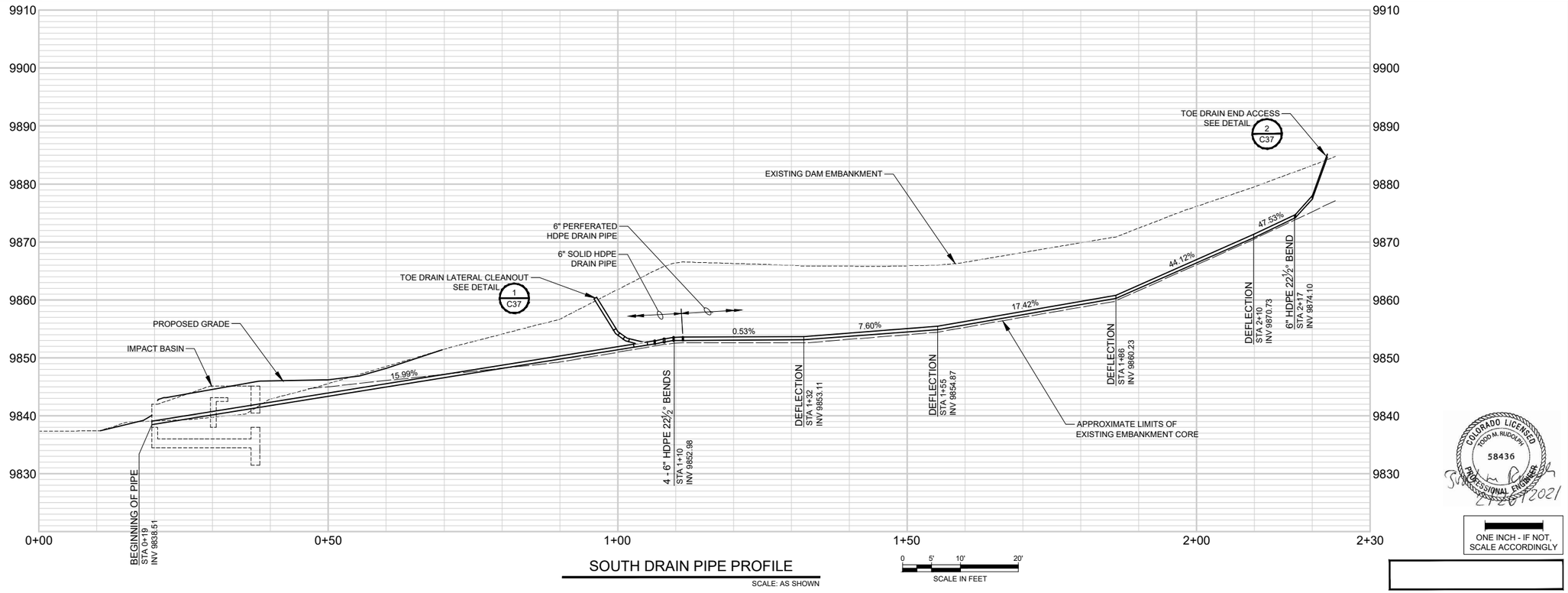
SHEET 19 OF 43

Drawing Name: i:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM PROFILE AND DETAIL.DWG Monday, March 08, 2021 9:35 AM By: WAYNE, ROBERT



NORTH DRAIN PIPE PROFILE

SCALE: AS SHOWN



SOUTH DRAIN PIPE PROFILE

SCALE: AS SHOWN

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3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

DAM DRAIN PIPE PROFILES

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

COLORADO LICENSED
TODD M. BUDCAPP
58436
PROFESSIONAL ENGINEER
2/26/2021

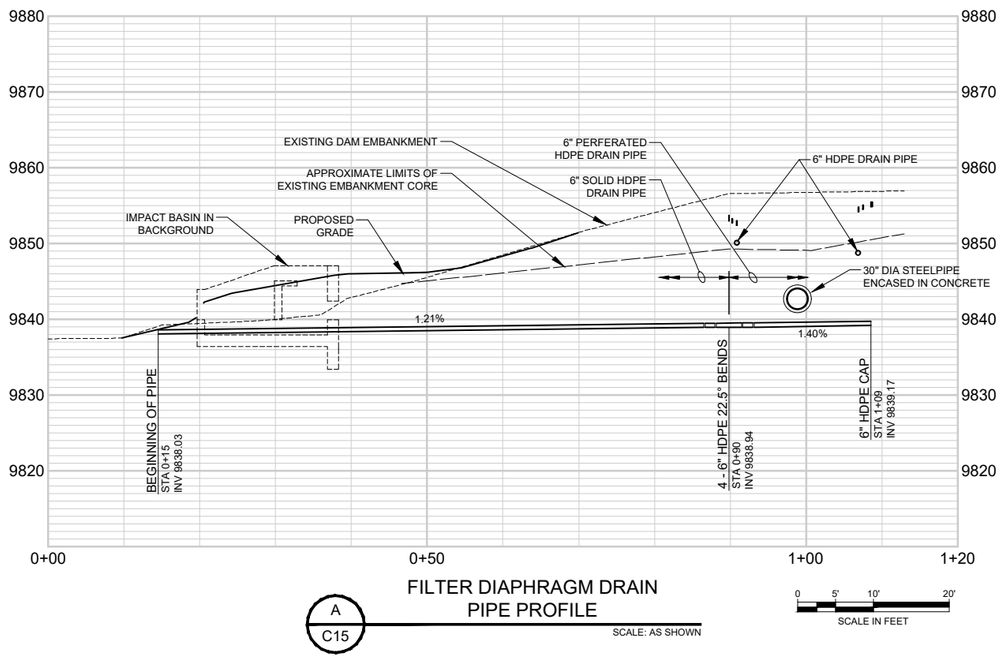
PROJECT NO.
26-1144.00

DRAWING NO.
C14

SHEET 20 OF 43

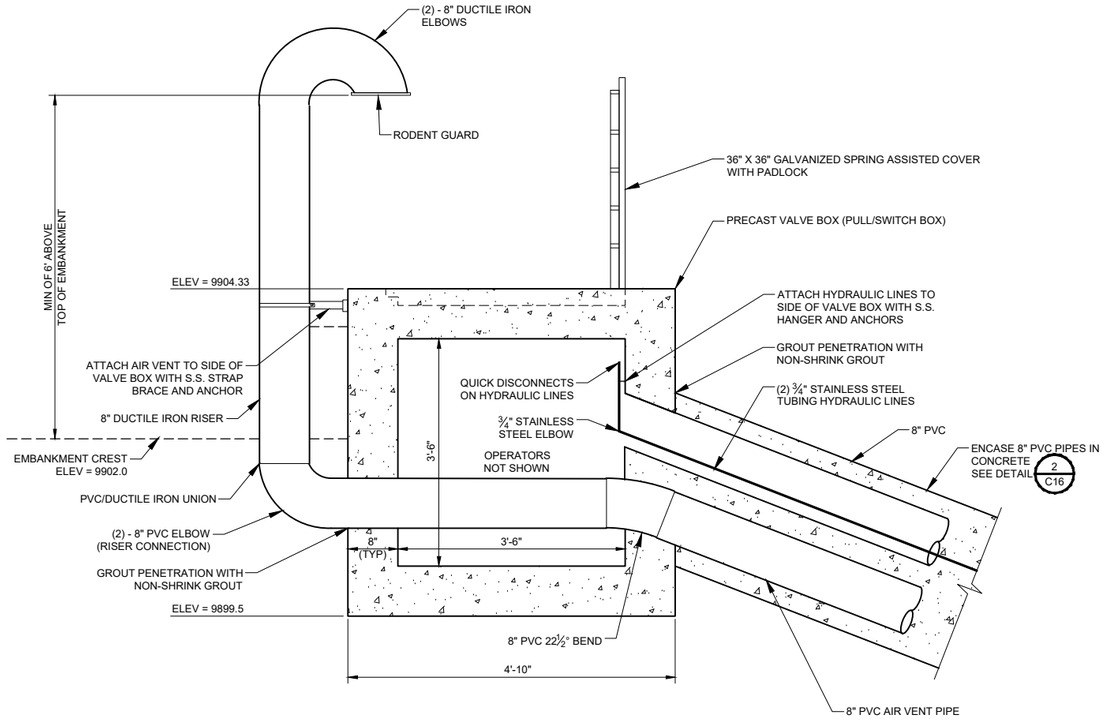
ONE INCH - IF NOT, SCALE ACCORDINGLY

Drawing Name: I:\36\26-1144\00 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DRAIN PIPE PROFILES.DWG Friday, February 26, 2021 6:46 PM By: WAYNE, ROBERT



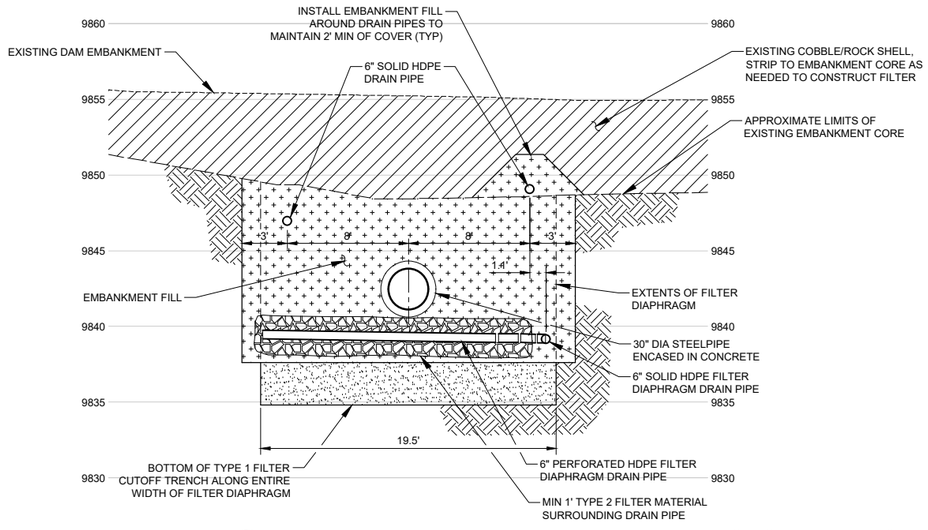
A C15 FILTER DIAPHRAGM DRAIN PIPE PROFILE

SCALE: AS SHOWN



C C15 LOCKING VALVE BOX DETAIL

NOT TO SCALE



B C15 SECTION

SCALE: AS SHOWN

COLORADO LICENSED TODD M. RUDDELL 58436 PROFESSIONAL ENGINEER 2/26/2021

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DAM DETAILS 1 OF 2

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

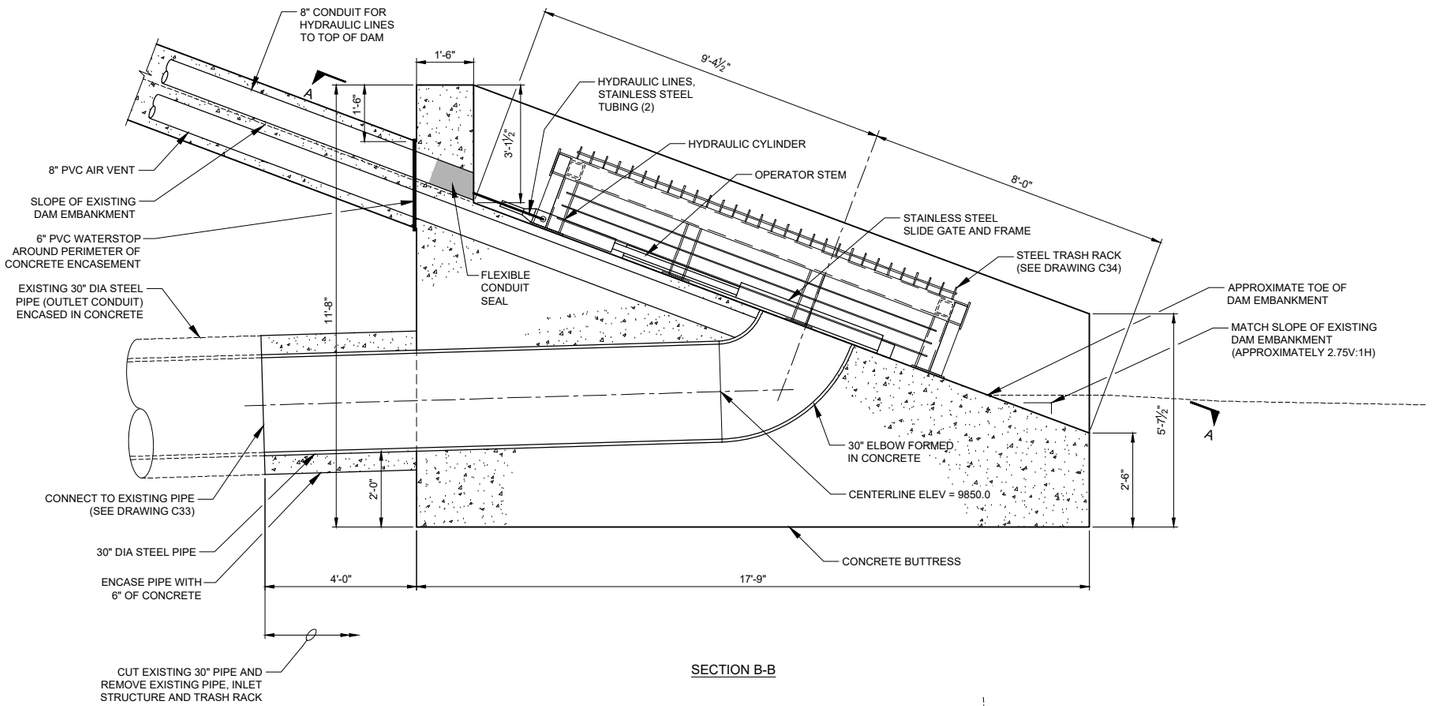
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO. 26-1144.00

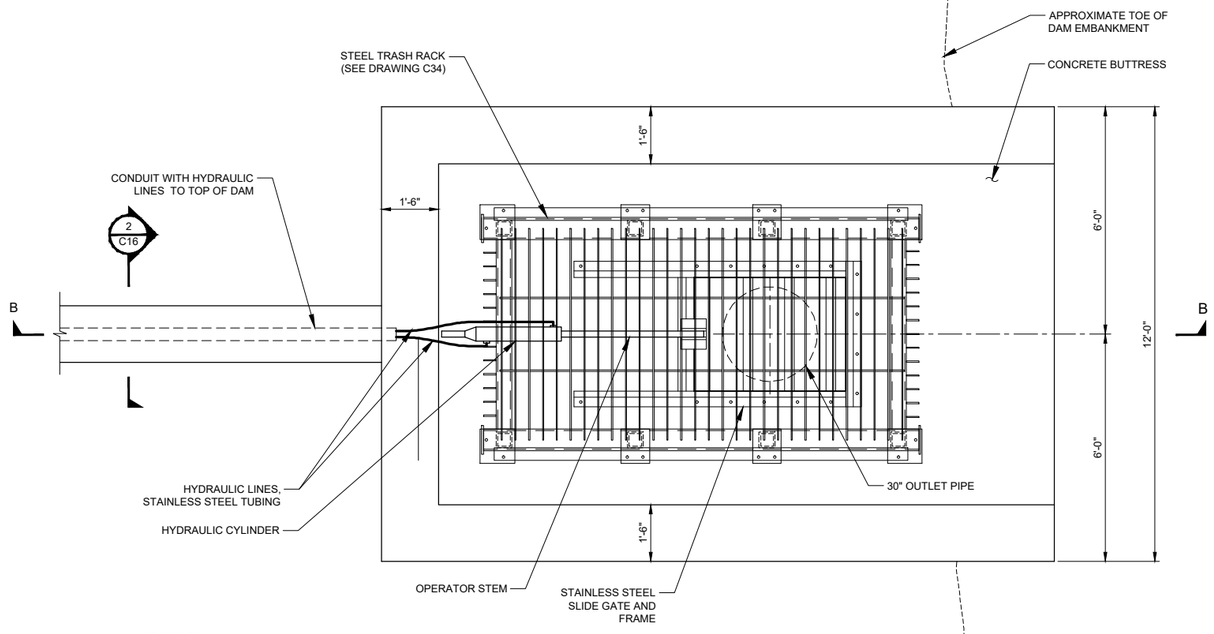
DRAWING NO. C15

SHEET 21 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM DETAILS 1 OF 2.DWG Friday, February 26, 2021 6:46 PM By: WAYNE, ROBERT



SECTION B-B



PLAN A-A

NOTES:

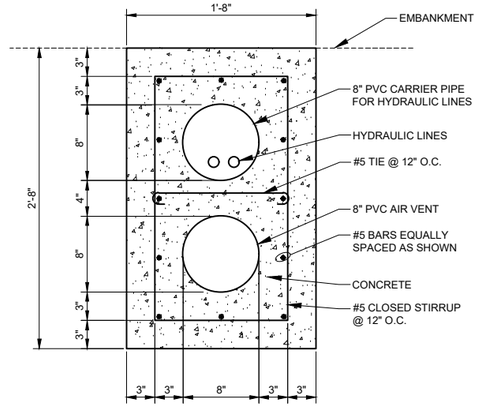
1. CONCRETE REINFORCEMENT NOT SHOWN (SEE DRAWING C32).

1 C16 — CONCRETE BUTTRESS DETAIL

SCALE: 1/2"=1'-0"

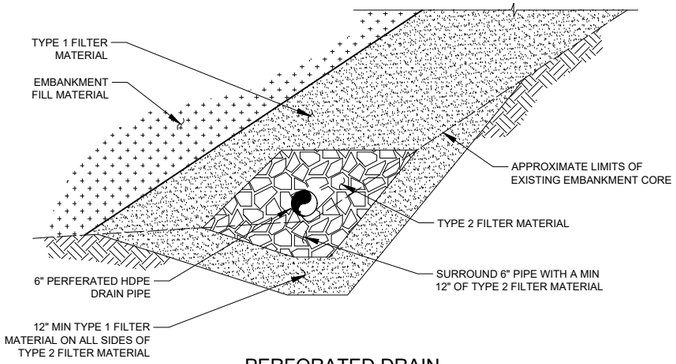
NOTES:

1. PROVIDE CONTINUOUS PVC WATERSTOPS AT ALL JOINTS.



2 C16 — CONDUIT ENCASEMENT

NOT TO SCALE



3 C16 — PERFORATED DRAIN PIPE TRENCH DETAIL

SCALE: 1"=2'

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
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DAM DETAILS 2 OF 2

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

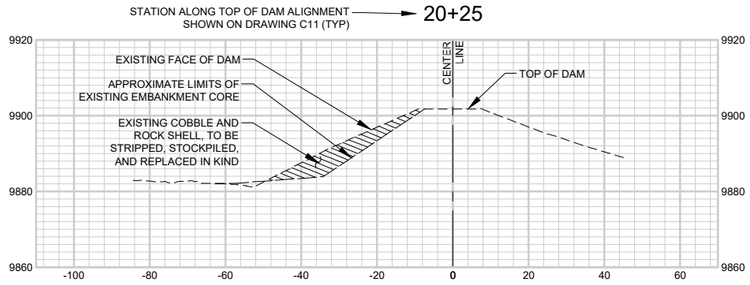
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

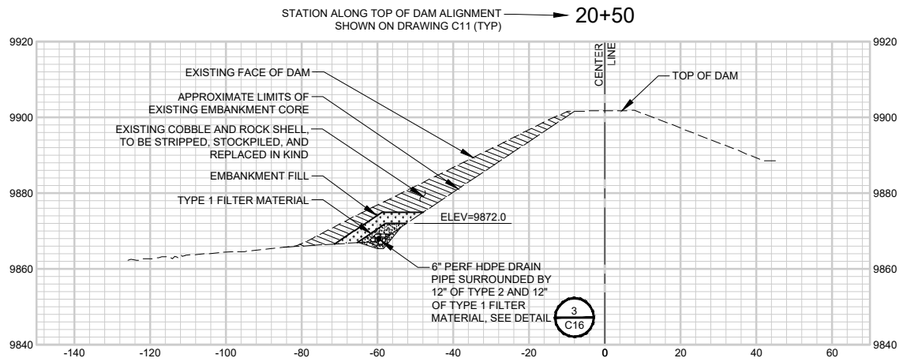
PROJECT NO. 26-1144.00

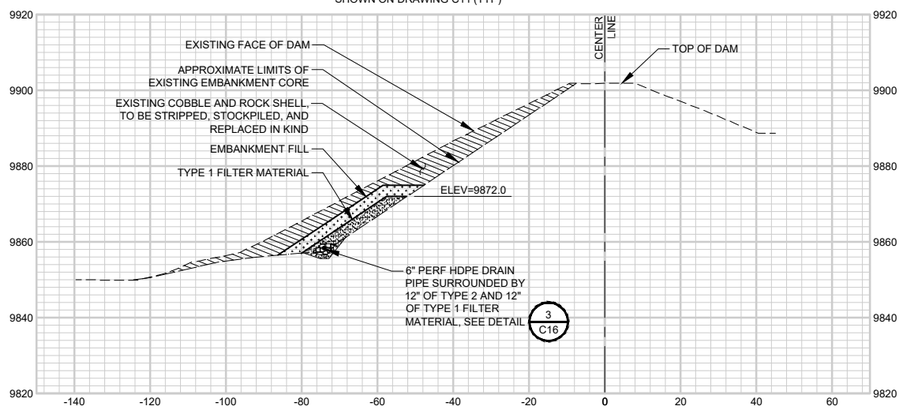
DRAWING NO. C16

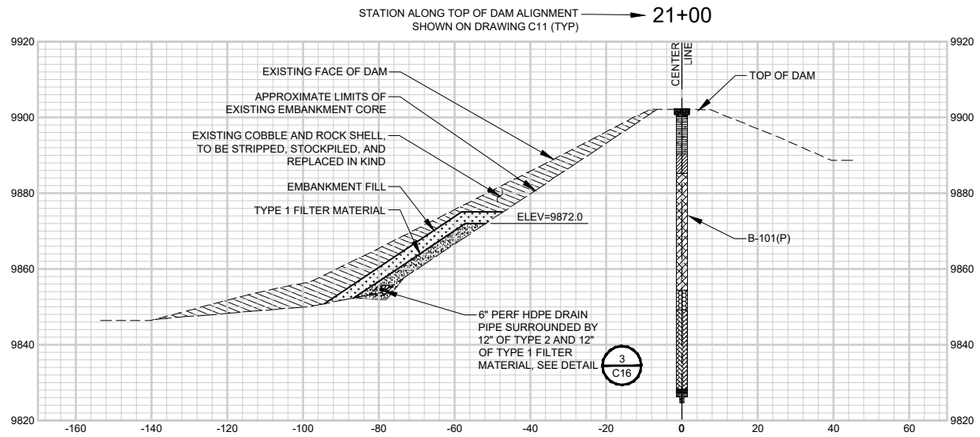
SHEET 22 OF 43

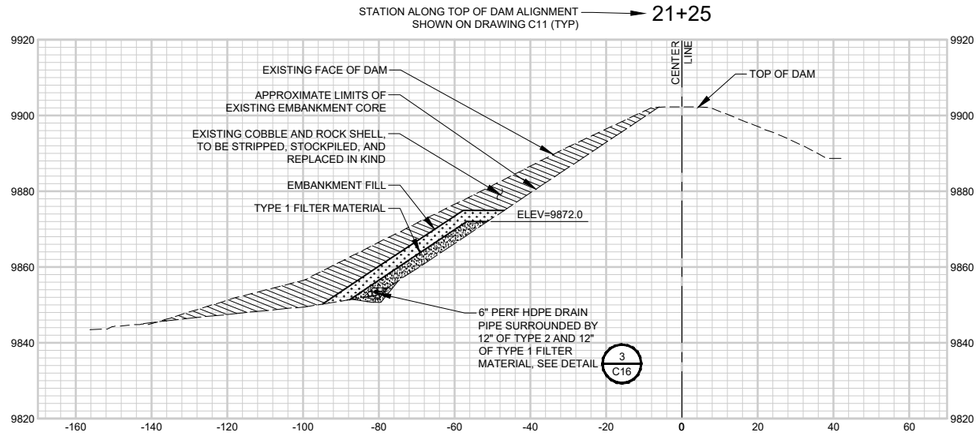
Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 IMPACT BASIN DETAILS.DWG Friday, February 26, 2021 6:51 PM By: WAYNE, ROBERT

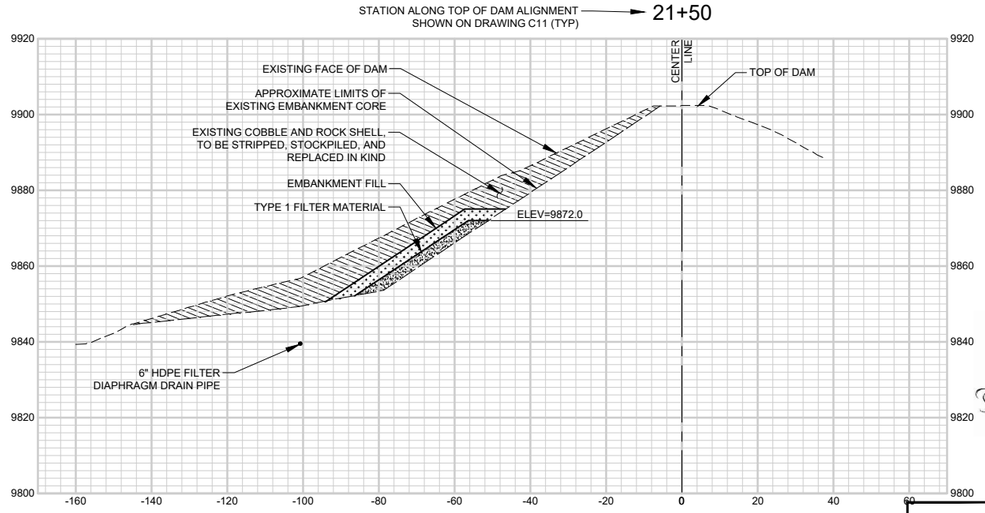












0 10' 20' 40'
SCALE IN FEET

ONE INCH - IF NOT,
SCALE ACCORDINGLY

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

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Building 2, Suite 100
Fort Collins, CO 80525
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DAM SECTIONS 1 OF 3

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Date	Revisions

Project Mgr.	CTG	Approved By:	TMR
Designed By:	AJS	Date:	02/26/21
Drawn By:	RBR		

PROJECT NO.
26-1144.00

DRAWING NO.
C17

SHEET 23 OF 43

Drawing Name: I:\26\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM SECTIONS 3 OF 3.DWG Friday, February 26, 2021 6:47 PM By: WAYNE, ROBERT

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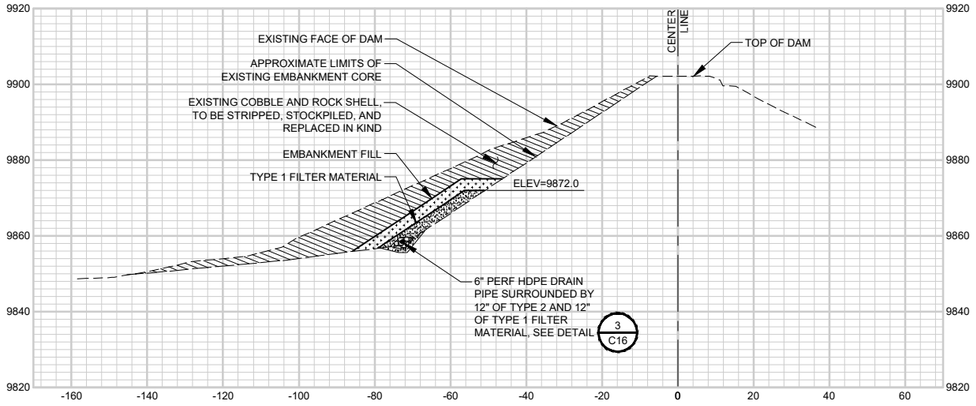
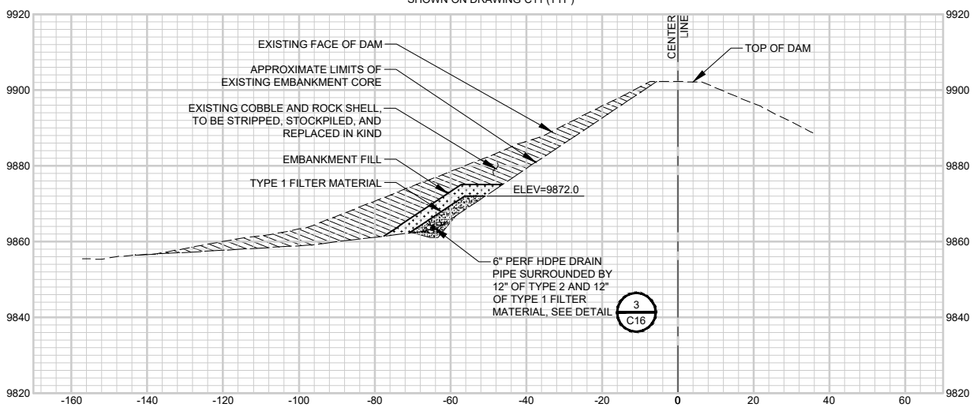


STATION ALONG TOP OF DAM ALIGNMENT SHOWN ON DRAWING C11 (TYP) → 22+00



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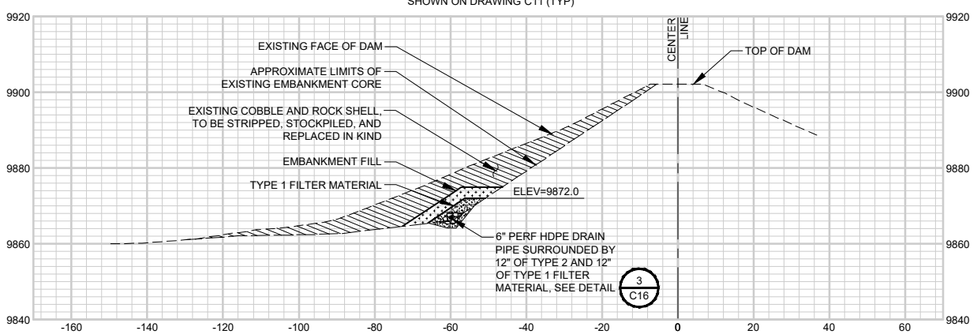

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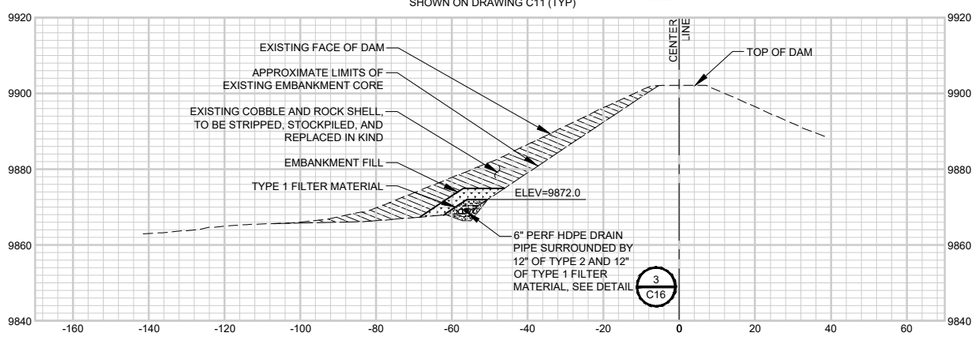

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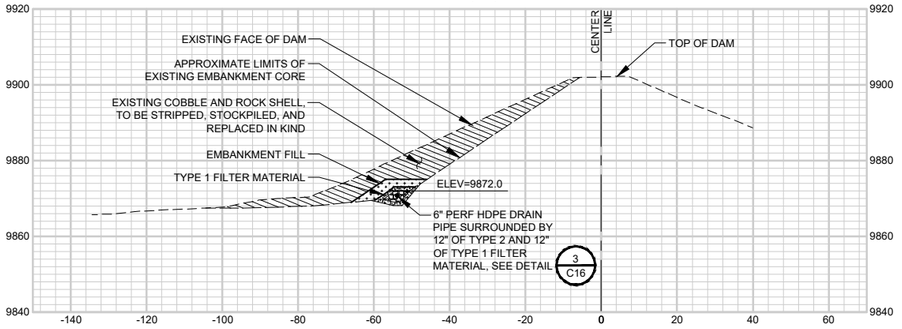

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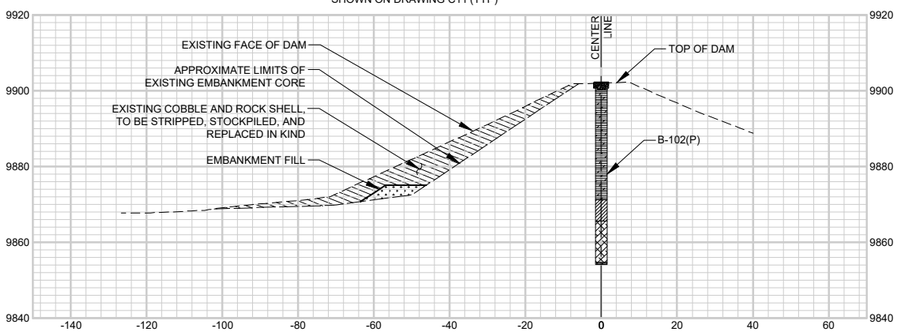

0 10' 20' 40'
SCALE IN FEET

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

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DAM SECTIONS 2 OF 3

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

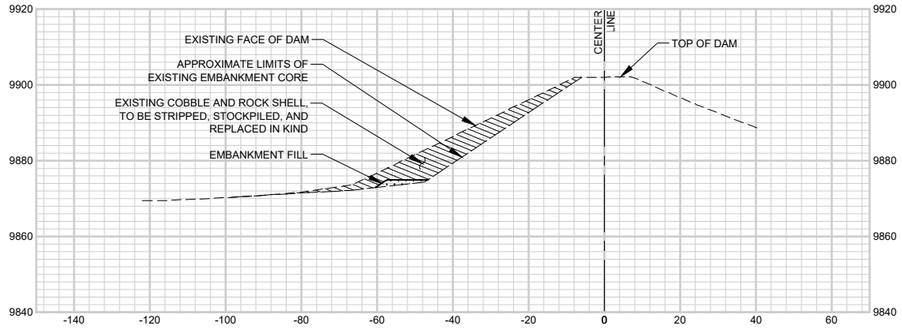
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C18

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM SECTIONS 3 OF 3.DWG Friday, February 26, 2021 6:47 PM By: WAYNE, ROBERT

STATION ALONG TOP OF DAM ALIGNMENT SHOWN ON DRAWING C11 (TYP) → 23+25



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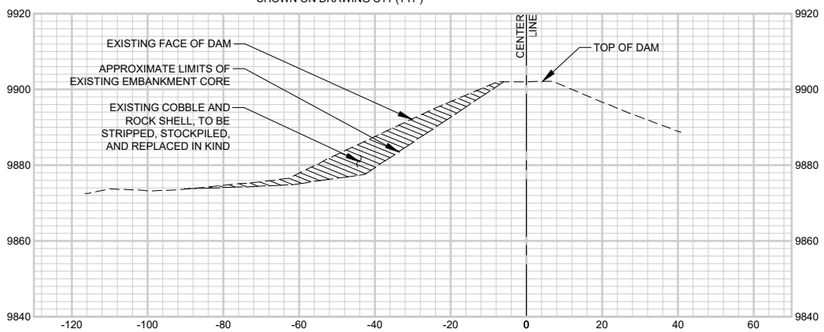

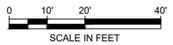

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3665 JFK Parkway
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DAM SECTIONS 3 OF 3

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

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Approved By:	TMR
Date:	02/26/21

COLORADO LICENSED
TODD M. RUDOLPH
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PROFESSIONAL ENGINEER
2/26/2021

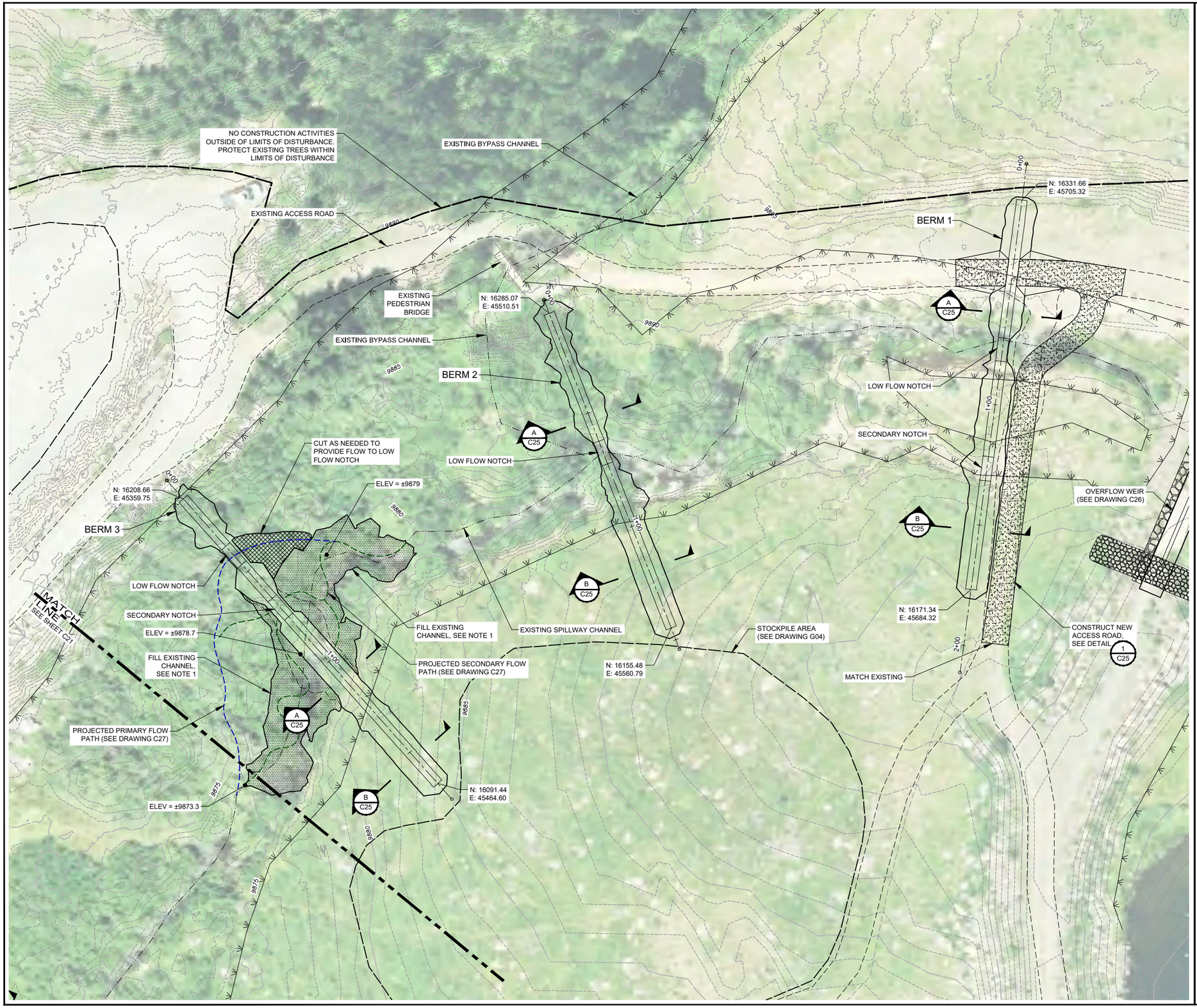
ONE INCH - IF NOT, SCALE ACCORDINGLY

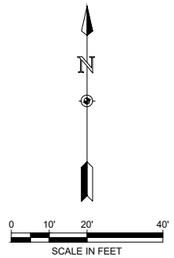
PROJECT NO.
26-1144.00

DRAWING NO.
C19

SHEET 25 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM SECTIONS 3 OF 3.DWG Friday, February 26, 2021 6:47 PM By: WAYNE, ROBERT





NOTES:

1. FILL N-VEGETATED AREA OF EXISTING CHANNEL APPROXIMATELY TO THE LIMITS AND ELEVATIONS SHOWN. FILL UP TO ADJACENT VEGETATED FLOOD PLAIN. FILL ELEVATION ADJACENT TO THE BERM SHOULD BE A MINIMUM 1' BELOW TOP OF BERM ELEVATION.
2. FILL ELEVATIONS SHOWN ARE APPROXIMATE AND MAY BE FIELD ADJUSTED DURING CONSTRUCTION.
3. FILL MATERIAL MAY INCLUDE COBBLE AND BOULDERS, BUT SHALL HAVE A MINIMUM OF 18" SOIL COVER.
4. WATTLES TO BE PLACED ON CHANNEL FILL AREAS. SEE DRAWING C27 FOR DETAILS.

COLORADO LICENSED
37109
CHRISTIN WAYNE ROBINSON
PROFESSIONAL ENGINEER
2-26-2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

SPILLWAY PLAN 1 OF 3

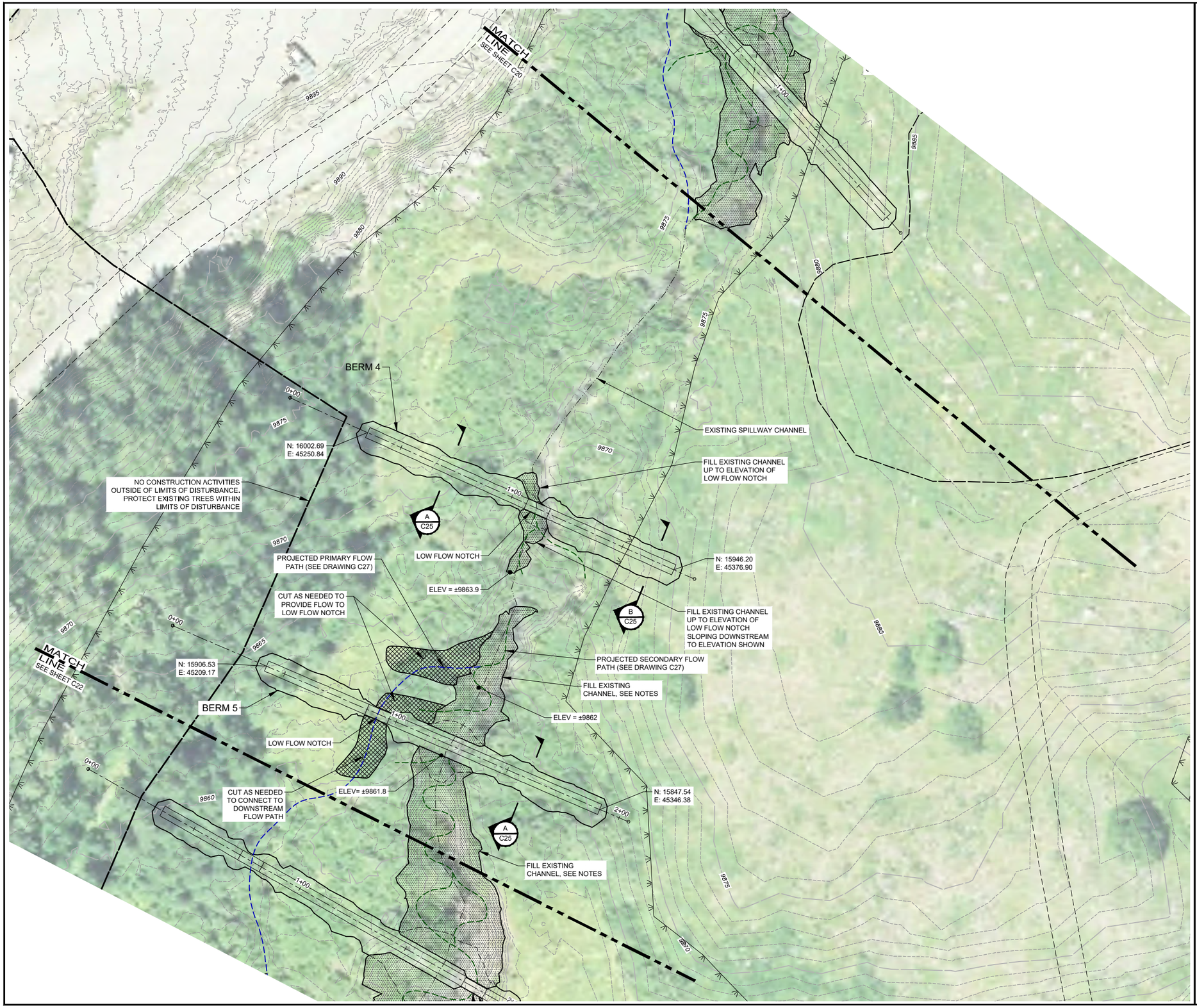
HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

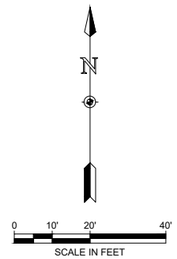
Revisions	Date

Project Mgr.	CTG
Designed By:	DWR
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00
DRAWING NO.
C20
SHEET 26 OF 43

Drawing Name: I:\26\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY PLAN 3 OF 3.DWG Friday, February 26, 2021 6:48 PM By: WAYNE, ROBERT





NOTES:

1. FILL UN-VEGETATED AREA OF EXISTING CHANNEL APPROXIMATELY TO THE LIMITS AND ELEVATIONS SHOWN. FILL UP TO ADJACENT VEGETATED FLOOD PLAIN. FILL ELEVATION ADJACENT TO THE BERM SHOULD BE A MINIMUM 1' BELOW TOP OF BERM ELEVATION.
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ONE INCH - IF NOT, SCALE ACCORDINGLY

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SPILLWAY PLAN 2 OF 3

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

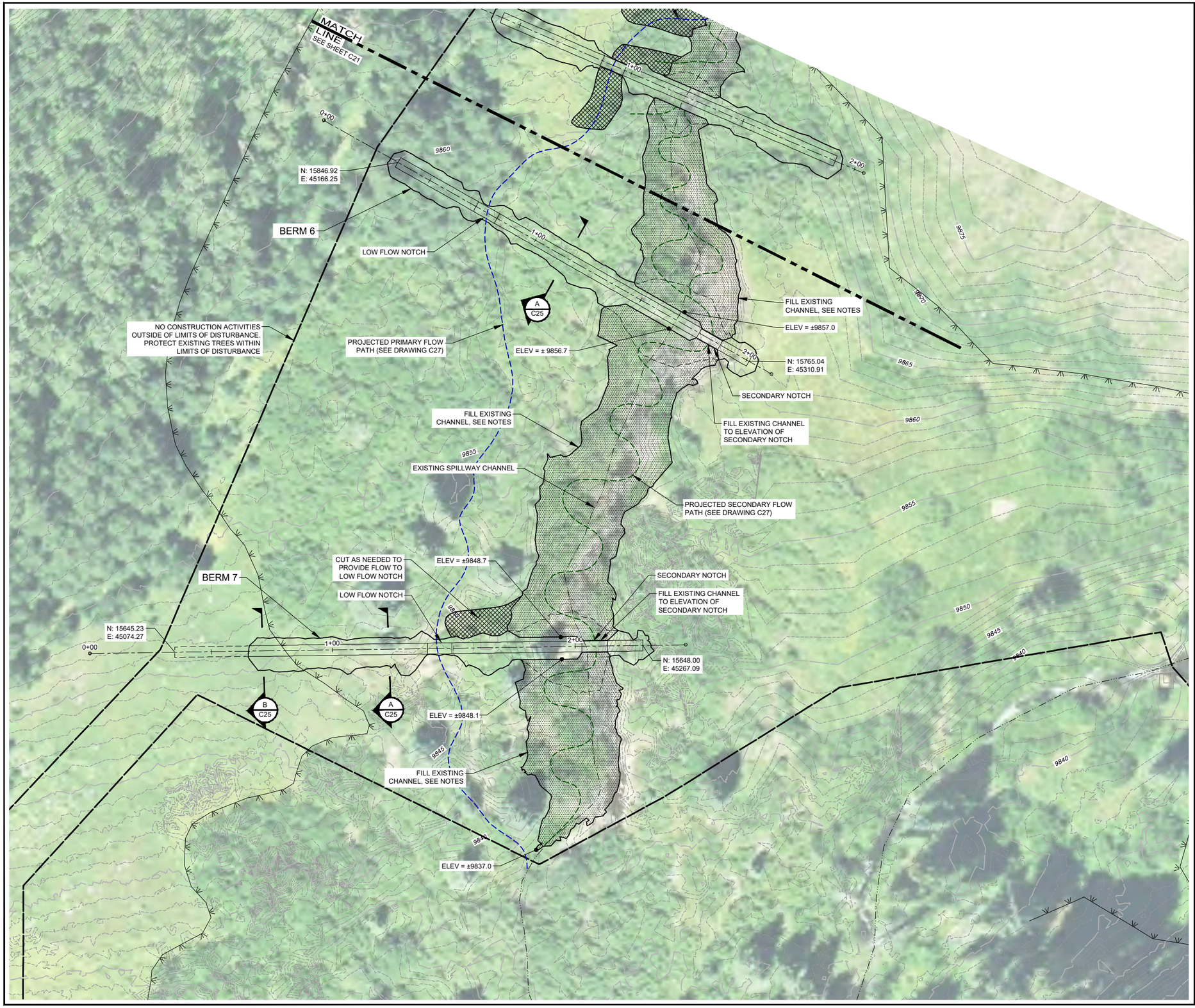
Project Mgr.	CTG
Designed By:	DWR
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C21

SHEET 27 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY PLAN 3 OF 3.DWG Friday, February 26, 2021 6:48 PM By: WAYNE, ROBERT



0 10' 20' 40'
SCALE IN FEET

NOTES:

1. FILL UN-VEGETATED AREA OF EXISTING CHANNEL APPROXIMATELY TO THE LIMITS AND ELEVATIONS SHOWN. FILL UP TO ADJACENT VEGETATED FLOOD PLAIN. FILL ELEVATION ADJACENT TO THE BERM SHOULD BE A MINIMUM 1' BELOW TOP OF BERM ELEVATION.
2. FILL ELEVATIONS SHOWN ARE APPROXIMATE AND MAY BE FIELD ADJUSTED DURING CONSTRUCTION.
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4. WATTLES TO BE PLACED ON CHANNEL FILL AREAS. SEE DRAWING C27 FOR DETAILS.

COLORADO LICENSED
37109
JUSTIN WAYNE ROBINSON
PROFESSIONAL ENGINEER
2-26-2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

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(970) 223-5556

SPILLWAY PLAN 3 OF 3

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

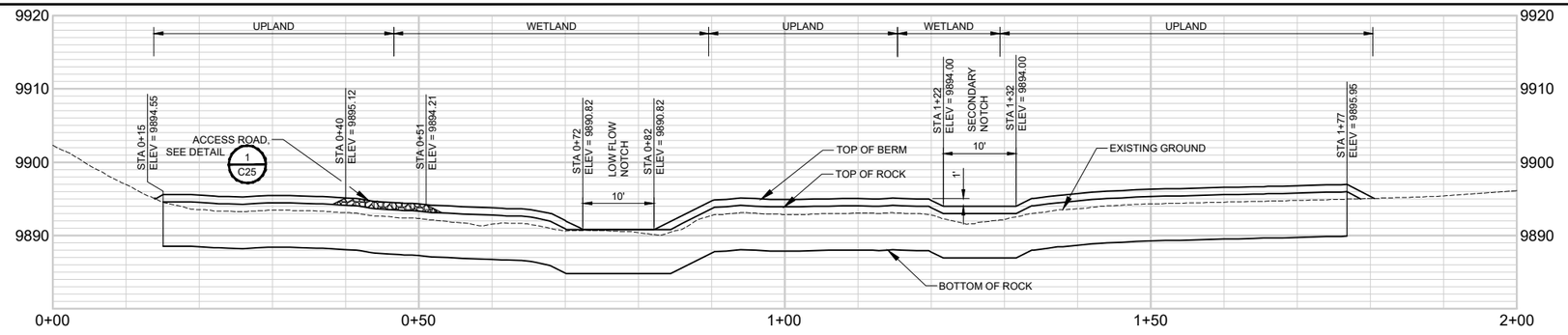
Project Mgr. CTG
Designed By: DWR
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C22

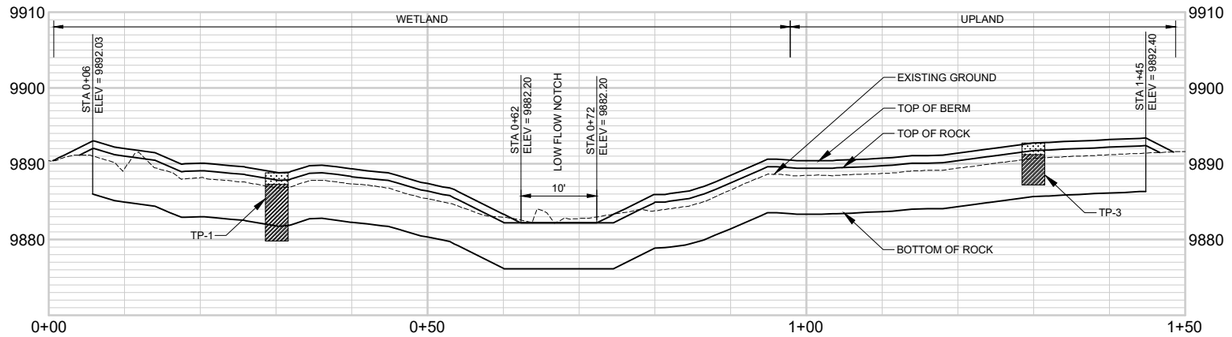
SHEET 28 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY PLAN 3 OF 3.DWG Friday, February 26, 2021 6:48 PM By: WAYNE, ROBERT



BERM 1 PROFILE

SCALE: AS SHOWN



BERM 2 PROFILE

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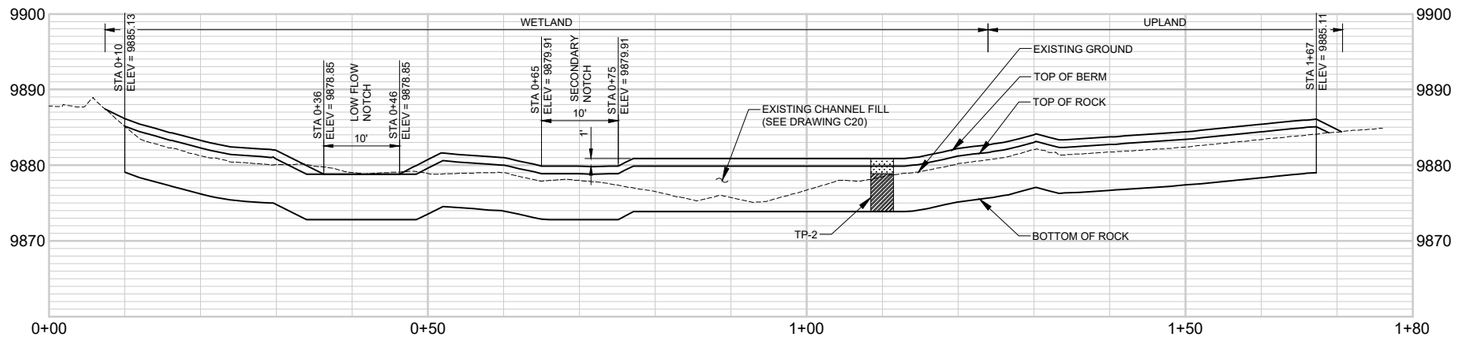

BERM 3 PROFILE

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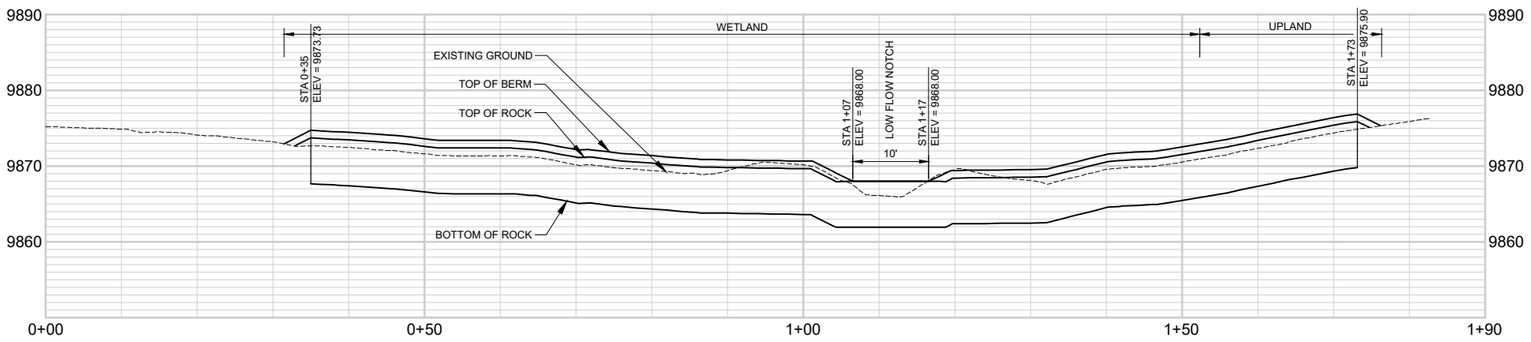
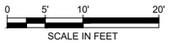


BERM 4 PROFILE

SCALE: AS SHOWN



NOTES:

1. BERM PROFILES ARE LOOKING UPSTREAM.
2. TP-1, TP-2, AND TP-3 SHOW EXISTING SOIL PROFILE AT TEST PIT LOCATIONS.
3. ALL LOW FLOW NOTCH SIDE SLOPES SHALL BE 2:1.

TEST PIT LEGEND:

CLAYEY SAND WITH GRAVEL AND ORGANICS (TOPSOIL)

CLAYEY SAND WITH GRAVEL TO CLAYEY GRAVEL WITH SAND, SOME COBBLES, TRACE BOULDERS, BROWN, MOIST, DENSE TO VERY DENSE.

COLORADO LICENSED
37109
JUSTIN WAYNE ROBINSON
PROFESSIONAL ENGINEER
2-26-2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

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Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

SPILLWAY ROCK BERM PROFILES 1 OF 2

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date
NOT FOR CONSTRUCTION	JAN 2021

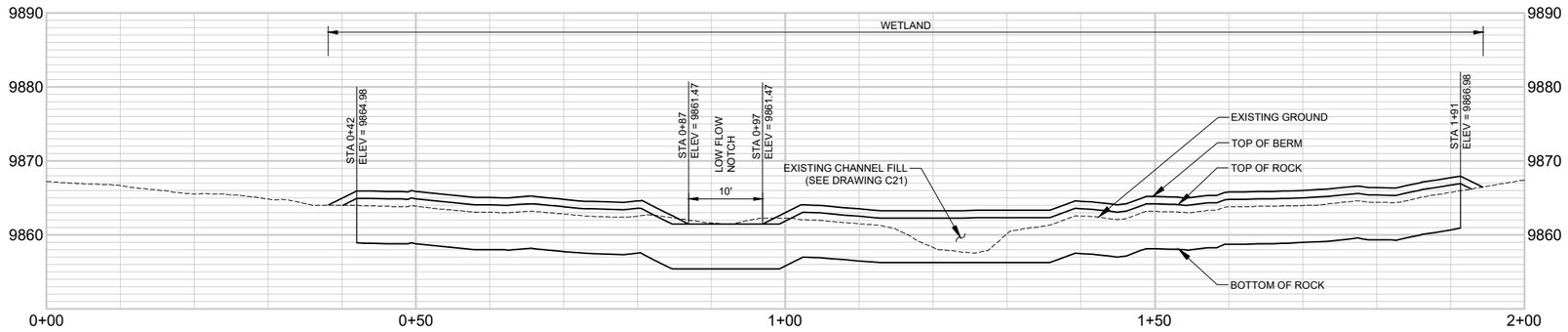
Project Mgr: CTG
Designed By: DWR
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C23

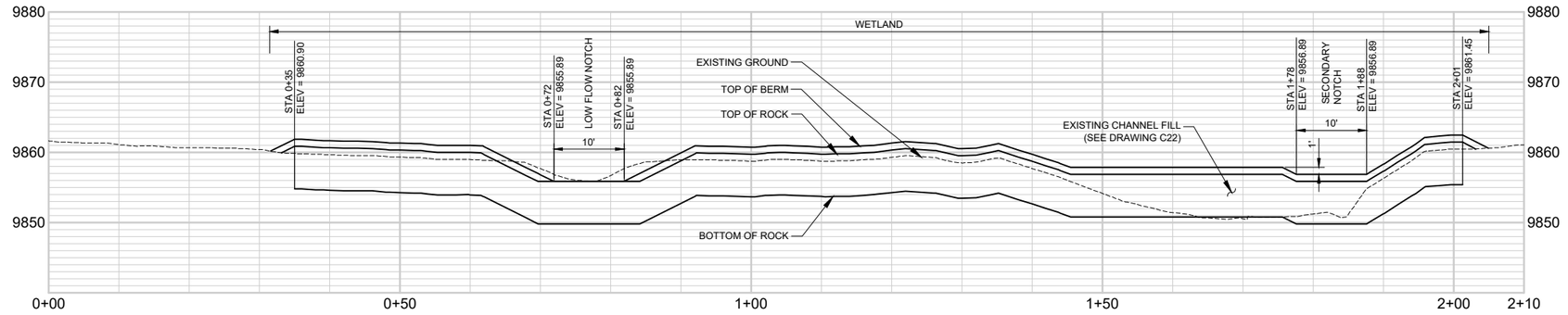
SHEET 29 OF 43

Drawing Name: I:\26\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY ROCK BERM PROFILES 2 OF 2.DWG Friday, February 26, 2021 6:48 PM By: WAYNE, ROBERT



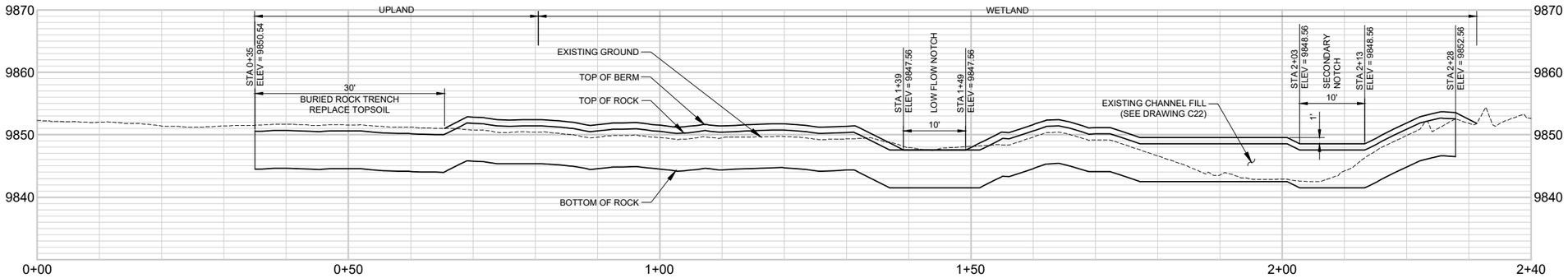
BERM 5 PROFILE

SCALE: AS SHOWN



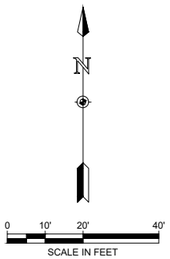
BERM 6 PROFILE

SCALE: AS SHOWN



BERM 7 PROFILE

SCALE: AS SHOWN



NOTES:

1. BERM PROFILES ARE LOOKING UPSTREAM.
2. ALL LOW FLOW NOTCH SIDE SLOPES SHALL BE 2:1.

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

SPILLWAY ROCK BERM PROFILES 2 OF 2

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date
NOT FOR CONSTRUCTION	JAN 2021

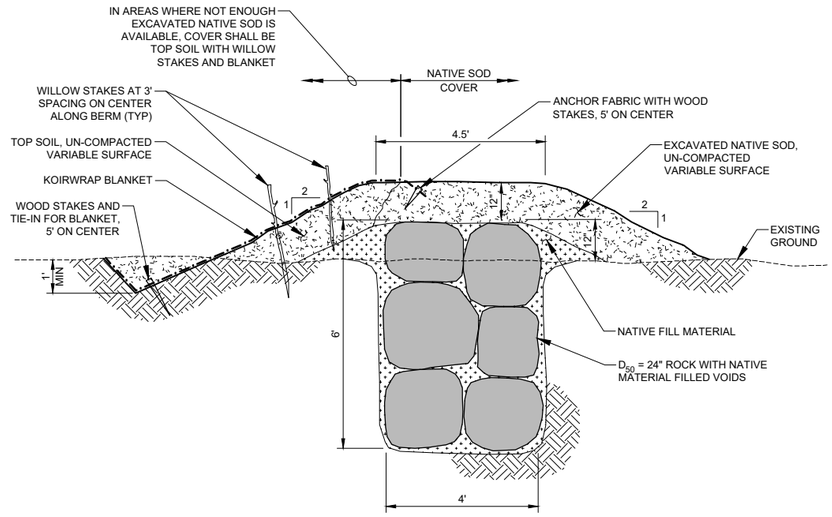
Project Mgr. CTG	
Designed By: DWR	
Drawn By: RBR	
Approved By: TMR	
Date: 02/26/21	

PROJECT NO.
26-1144.00

DRAWING NO.
C24

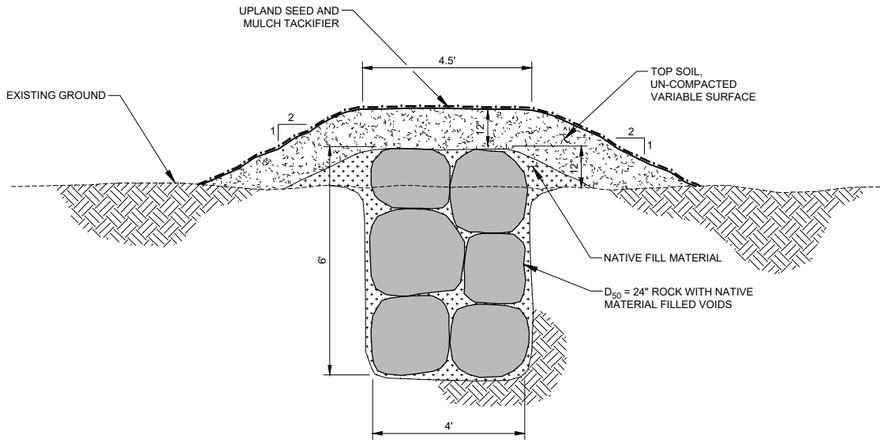
SHEET 30 OF 43

Drawing Name: I:\26\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY ROCK BERM PROFILES 2 OF 2.DWG Friday, February 26, 2021 6:48 PM By: WAYNE, ROBERT



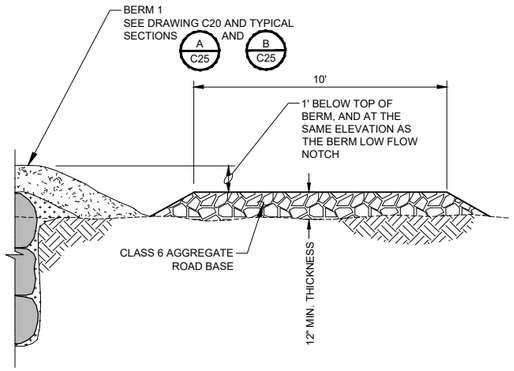
A / C25 WETLAND TYPICAL BERM SECTION

NOT TO SCALE



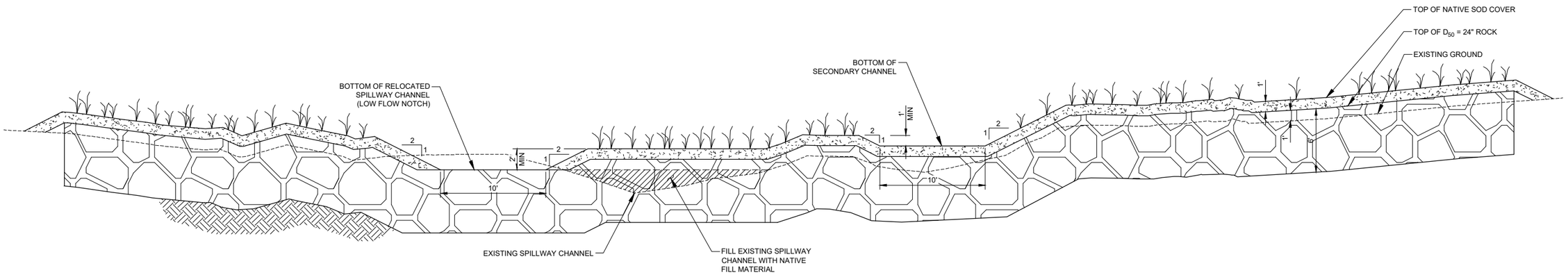
B / C25 UPLAND TYPICAL BERM SECTION

NOT TO SCALE



1 / C25 ACCESS ROAD AT BERM 1 DETAIL

NOT TO SCALE



C / C25 TYPICAL BERM PROFILE

SCALE: 1"=5'

COLORADO LICENSED
JUSTIN WAYNE ROBINSON
37109
PROFESSIONAL ENGINEER
2-26-2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

SPILLWAY DETAILS AND SECTIONS

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

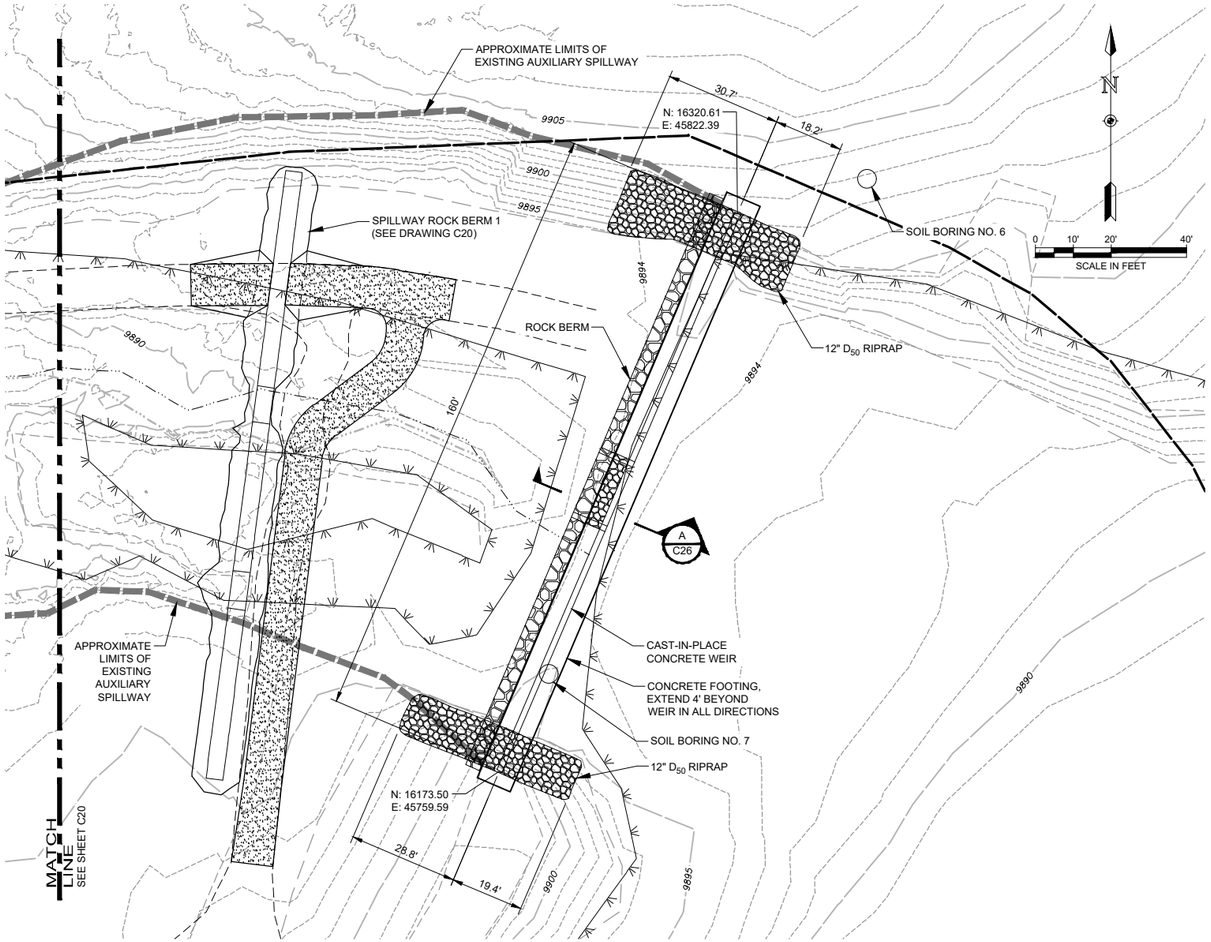
Project Mgr.	CTG
Designed By:	DWR
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C25

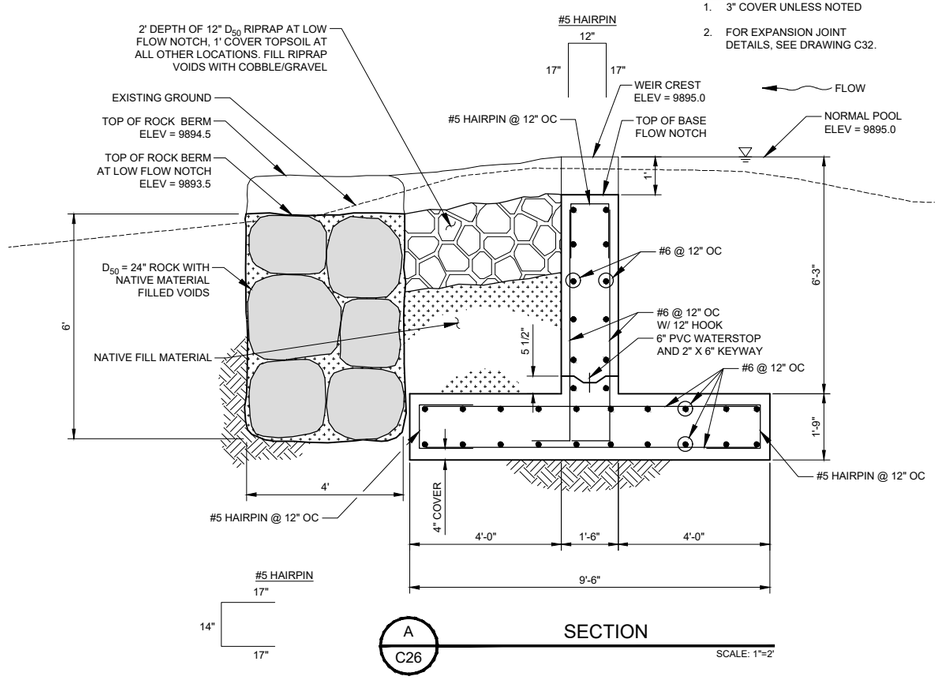
SHEET 31 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY DETAILS AND SECTIONS.DWG Friday, February 26, 2021 6:48 PM By: WAYNE, ROBERT



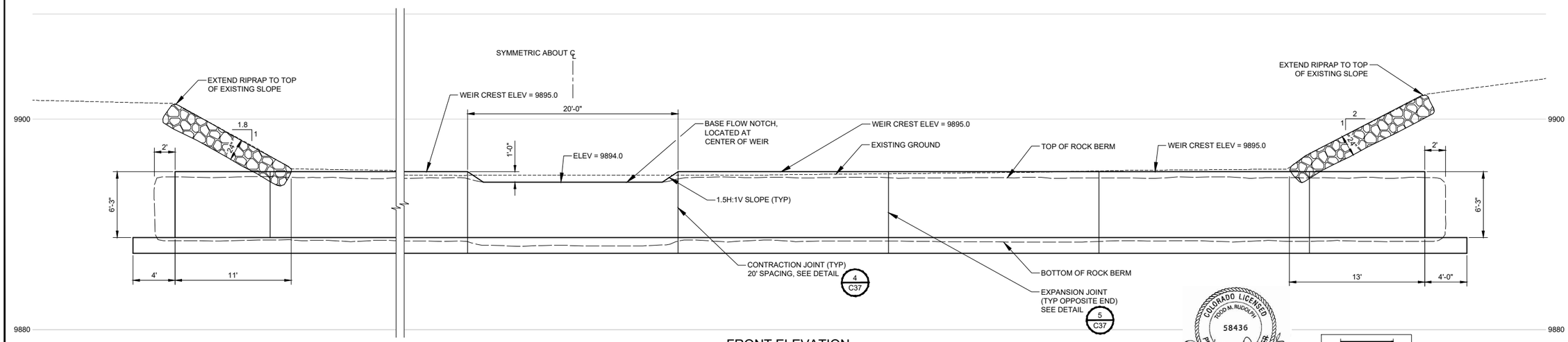
OVERFLOW WEIR PLAN

SCALE: AS SHOWN



SECTION

SCALE: 1"=2'



FRONT ELEVATION

SCALE: 1"=5'

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

SPILLWAY OVERFLOW WIER PLAN AND DETAILS

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Date	Revisions

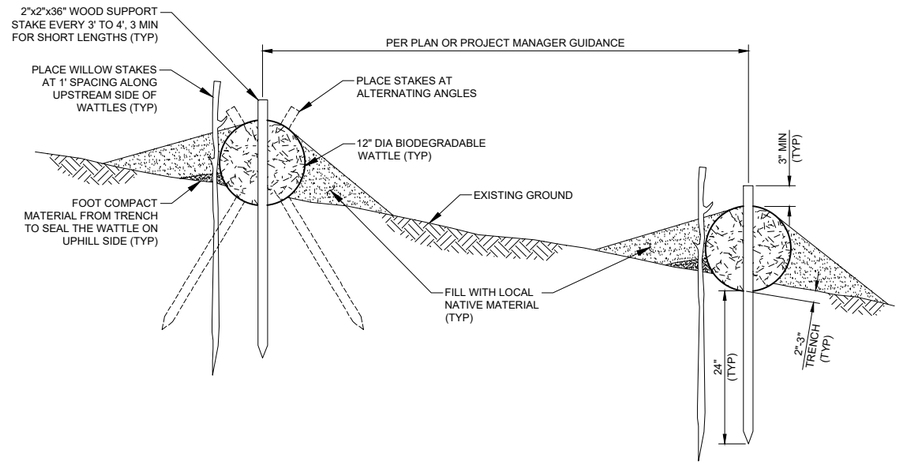
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C26

SHEET 32 OF 43

Drawing Name: I:\36\26-1144.00 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY OVERFLOW WIER PLAN AND DETAILS.DWG Friday, February 26, 2021 6:49 PM By: WAYNE, ROBERT

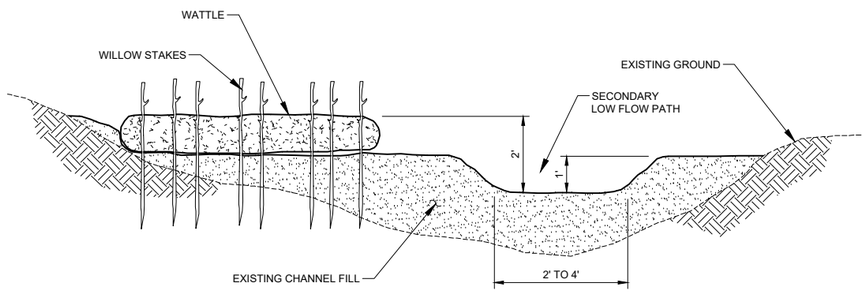


1 / C27 WATTLE DETAIL

NOT TO SCALE

NOTES:

1. WATTLES MAY ALSO BE CALLED FIBER ROLLS, SEDIMENT LOGS, OR EROSION LOGS.
2. PLACE WATTLES WORKING DOWNWARD FROM THE HIGHEST POINT.
3. INSTALL WATTLE IN A SHALLOW TRENCH AS SHOWN IN THE DETAIL. FOOT COMPACT EXCAVATED MATERIAL ON THE UPHILL SIDE OF WATTLE. WHEN JOINING ENDS OF WATTLES, FIT THE ENDS TIGHTLY TOGETHER OR OVERLAP THEM. STAKE WATTLES ON EACH END AND EVERY 3' TO 4' FEET ON CENTER.
4. USE WEED-FREE BIODEGRADABLE MATERIALS WITH A FUNCTIONAL LIFE OF AT LEAST 2 YEARS. THE DENSITY OF THE MATERIAL SHALL BE IN THE RANGE OF 2.7 TO 4.5 LBS PER CUBIC FOOT.
5. INSPECT THE WATTLES AFTER ANY STORMS (DURING CONSTRUCTION) THAT PRODUCE RUNOFF AT THE SITE. REPAIR DAMAGED WATTLES, RESTORE SEAL ON THE UPHILL SIDES, REPLACE STAKES, AND ADD MORE STAKES IF NEEDED.

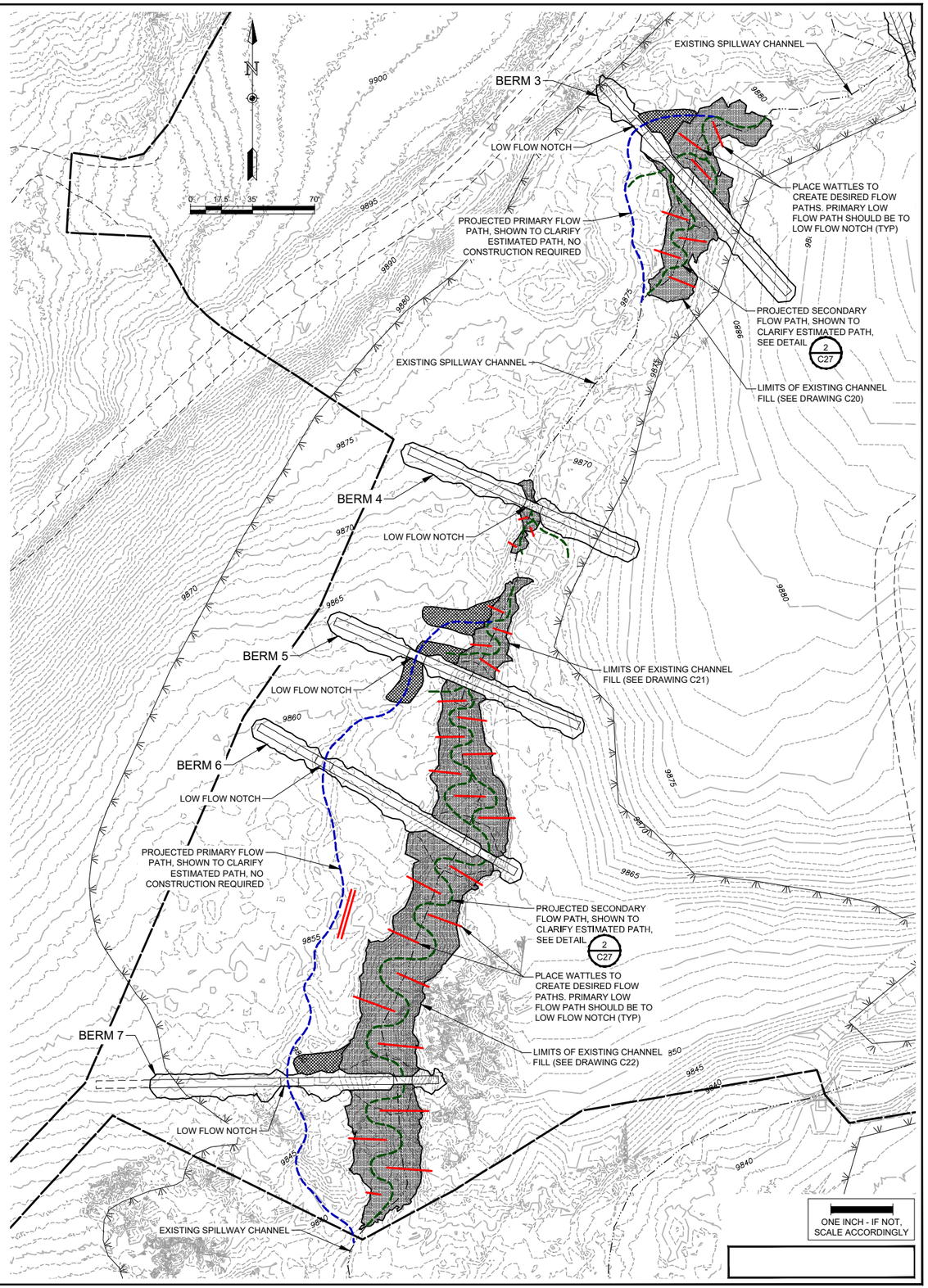


2 / C27 WATTLE, SECONDARY FLOW LOW PATH DETAIL

NOT TO SCALE

LEGEND:

- WATTLE
- PRIMARY FLOW PATH
- SECONDARY FLOW PATH
- EXISTING SPILLWAY CHANNEL



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Fort Collins, CO 80525
(970) 223-5556

SPILLWAY PRIMARY AND SECONDARY CHANNEL PLAN AND WATTLE DETAIL

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Date	Revisions

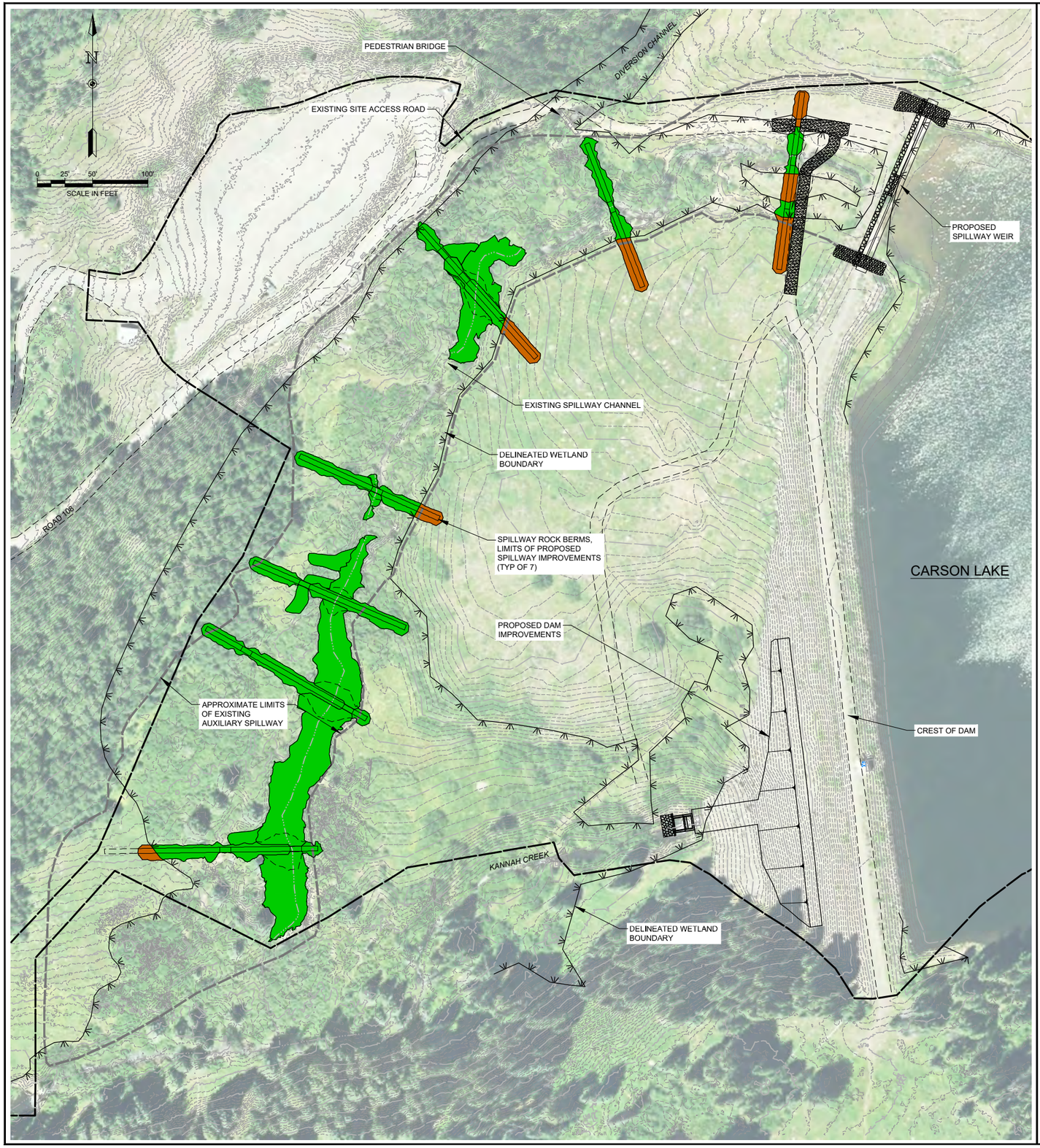
Project Mgr.	CTG
Designed By:	DWR
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO. 26-1144.00

DRAWING NO. C27

SHEET 33 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY WATTLE PLAN AND DETAILS.DWG Friday, February 26, 2021 6:49 PM By: WAYNE, ROBERT



REVEGETATION NOTES:

1. TOPSOIL WILL BE SALVAGED AND REUSED IN ALL AREAS.
2. WETLAND AREA TOPSOIL WILL BE REMOVED AS A SOD LAYER AND STORED AND WATERED ONSITE. WETLAND SOD WILL BE PLACED ON TOP OF BERMS WITHIN THE WETLAND ZONE FOLLOWING CONSTRUCTION. EXISTING WILLOWS CAN BE TRIMMED AND INCLUDED IN SOD OR EXCAVATED WITH ROOT MASS, STORED AND WATERED, AND REPLANTED ALONG BERM.
3. WETLAND LOCATIONS THAT ARE NOT ABLE TO BE COVERED IN HARVESTED WETLAND SOD WILL BE SEEDED AND PLANTED WITH WILLOW CUTTINGS.
4. THE DEPTH OF TOPSOIL SALVAGE IS APPROXIMATELY 6 INCHES, BUT MAY VARY DEPENDING ON ACTUAL SITE CONDITIONS. TOPSOIL GENERALLY INCLUDES THE UPPER PART OF THE SOIL PROFILE WITH THE FOLLOWING CHARACTERISTICS (NRCS 1993, 1999; HECKMAN 2003).
   A. WHERE PLANTS HAVE MOST OF THEIR ROOTS
   B. WHERE THE SOIL IS DARKENED FROM THE PRESENCE OF ORGANIC MATTER
   C. WHERE THERE IS A COMBINATION OF PARTICLE SIZES WITH GENERALLY NO MORE THAN APPROXIMATELY 10 PERCENT GRAVEL, 65 PERCENT SAND, 60 PERCENT SILT, OR 20 PERCENT CLAY.
5. ALL FINISH GRADES TO BE SEEDED/PLANTED WILL BE LOOSE AND ROUGH WITH SUBSTANTIAL "MICROTOPOGRAPHY" (PLUS OR MINUS 4 INCHES). STRAIGHT EDGES AND RIGHT ANGLES ARE TO BE AVOIDED. SOIL WILL BE LOOSENED TO A DEPTH OF 12 INCHES BEFORE PLANTING OR SEEDING.
6. ALL AREAS DISTURBED BY THE PROJECT WILL RECEIVE 4 INCHES OF TOPSOIL. ANY DISTURBED AREAS OUTSIDE THE PROJECT LIMITS SHALL BE REPLACED AT THE CONTRACTORS EXPENSE.
7. LIVE CUTTINGS SHOULD BE HARVESTED FROM SITES WITHIN 1,000 VERTICAL FEET OF THE PROJECT SITE ELEVATION. CUTTINGS SHALL BE 3-4 FEET IN LENGTH.
8. LIVE CUTTINGS WILL BE HARVESTED WHEN DORMANT (BEFORE LEAVES EMERGE OR AFTER THEY ARE DROPPED) FROM LIVE PLANTS 0.5 TO 1.0 INCH IN DIAMETER. THE STEM WILL BE STRIPPED OF ALL BRANCHES BEFORE CUTTING AND THEN TRIMMED TO THE DESIRED LENGTH. THE LOWER (ROOTING) END OF THE STEM WILL BE CUT AT A 45-DEGREE ANGLE AND THE UPPER END WILL BE CUT AT A 90-DEGREE ANGLE. THE LOWER END OF THE CUTTINGS WILL BE PLACED INTO COLD WATER (<50-DEGREES F) WITHIN ONE MINUTE OF CUTTING AND THEN TRANSFERRED TO A STORAGE VESSEL WITHIN 1 HOUR WHERE THEY WILL REMAIN COMPLETELY SUBMERGED FOR AT LEAST 72 HOURS, BUT NOT MORE THAN 14 DAYS, PRIOR TO PLANTING. DURING PLANTING, THE CUTTINGS WILL BE KEPT WET UNTIL PLACED INTO THE GROUND AND WILL NOT BE ALLOWED OUT OF WATER FOR MORE THAN 10 MINUTES.
9. CUTTINGS SHALL BE INSTALLED SO THAT 2/3 OF THE CUTTING ARE BELOW GRADE.
10. NO EQUIPMENT SHALL BE ALLOWED IN THE RESTORATION AREA AFTER THE SOIL IS LOOSENED.
11. ALL BLANKETED AREAS WILL BE LOOSENED, SEEDED, AND MULCHED ACCORDING TO THE REVEGETATION PLAN PRIOR TO BLANKET OR MULCH INSTALLATION.
12. SEEDING WILL ONLY BE PERFORMED BETWEEN SEPTEMBER 1 AND WHEN THE GROUND FREEZES, OR WITHIN 30 DAYS AFTER GROUND THAWS.
13. SEEDED AREAS WILL BE HYDRO MULCHED TO ACHIEVE APPROXIMATELY 80 PERCENT GROUND COVER.
14. WETLAND SEED MIX SHALL BE MONTANE WET MEADOW MIX BY WESTERN NATIVE SEED (COALDALE, CO, 719-942-3935, westernnativeseed.com), OR APPROVED EQUAL.
15. UPLAND SEED MIXTURE SHALL BE HIGH ALTITUDE NATIVE GRASS MIX BY BEAUTY BEYOND BELIEF WILDFLOWER SEED (BOULDER, COLORADO, 303.530.1222, www.bbbseed.com), OR APPROVED EQUAL.
16. WILLOW CUTTINGS SHALL BE INSTALLED WHEN LOCAL WILLOWS ARE DORMANT. CONTAINER STOCK MAY BE SUBSTITUTED FOR CUTTINGS. WILLOWS WILL BE COMPOSED OF THE FOLLOWING SPECIES, WITH EACH SPECIES EVENLY DISTRIBUTED THROUGHOUT THE PLANTING AREAS:
    - SANDBAR WILLOW (SALIX EXIGUA) - 60%
    - BEBB WILLOW (SALIX BEBBIANA) - 20%
    - DRUMMOND WILLOW (SALIX DRUMMONDIANA) - 20%

LEGEND:

- WETLAND PLANTING
- UPLAND PLANTING

COLORADO LICENSED 37109 JUSTIN WAYNE ROBINSON PROFESSIONAL ENGINEER
2-26-2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

SPILLWAY PLANTING PLAN

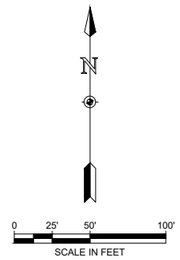
HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Date	Revisions

Project Mgr. CTG
Designed By: DWR
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO. 26-1144.00
DRAWING NO. C28
SHEET 34 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 SPILLWAY PLANTING PLAN.DWG Friday, February 26, 2021 6:49 PM By: WAYNE, ROBERT



NOTES:

COORDINATE WITH EARLY WARNING SYSTEM VENDOR TO CONFIRM LOCATION AND DESIGN OF REQUIRED APPURTENANCES.

PRIMARY STATION COMPONENTS:
1) BULLET-RESISTANCE ENCLOSURE
2) BUBBLER WATER LEVEL SENSOR
3) PRESSURE TRANSDUCER
4) TIPPING BUCKET
5) DATA LOGGER
6) SOLAR PANEL
7) BATTERY (2)
8) REGULATOR
9) INTRUSION SENSOR

SECONDARY STATION COMPONENTS:
1) BULLET-RESISTANCE ENCLOSURE
2) PRESSURE TRANSDUCER
3) DATA LOGGER WITH SPREAD-SPECTRUM WIRELESS CONNECTION TO PRIMARY STATION
4) SOLAR PANEL
5) BATTERY (2)
6) REGULATOR
7) INTRUSION SENSOR

LEGEND:

EXISTING PIEZOMETERS (DO NOT DISTURB)

SURVEY MONUMENTS (DO NOT DISTURB)

COLORADO LICENSED
TODD M. RUDD, P.E.
58436
PROFESSIONAL ENGINEER
2/26/2021

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DAM INSTRUMENTATION

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

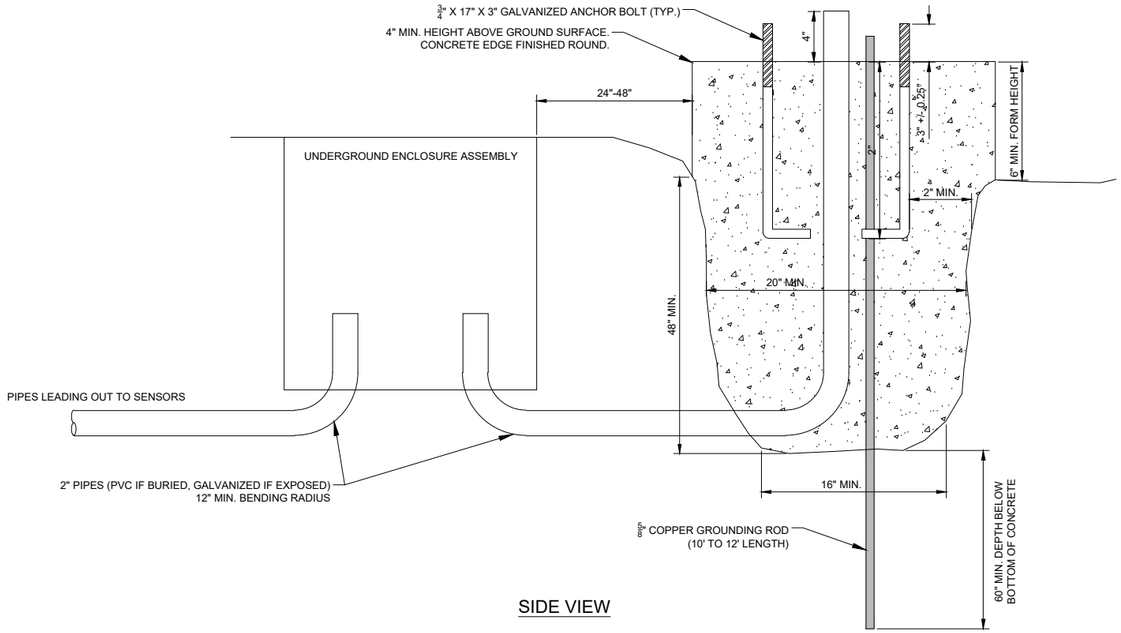
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

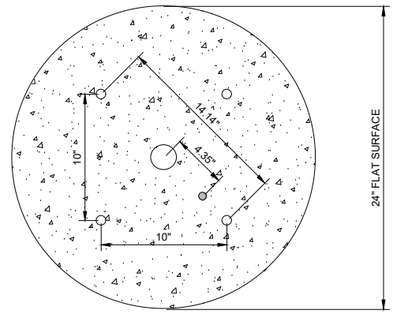
DRAWING NO.
C29

SHEET 35 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM INSTRUMENTATION.DWG Friday, February 26, 2021 6:50 PM By: WAYNE, ROBERT



SIDE VIEW

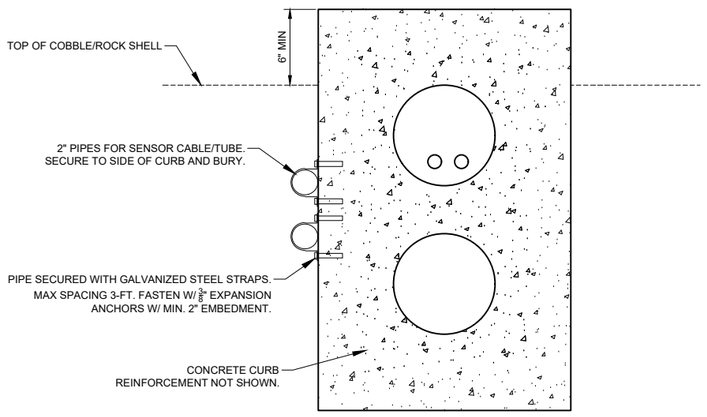


TOP VIEW

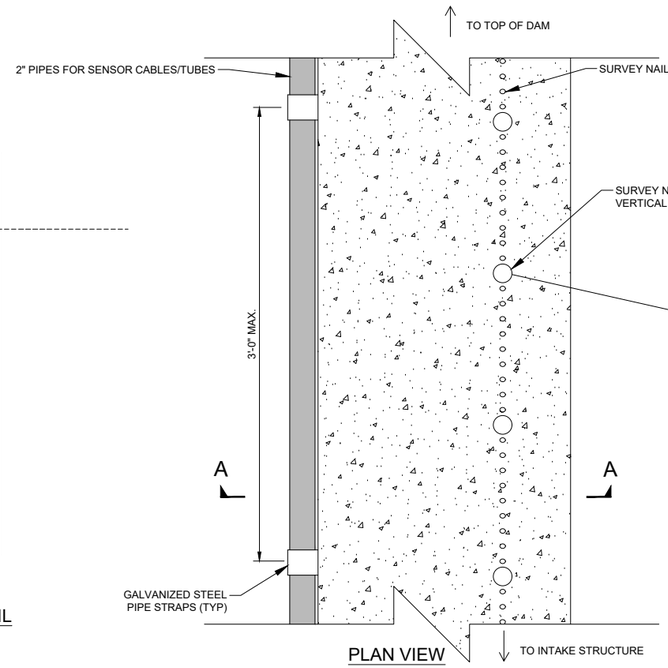
INSTRUMENTATION
FOUNDATION PRIMARY &
SECONDARY STATIONS DETAIL

1 / C28

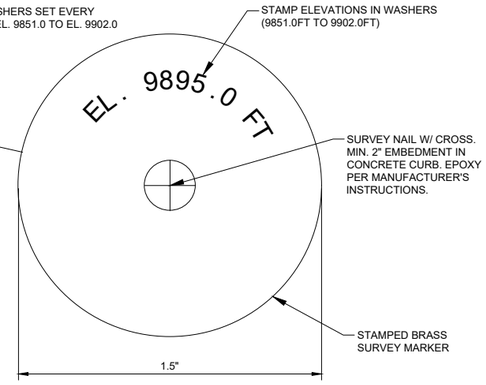
NOT TO SCALE



SECTION A-A: OUTER CURB DETAIL



PLAN VIEW



STAFF GAGE FOOT-MARKER DETAIL

2 / C28

STAFF GAGE & PIPE RUN DETAIL

NOT TO SCALE

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TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

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(970) 223-5556

INSTRUMENTATION DETAILS

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

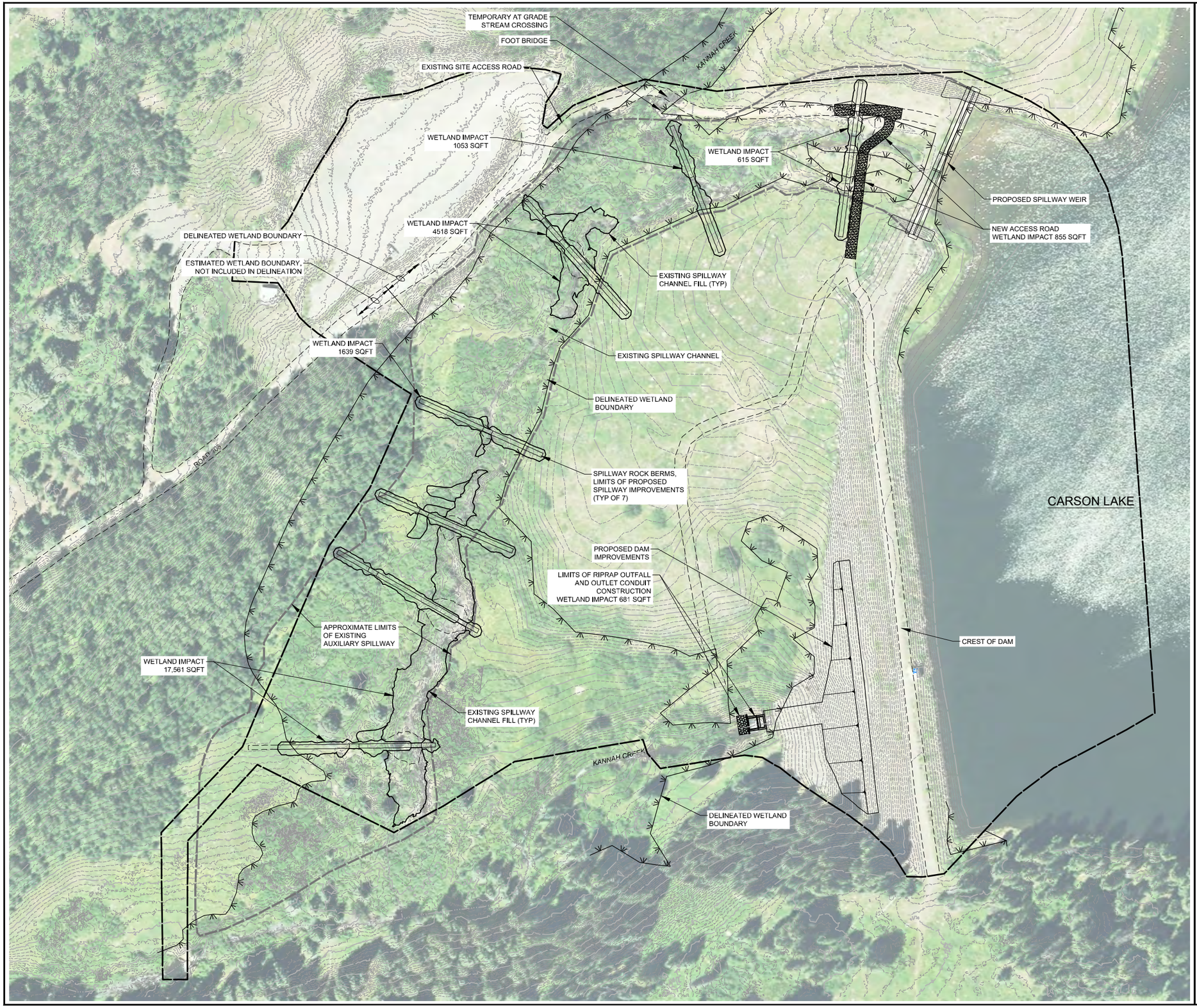
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

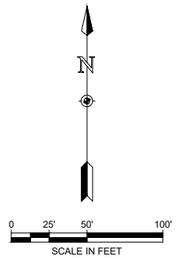
PROJECT NO.
26-1144.00

DRAWING NO.
C30

SHEET 36 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 DAM INSTRUMENTATION.DWG Friday, February 26, 2021 6:50 PM By: WAYNE, ROBERT





NOTES:

1. THE TOTAL ESTIMATED WETLAND IMPACT OF THE SPILLWAY CHANNEL CONSTRUCTION AND THE OUTFALL RIPRAP BASIN IS 26,922 SQFT (0.62 ACRES).

LEGEND:

LIMITS OF WETLAND BOUNDARY

APPROXIMATE LIMITS OF EXISTING AUXILIARY SPILLWAY

COLORADO LICENSED
TODD M. RUDOLPH
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PROFESSIONAL ENGINEER
2/26/2021

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WETLAND IMPACT

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr. CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO.
26-1144.00

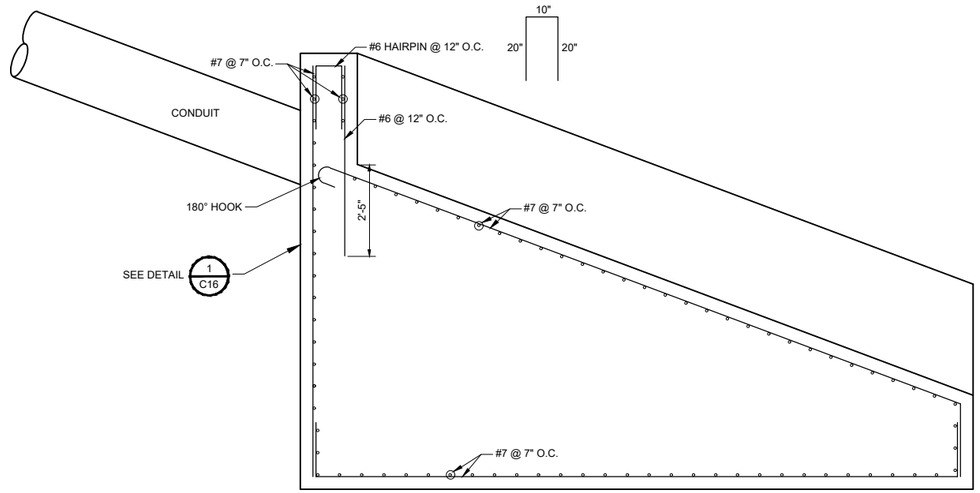
DRAWING NO.
C31

SHEET 37 OF 43

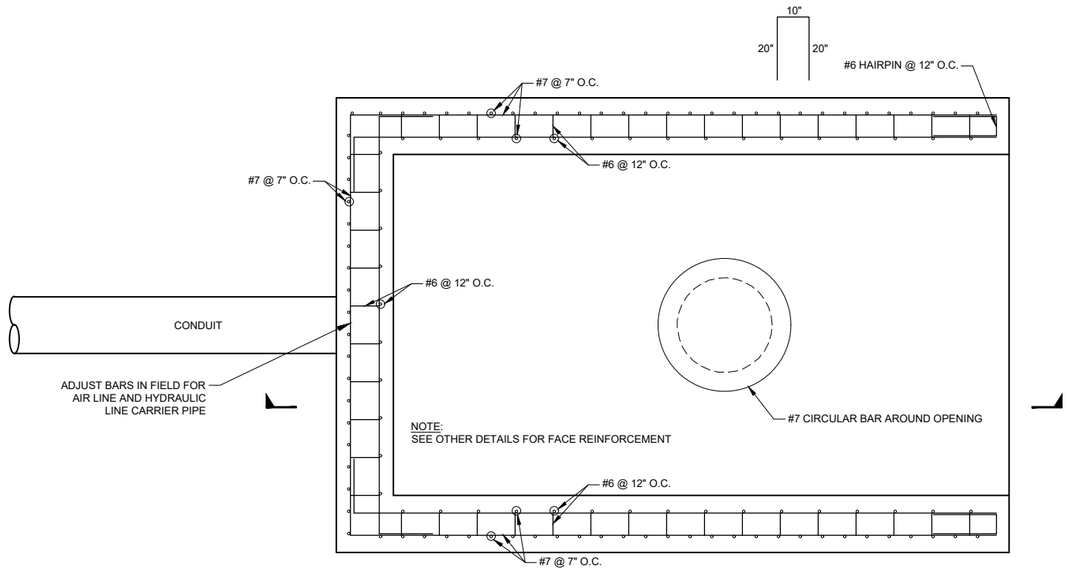
Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 WETLAND IMPACT.DWG Friday, February 26, 2021 6:50 PM By: WAYNE, ROBERT

NOTES:

1. 4" COVER UNLESS NOTES.
2. ¾" CHAMFER ALL EXPOSED CONCRETE EDGES (TYP).



SECTION



PLAN

1 / C32 CONCRETE BUTTRESS DETAIL

SCALE: 1/2"=1'-0"

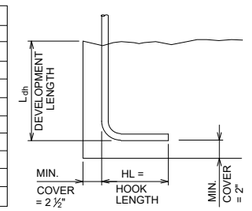
MINIMUM DEVELOPMENT AND SPLICE LENGTHS				
f'_c = 4500 psi clear spacing $\geq 2d_b$ & clear cover $\geq d_b$				
Bar Size Grade 60	Minimum Straight Tension Development Length, l_d		Minimum Lap Splice For Center To Center Bar Spacing	
	TOP BARS	OTHER BARS	TOP BARS	OTHER BARS
#3	1'-6"	1'-2"	1'-11"	1'-6"
#4	2'-0"	1'-6"	2'-7"	2'-0"
#5	2'-6"	1'-11"	3'-2"	2'-6"
#6	2'-11"	2'-3"	3'-10"	2'-11"
#7	4'-3"	3'-4"	5'-7"	4'-3"
#8	4'-11"	3'-9"	6'-4"	4'-11"
#9	5'-6"	4'-3"	7'-2"	5'-6"
#10	6'-2"	4'-9"	8'-0"	6'-2"
#11	6'-10"	5'-4"	8'-11"	6'-10"

NOTES:

1. TOP BARS ARE HORIZONTAL BARS SO PLACED THAT MORE THAN 12" OF CONCRETE IS CAST IN THE MEMBER BELOW THE BAR.
2. EXCEPT AS OTHERWISE INDICATED ON THE DRAWINGS, LAP SPLICE LENGTH AND TENSION DEVELOPMENT l_d SHALL BE NO LESS THAN (NO MINUS TOLERANCE) SHOWN ABOVE.
3. LAP SPLICES SHALL NOT BE MADE AT POINTS OF MAXIMUM STRESS AS DETERMINED BY THE ENGINEER.
4. WHERE BARS OF DIFFERENT SIZE ARE LAP SPLICED, SPLICE LENGTH SHALL BE THE SPLICE LENGTH OF THE LARGER BAR.
5. ALL SPLICES SHALL BE CONTACT SPLICES AND WIRED TOGETHER.

MINIMUM DEVELOPMENT & SPLICE LENGTH FOR REBARS

BAR SIZE GRADE 60	f'_c = 4500 psi	
	L_{dh}	HL
#3	6"	6"
#4	7"	8"
#5	8"	10"
#6	10"	1'-0"
#7	11"	1'-2"
#8	1'-1"	1'-4"
#9	1'-3"	1'-8"
#10	1'-4"	1'-10"
#11	1'-6"	2'-0"



COMPLYING WITH MINIMUM COVER REQUIREMENTS OF ACI 318, OTHERWISE L_{dh} MUST BE RE-CALCULATED

TYPICAL 90 DEGREE HOOK FOR REBAR

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER 2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
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(970) 223-5556

CONCRETE BUTTRESS REINFORCEMENT

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

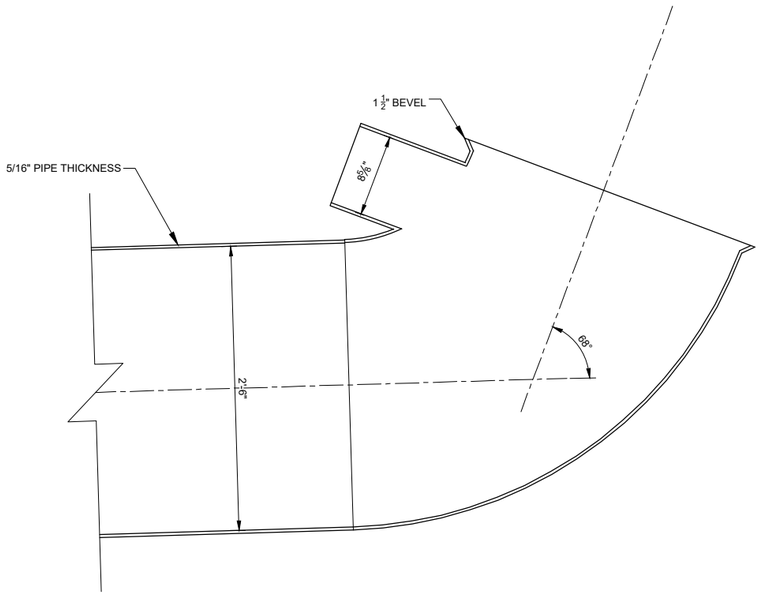
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO. 26-1144.00

DRAWING NO. C32

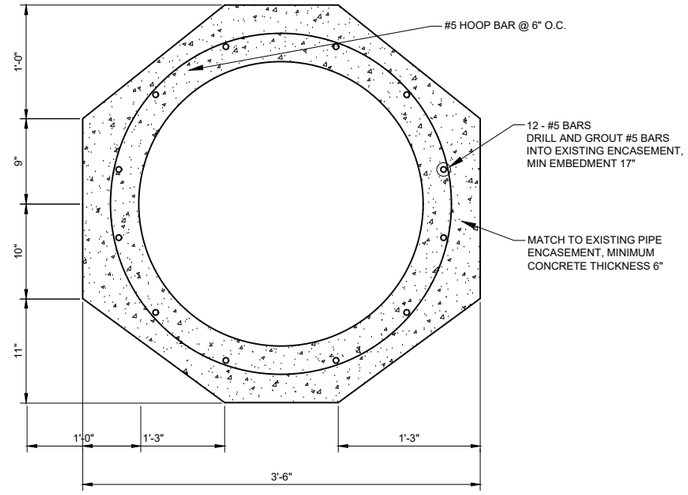
SHEET 38 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 IMPACT BASIN DETAILS.DWG Friday, February 26, 2021 6:51 PM By: WAYNE, ROBERT



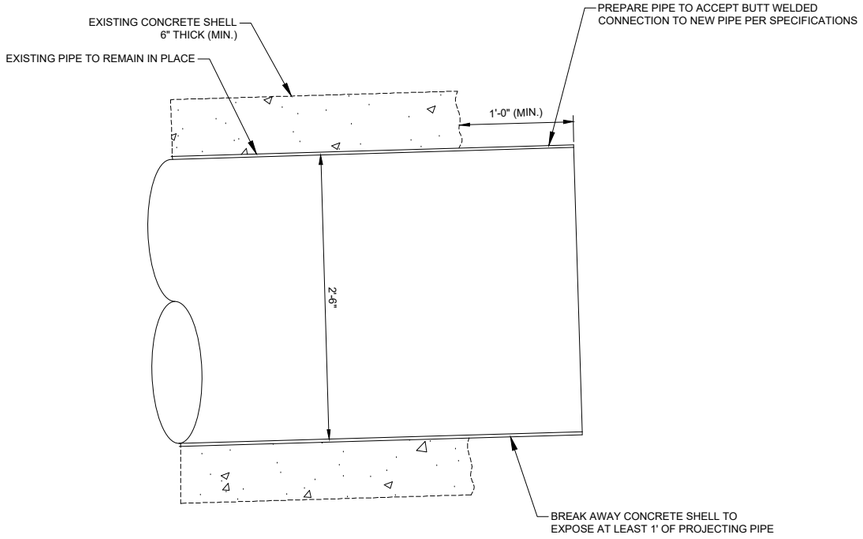
1 C33 STEEL ELBOW DETAIL

SCALE: 1"=1'-0"



3 C33 STEEL PIPE ENCASEMENT DETAIL

SCALE: 1"=1'-0"



2 C33 STEEL PIPE CONNECTION DETAIL

SCALE: 1"=1'-0"

COLORADO LICENSED
TODD M. RUDOLPH
58436
PROFESSIONAL ENGINEER
2/26/2021

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(970) 223-5556

STEEL PIPE DETAILS

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

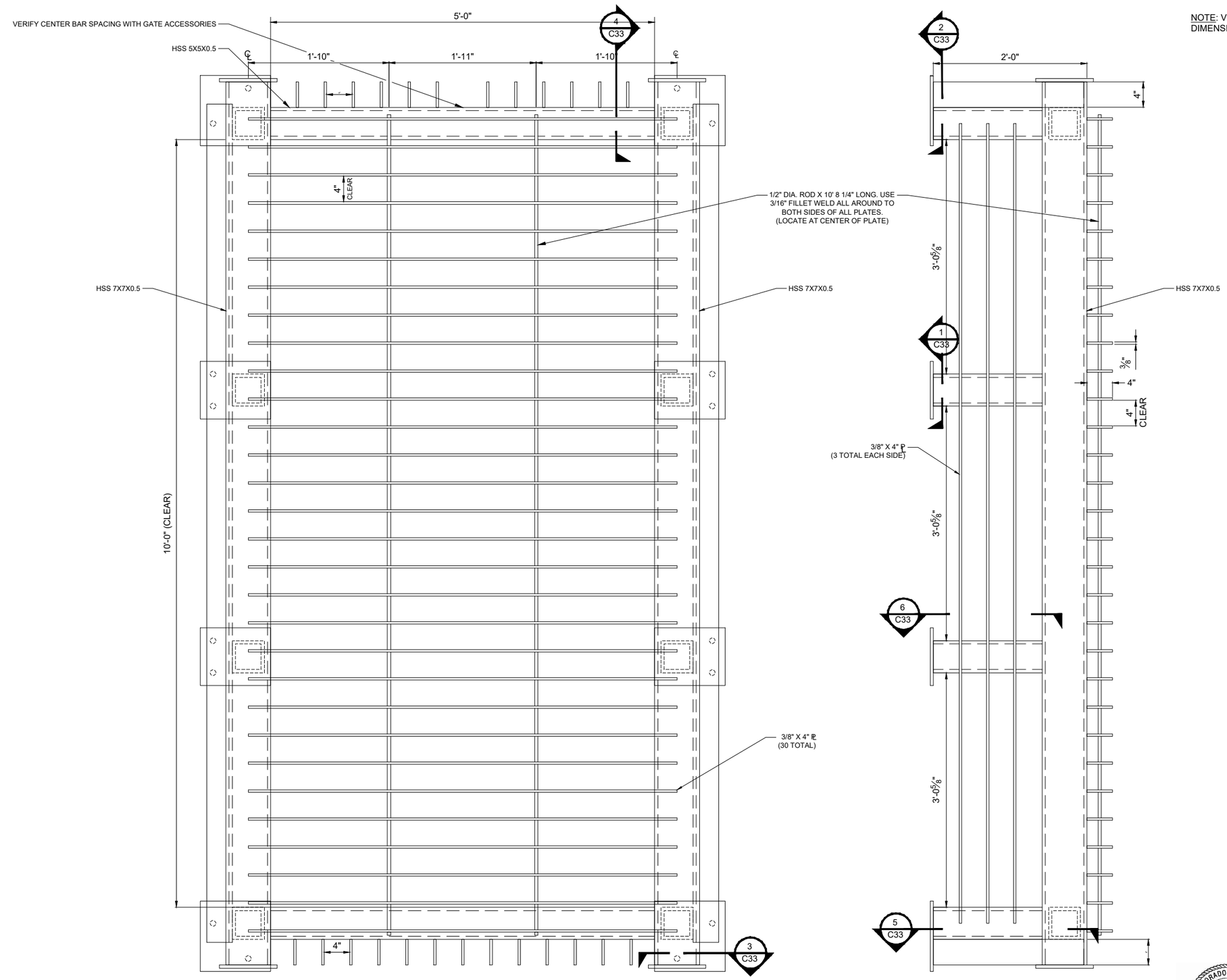
Revisions	Date

Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO.
26-1144.00

DRAWING NO.
C33

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 IMPACT BASIN DETAILS.DWG Friday, February 26, 2021 6:51 PM By: WAYNE, ROBERT



NOTE: VERIFY ALL FRAME DIMENSIONS WITH GATE DIMENSIONS DURING SHOP DRAWING PREPARATION.

1 / C32 SLIDE GATE TRASH RACK

SCALE: 1 1/2"=1'-0"

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER 2/26/2021

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TRASH RACK DETAILS

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

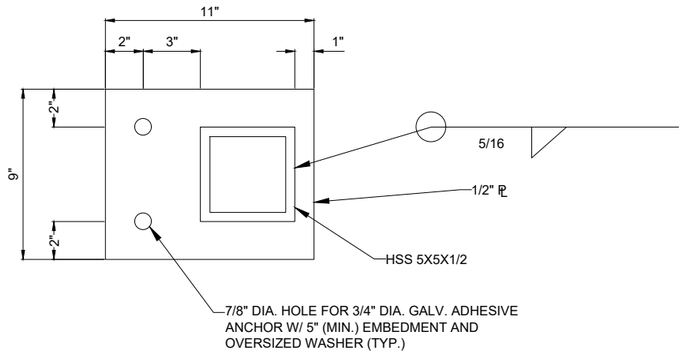
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO. 26-1144.00

DRAWING NO. C34

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 IMPACT BASIN DETAILS.DWG Friday, February 26, 2021 6:51 PM By: WAYNE, ROBERT

NOTE: VERIFY ALL FRAME DIMENSIONS WITH GATE DIMENSIONS DURING SHOP DRAWING PREPARATION.



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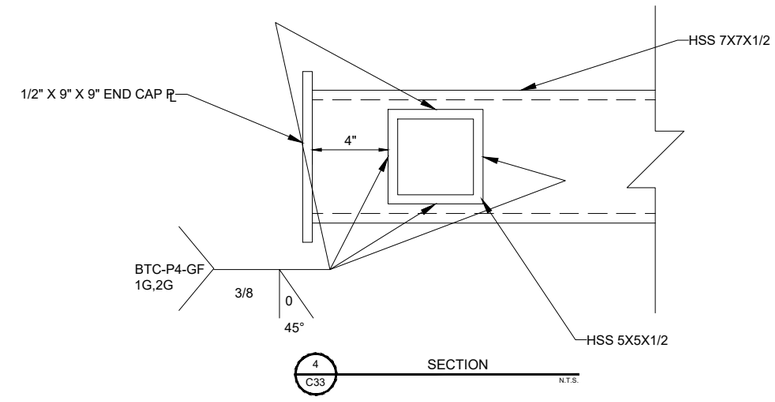
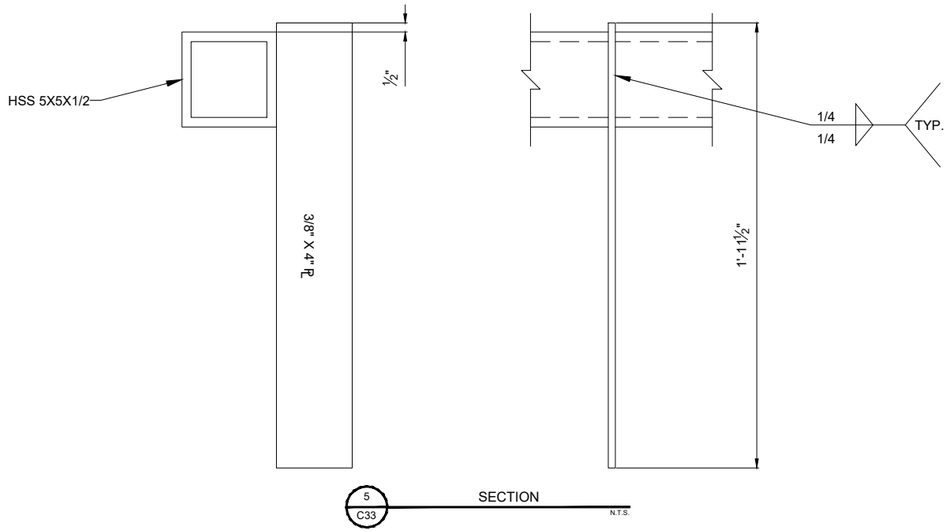
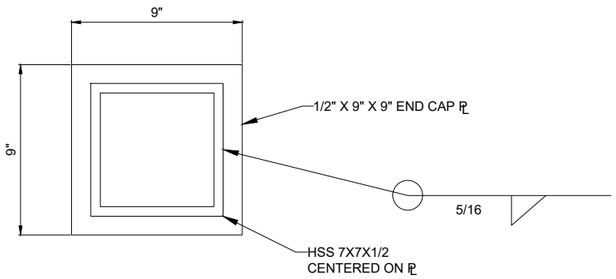


4 C33 SECTION N.T.S.



5 C33 SECTION N.T.S.

2 C33 SECTION N.T.S.



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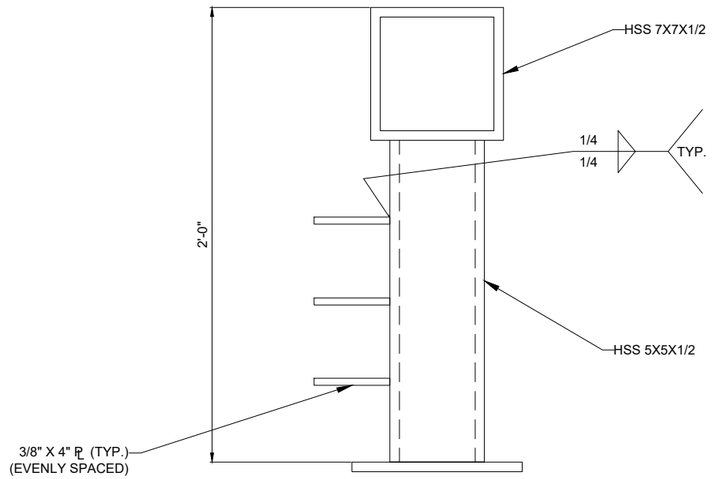

6 C33 SECTION N.T.S.

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3665 JFK Parkway
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TRASH RACK DETAILS

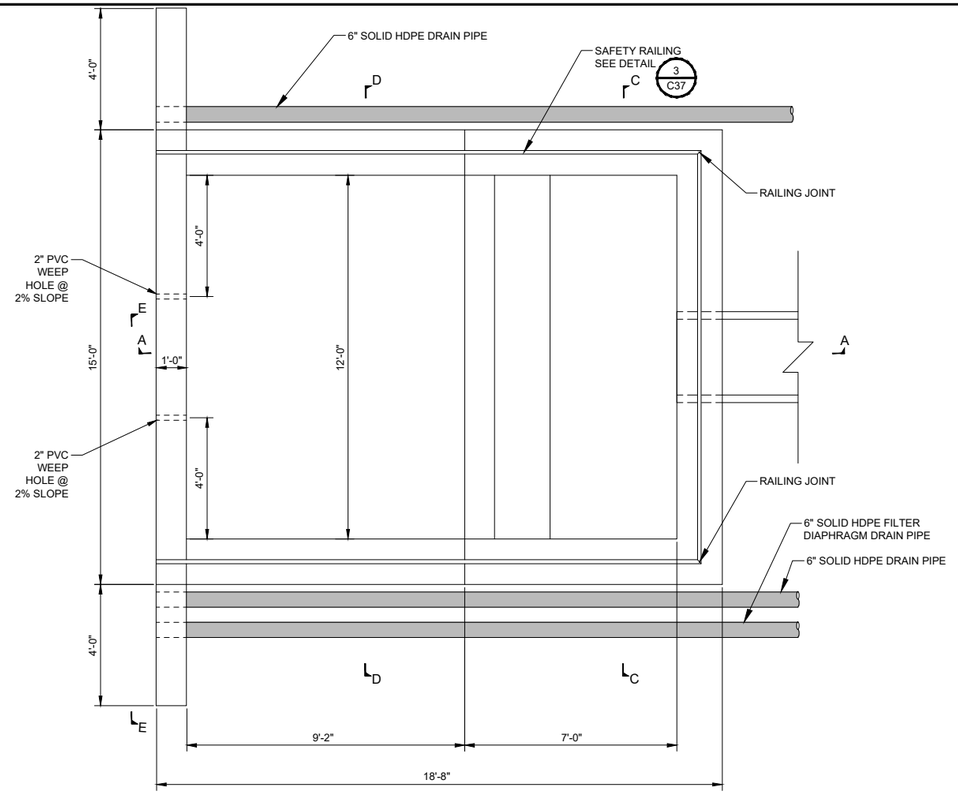
HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

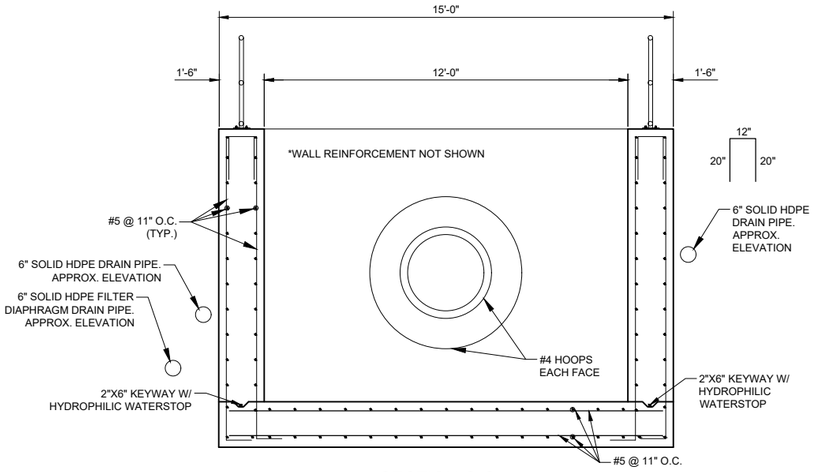
Project Mgr. CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO. 26-1144.00
DRAWING NO. C35
SHEET 41 OF 43

Drawing Name: I:\26\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 IMPACT BASIN DETAILS.DWG Friday, February 26, 2021 6:51 PM By: WAYNE, ROBERT



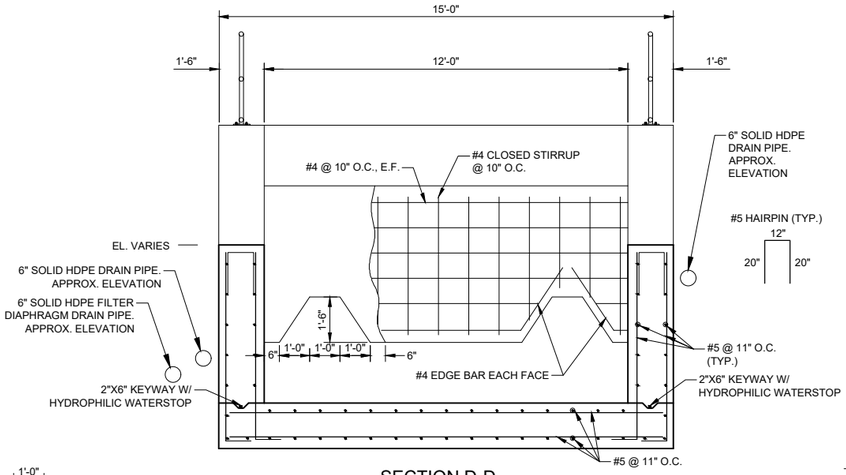
PLAN



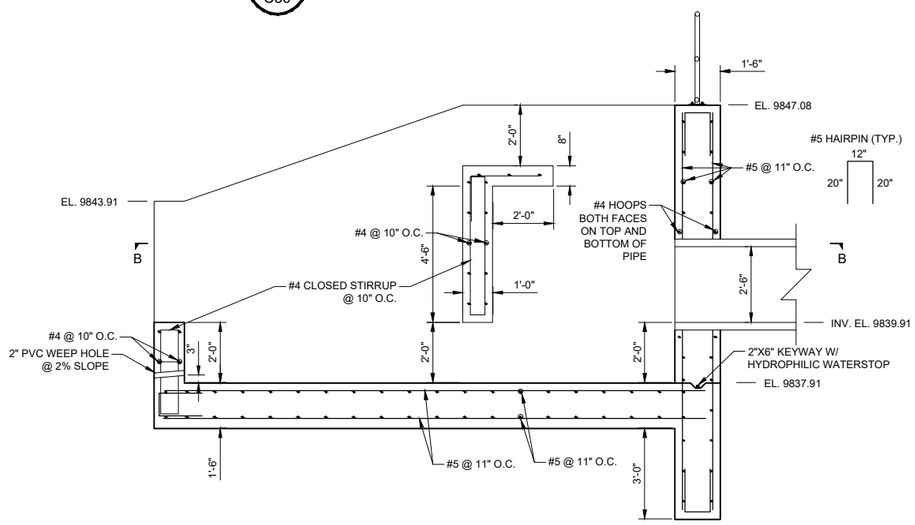
SECTION C-C

NOTES:

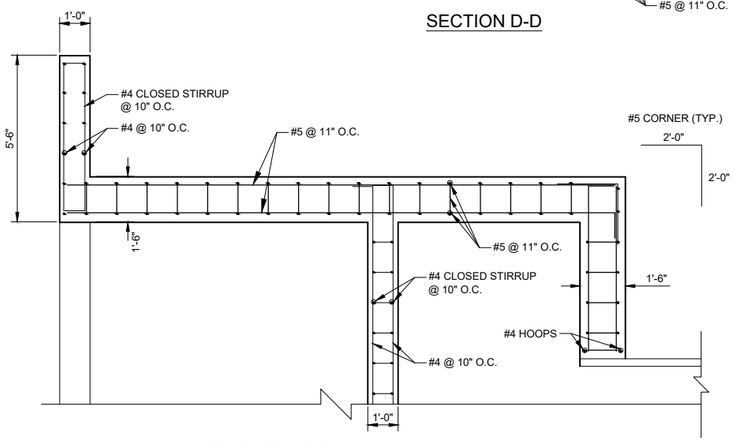
1. 3" COVER ON 1'-6" THICK SECTIONS.
2. 2" COVER ON 1'-0" THICK SECTIONS.
3. PLACE REINFORCEMENT IN CENTER FOR SECTIONS LESS THAN 1'-0".
4. ¾" CHAMFER ALL EXPOSED CONCRETE EDGES (TYP).
5. SEE SHEET C29 FOR REINFORCEMENT DEVELOPMENT, SPLICE, AND HOOK LENGTHS.



SECTION D-D



SECTION A-A



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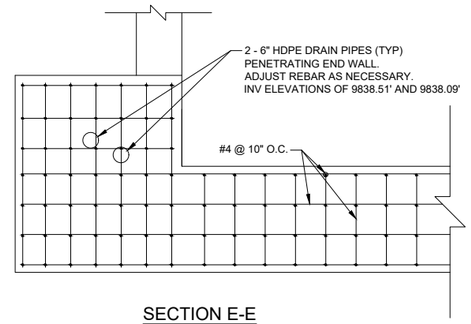


SECTION E-E

1 C36 IMPACT BASIN DETAIL

NOT TO SCALE

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER

2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

IMPACT BASIN DETAILS

HOGCHUTE (AKA CARSON)
RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

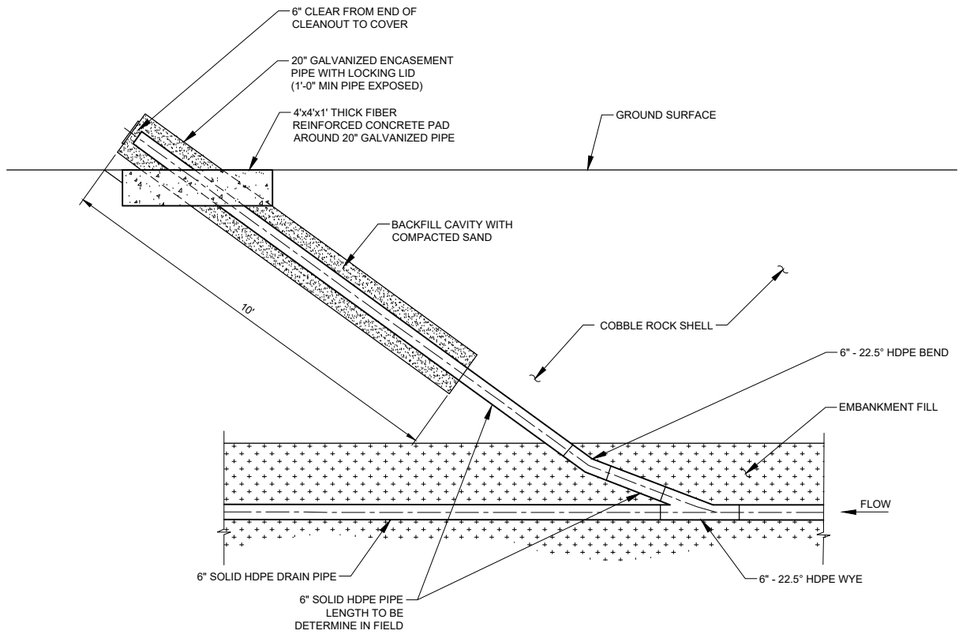
Project Mgr.	CTG
Designed By:	AJS
Drawn By:	RBR
Approved By:	TMR
Date:	02/26/21

PROJECT NO. 26-1144.00

DRAWING NO. C36

SHEET 42 OF 43

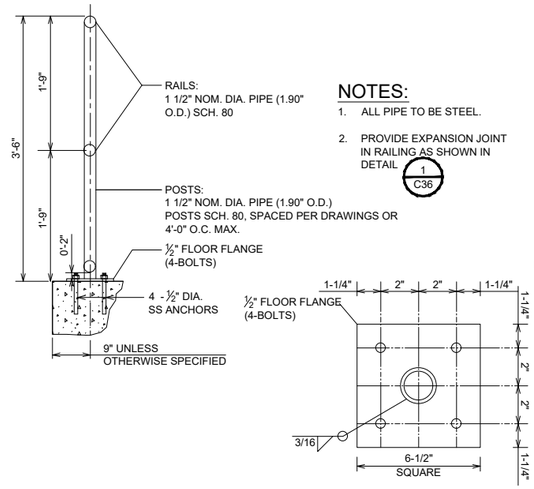
Drawing Name: i:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\26-1144.00 IMPACT BASIN DETAILS.DWG Friday, February 26, 2021 6:51 PM By: WAYNE, ROBERT



HDPE DRAIN PIPE LATERAL CLEANOUT DETAIL

1 / C37

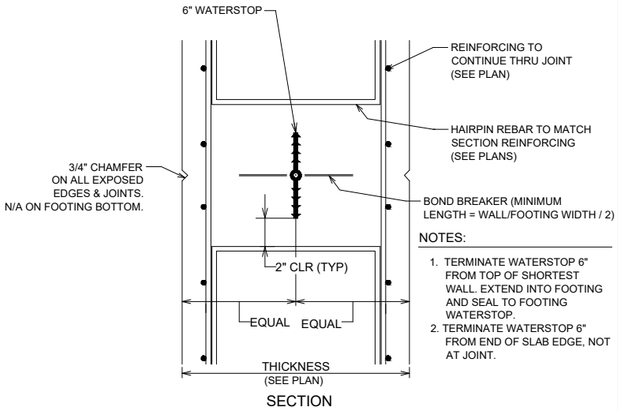
NOT TO SCALE



SAFETY RAILING DETAIL

3 / C37

NOT TO SCALE



SECTION

WALL/FOOTING CONTRACTION JOINT DETAIL

4 / C37

(REINFORCEMENT PENETRATES JOINT)

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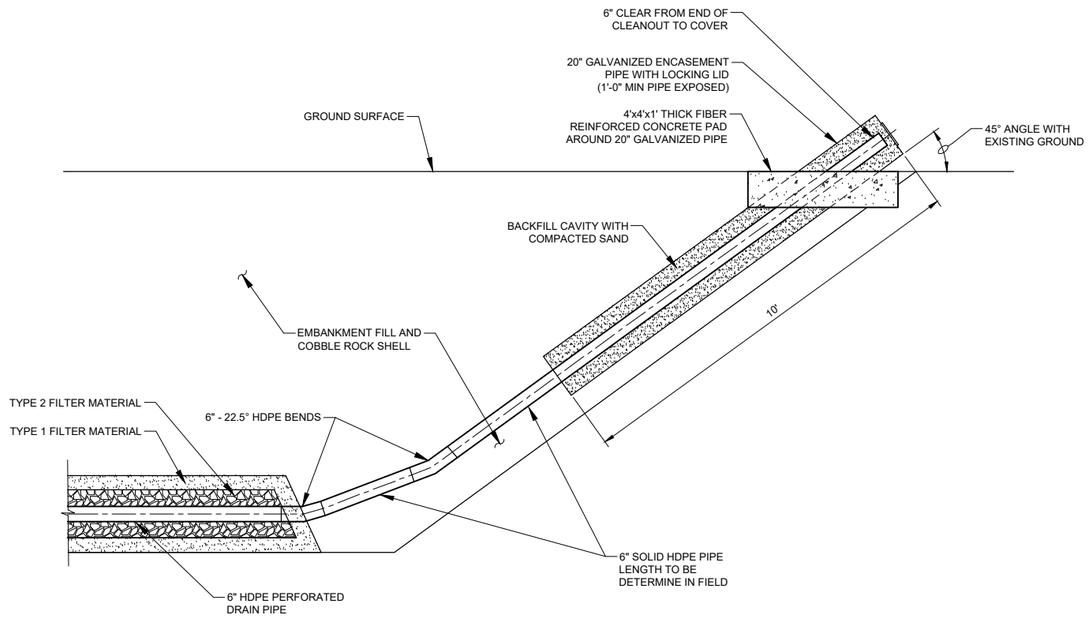
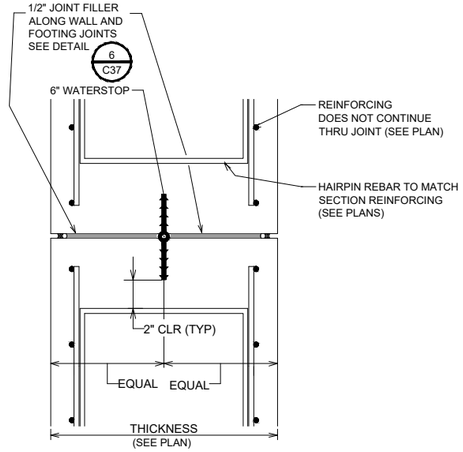


HDPE DRAIN PIPE END ACCESS DETAIL

2 / C37

NOT TO SCALE



TOP/EDGE VIEW

WALL/FOOTING EXPANSION JOINT DETAIL

5 / C37

(REINFORCEMENT DOES NOT PENETRATE JOINT)

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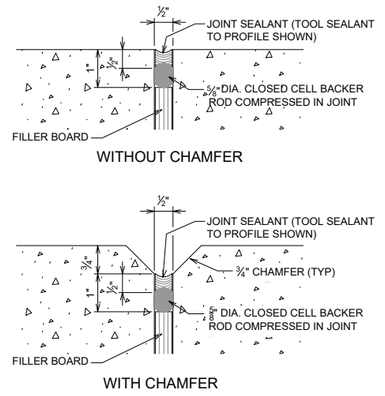


EXPANSION JOINT SEALANT DETAIL

6 / C37

N.T.S.

COLORADO LICENSED TODD M. RUDOLPH 58436 PROFESSIONAL ENGINEER

2/26/2021

ONE INCH - IF NOT, SCALE ACCORDINGLY

AYRES
3665 JFK Parkway
Building 2, Suite 100
Fort Collins, CO 80525
(970) 223-5556

MISCELLANEOUS DETAILS

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION
CITY OF GRAND JUNCTION
MESA COUNTY, COLORADO

Revisions	Date

Project Mgr.: CTG
Designed By: AJS
Drawn By: RBR
Approved By: TMR
Date: 02/26/21

PROJECT NO. 26-1144.00

DRAWING NO. C37

SHEET 43 OF 43

Drawing Name: I:\36\26-114400 GRAND JUNCTION HOGCHUTE DAM\MAINDWGS 2020\SHEETS\C35 TOE DRAIN DETAILS.DWG Friday, February 26, 2021 6:51 PM By: WAYNE, ROBERT

PROJECT MANUAL

FOR

HOGCHUTE (AKA CARSON) RESERVOIR DAM REHABILITATION

DAMID 420127

MESA COUNTY, CO

FEBRUARY 2021

Owned by City of Grand Junction, CO

AYRES

3433 Oakwood Hills Parkway
Eau Claire, WI 54701-7698
715.834.3161 • Fax: 715.831.7500
www.AyresAssociates.com

Ayres Project No. 26-1144.00

HOGCHUTE DAM REPAIRS
DAMID 420127
MESA COUNTY, CO

TABLE OF CONTENTS

Title		No. of Pages
Project Title Page		1
Table of Contents		1
SPECIFICATIONS		
DIVISION 01 - GENERAL REQUIREMENTS		
01 01 00	General Requirements	5
01 22 50	Measurement and Payment	5
01 33 00	Submittal Procedures	3
01 57 19	Temporary Environmental Controls	4
01 57 60	Construction Dewatering	4
DIVISION 02 - EXISTING CONDITIONS		
02 41 00	Demolition	3
DIVISION 03 - CONCRETE		
03 30 00	Cast-In-Place Concrete	9
03 62 00	Non-Shrink Grouting	2
DIVISION 05 - METALS		
05 50 00	Metal Fabrications	3
05 52 04	Steel Railings	2
DIVISION 09 - FINISHES		
09 90 00	Painting and Coating	3
DIVISION 26 - ELECTRICAL		
26 05 43	Underground Conduit and Pull Boxes	2
DIVISION 31 - EARTHWORK		
31 05 10	Site Preparation	2
31 20 00	Earth Moving	7
31 23 33	Trenching and Backfilling	4
31 24 00	Embankments	4
31 37 00	Riprap	2
DIVISION 32 - EXTERIOR IMPROVEMENTS		
32 92 36	Native Species Seeding	3
DIVISION 33 - UTILITIES		
33 01 36	Cured-in-Place Pipe Lining	2
33 41 66	Drain Fill	3
33 42 15	Piping and Accessories	4
DIVISION 35 - WATERWAY AND MARINE CONSTRUCTION		
35 22 28	Stainless Steel Slide Gates	3
DIVISION 40 - PROCESS INTERCONNECTIONS		
40 70 10	Instrumentation	6

Specifications

SECTION 01 01 00

GENERAL REQUIREMENTS

PART 1 GENERAL

1.01 PROJECT DESCRIPTION

A. In general, the project consists of:

1. Mobilization.
2. Drawdown, diversion, and dewatering.
3. Erosion and sediment controls.
4. Clearing and grubbing.
5. Stripping and stockpiling.
6. Developing borrow area.
7. Construction of new auxiliary spillway weir and improved spillway chute.
8. Excavation of embankment.
9. Demolition of existing outlet works (buttress) intake structure.
10. Construction of new outlet works (buttress) intake structure.
11. Replacement and rehabilitation of existing outlet works conduit.
12. Construction of embankment filter, filter diaphragm around conduit, and toe drain.
13. Backfill and site restoration.

1.02 WORK BY OTHERS

A. Owner will draw down the reservoir and bypass flows prior to and during construction.

1.03 WORK SEQUENCE

A. The work shall be performed in accordance with general sequence or phasing included on the Drawings.

1.04 PROJECT MEETINGS

A. A preconstruction conference will be scheduled after award of contract and prior to beginning work. This meeting shall be attended by A/E, Owner, and an authorized representative of Contractor.

B. Periodic progress meetings will be held at project site at times designated by Owner or A/E. A responsible representative of Contractor who can bind Contractor to decisions shall attend.

1.05 WORK HOURS

A. Work shall be conducted between the hours of 7:00 a.m. to 6:00 p.m. on normal work days, unless approved for unusual circumstances.

B. Give written notice to A/E whenever it is desired to perform work at night, or on a Saturday, Sunday, or holiday, or to vary period of hours during which work is carried on each day. If approved, such work shall be subject to requirements furnished in writing by A/E, and no extra compensation will be allowed.

1.06 SUBMITTAL PROCEDURES

A. See Section 01 33 00.

1.07 PERMITS AND CODES

A. Owner will obtain permits, licenses, and approvals, which may include the following:

1. State of Colorado Division of Water Resources Office of the State Engineer Dam Safety Branch Approval for the Alteration, Modification, or Repair of a Dam and Reservoir.
2. US Army Corps of Engineers (USACE) Clean Water Act Section 404 Nationwide Permit and Clean Water Act Section 401 Water Quality Certification Authorization.
3. Colorado State Historic Preservation Office (SHPO) Section 106 review.
4. US Forest Service (USFS) Special Use Permit.
5. Colorado Discharge Permit System (CDPS) General Permit for Stormwater Discharges Associated with Construction Activities, COR400000.
6. Mesa County Construction Stormwater Permit.

B. Contractor shall comply with the requirements of the above permits, licenses, and approvals. If a copy of a permit, license, or approval is not available for review prior to the Bid Deadline, and if it contains a requirement not covered by the Contract Documents, such a requirement will be considered extra work if Contractor makes a claim under the terms of the General Conditions. Work shall not begin on items applicable to the above until the required permit, license, or approval is received.

C. Contractor shall provide all other necessary permits and licenses and pay all fees, taxes, and royalties, unless otherwise indicated.

D. Comply with local and municipal ordinances and applicable state and national codes.

1.08 TEMPORARY UTILITIES

A. Contractor shall be responsible for providing temporary electric power as required for construction purposes. Provide portable power supply or make arrangements with local utility company.

B. Contractor shall be responsible for obtaining water for its needs. Pay cost of water used and meter rental, if applicable.

C. Contractor shall provide temporary outside toilets sufficient for construction workers. Toilets shall be self-contained chemical type and shall comply with applicable Codes. Maintain sanitary facilities in a clean and sanitary condition; supply toilet paper until completion of project. Contractor may not use USFS pit toilets located in project parking lot.

1.09 FIELD OFFICES

A. Contractor's Office:

1. Contractor shall provide and maintain a temporary office where directed for itself and its subcontractors.
2. Office shall have a meeting area for holding construction project meetings. Meeting will be attended by representatives of Contractor, A/E, Owner, and each subcontractor actively working on site.

B. A/E's Office:

1. Contractor shall provide and maintain a temporary office for use by A/E. Office may be in same mobile trailer as Contractor's office.
2. Provide the following equipment in A/E office: standard desk and chair, first aid kit and access to a copy machine.
3. Contractor shall arrange and pay for the following services: Power, heating, air conditioning, trash cans, and weekly trash pick-up service.
4. When directed, move A/E's office into a suitable area in building.

C. Internet Access:

1. Contractor shall arrange and pay for a wireless internet access point in the Contractor’s or A/E’s temporary office.

a. The access point shall allow dynamic IP addresses (DHCP) and be able to accommodate IPSec based VPN products.
b. The wireless internet access speed shall be a minimum of 10 Mbps download and 3 Mbps upload such as provided through a 4G LTE cellular connection.
c. The wireless service shall be secured with a password. Password shall be provided to A/E.
d. The wireless router shall have sufficient capacity to handle at least four computers operating simultaneously and accessing the internet simultaneously. A/E could have two wireless devices simultaneously accessing the internet.

1.10 PROTECTION

A. Furnish and maintain proper barricades, fences, signal lights, warning signs, and personnel as required to properly protect and safeguard the work, persons, animals, and property against injury.

1.11 ENVIRONMENTAL CONTROLS

A. See Section 01 57 19.

1.12 TRAFFIC CONTROL

A. Conduct operations to ensure minimum interference with streets, walks, and adjacent facilities not part of construction project.

B. Do not close or obstruct streets, walks, or other occupied or used facilities without permission from authorities having jurisdiction. Provide alternate routes around closed or obstructed traffic ways if required by governing regulations.

1.13 PRODUCT REQUIREMENTS

A. General: Provide new products manufactured and conditioned for the particular application as recommended by manufacturer, unless otherwise noted. Transport, handle, store, and protect products as specified and in accordance with manufacturer's recommendations.

B. Acceptable Manufacturers: Products, materials, and equipment identified by reference to a manufacturer's name, catalog number, or model are identified for the purpose of establishing a standard of type, function, appearance, and quality. Unless otherwise noted, any other product, material, or equipment which will perform adequately the duties imposed by the general design will be considered for substitution in accordance with the provisions below.

C. Bid Phase Substitutions: Substitutions and "or equal" items proposed prior to the Bid Deadline shall be submitted in accordance with the Instructions to Bidders.

D. Construction Phase Substitutions: Substitutions and "or equal" items proposed after Contract has been awarded shall be submitted for approval prior to their use. Consideration will be given only to proposed substitutions and "or equal" items where:

1. The products named in the Contract Documents are no longer available or cannot be provided within the Contract Time.
2. The manufacturers' standard products are no longer in conformance with the specified requirements.
3. Owner's interests may be adversely affected.

E. Substitution Procedures: Requests for substitution of alternate products or use of "or equal" items shall be submitted with complete references to manufacturer's product identification and product data indicating composition, guarantee, availability, applicable standards or agency approvals met or exceeded, restrictions imposed on product, and manufacturer's recommended method of application or installation. A substitution or an "or equal" item will be considered acceptable if the product will perform adequately the duties imposed by the

general design and, in opinion of A/E, is of equal substance, quality, appearance, and function, unless the named item is necessary for interchangeability or if the named product has been demonstrated to be most cost-effective.

1.14 SURVEYS, STAKING, LINE AND GRADE

A. Owner will provide baseline reference points and benchmarks as indicated on Drawings. Contractor shall provide all other survey staking and layout as required to complete the Work.

1.15 FIELD MEASUREMENTS AND INSPECTION OF SURFACES

A. Contractor shall layout its Work based on reference points furnished by Owner and shall be solely responsible for the accuracy of its measurements. Verify grades, lines, levels, locations, and dimensions as shown on Drawings, and inspect surfaces that are to receive work before proceeding with fabricating, assembling, fitting, or erecting. Notify A/E in writing in case of unsuitable conditions, defective substrates, or discrepancies in Contract Documents. Starting of work shall imply acceptance of conditions.

B. Correct any errors or defects due to faulty measurements, improper layout, or failure to report discrepancies.

1.16 CONSTRUCTION CLEANING

A. Keep work area free of accumulations of surplus materials, rubbish, and debris.

1.17 ACCESS ROAD MAINTENANCE

A. Notify A/E and Owner before removing any existing trees along the access road.

B. Restore access road to dam to conditions prior to construction.

1.18 PUNCH LIST

A. A "punch list" will be prepared and distributed to Contractor at Substantial Completion. Items on punch list shall be completed within 30 days. Required submittals (see below) shall be completed prior to or when requesting final payment.

1.19 CLOSEOUT SUBMITTALS

A. Submit the following items to A/E prior to or with final Application for Payment:

1. Project record drawings marked to show all changes made during construction. Dimension underground and concealed work and utilities from permanent reference points; record vertical distances. Make and record measurements to the nearest 0.5 ft on a clean drawing set.
2. Evidence of continuing insurance coverage complying with insurance requirements (see Conditions of the Contract).
3. Contractor's affidavit, along with final releases and waivers of liens as required by Owner, indicating that all debts and claims against project (less amounts withheld by Owner) have been paid in full or otherwise satisfied.
4. Consent of surety company to final payment.

1.20 DEFINITIONS

A. Dimensions on drawings and details are subject to field measurements.

B. The term "working days" shall exclude weekends (Saturday and Sunday) and holidays.

C. References to "Division 00" shall mean the Bidding Requirements and Contracting Requirements.

D. References to "DSB" shall mean State of Colorado Division of Water Resources Office of the State Engineer Dam Safety Branch.

E. References to "USACE" shall mean US Army Corps of Engineers.

F. References to "Colorado DOT Std. Spec." shall mean Colorado Department of Transportation, Standard Specifications for Road and Bridge Construction, latest edition.

G. References to "USFS" shall mean US Forest Service.

H. References to "CDPS" shall mean Colorado Discharge Permit System.

I. References to "A/E", "Architect", or "Engineer" shall mean Ayres.

J. References to "Owner" shall mean City of Grand Junction.

PART 2 (NOT USED)

PART 3 (NOT USED)

END OF SECTION

SECTION 01 22 50

MEASUREMENT AND PAYMENT

PART 1 GENERAL

1.01 GENERAL REQUIREMENTS

A. Payment for all work done in compliance with the Contract Documents, including all labor, equipment, materials, and performance of operations relative to construction of this project, will be made under the Bid Items listed below. Work required by the Contract Documents for which there is not a Bid Item will be considered incidental to the Contract and no additional compensation will be allowed.

B. Owner reserves the right to alter Drawings, modify incidental work as may be necessary, and increase or decrease quantities of work to be performed, including deduction or cancellation of any one or more Bid Items. Changes in the Work shall not be considered as a waiver of any conditions of the Contract nor invalidate any provisions thereof. When changes result in revised quantities of work to be performed, Contractor shall accept payment according to contract unit prices appearing in the original Contract. A supplemental agreement between Contractor and Owner may be requested by either party when cumulative changes involve a net increase or decrease of more than 20 percent in total contract amount.

C. Final measured quantities determined in field by A/E at time of construction shall govern over approximate quantities shown on the Bid Schedule, unless otherwise noted. Contractor shall take no advantage of any apparent error or omission in Drawings or Specifications, and A/E shall be permitted to make corrections and interpretations as may be deemed necessary for fulfillment of the intent of the Contract Documents.

PART 2 (NOT USED)

PART 3 EXECUTION

3.01 MOBILIZATION

A. This work consists of work and operations necessary for movement of personnel, equipment, supplies, and incidentals to project site and for establishment of Contractor's offices and other temporary facilities necessary for work on project; and of all other work and operations which must be performed, or for which costs must be incurred before beginning work on various items on project site; and demobilization at completion of work.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract lump sum price for MOBILIZATION, payable to Contractor in accordance with the following schedule:

1. When 5 percent or more of original contract amount is earned, 40 percent of amount bid for mobilization will be paid.
2. When 25 percent or more of original contract amount is earned, 80 percent of amount bid for mobilization will be paid.
3. When 90 percent or more of original contract amount is earned, 100 percent of amount bid for mobilization will be paid.

3.02 EROSION CONTROL

A. This work consists of furnishing, installing, and maintaining erosion and other environmental control measures in accordance with Section 01 57 19.

B. Measurement for payment will be as a complete unit of work acceptably completed.

C. Payment will be made at the contract lump sum price for EROSION CONTROL, payable to Contractor in accordance with the following schedule:

1. When erosion control measures are installed, 75 percent of amount bid for erosion control will be paid.
2. When erosion control measures have been removed, 100 percent of amount bid for erosion control will be paid.

3.03 CONSTRUCTION DEWATERING

A. This work consists of diverting surface water, constructing cofferdams, and dewatering construction site in accordance with Section 01 57 60 as required to complete the work.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract lump sum price for CONSTRUCTION DEWATERING, payable to Contractor in accordance with the following schedule:

1. When site is dewatered for construction, 75 percent of amount bid for dewatering will be paid.
2. When all dewatering measures are removed, 100 percent of amount bid for dewatering will be paid.

3.04 DEMOLITION

A. This work consists of removing intake and outfall structures, piping and controls (as shown on drawings) and grouted riprap accordance with Section 02 41 00.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract lump sum price for DEMOLITION.

3.05 CONCRETE OVERFLOW SPILLWAY

A. This work consists of constructing the Concrete Overflow Spillway as designated in the Drawings and in accordance with Section 03 30 00.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract lump sum price for CONCRRETE OVERFLOW SPILLWAY.

3.06 ROCK SHELL REMOVAL

A. This work consists of excavating and stockpiling the downstream rock shell defined in Section 31 24 00.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract unit price per cubic yard for Rock SHELL REMOVAL.

3.07 TYPE I FILTER MATERIAL

A. This work consists of furnishing and placing Type I filter material in accordance with Sections 31 24 00 and 33 42 15 and as shown on the Drawings.

B. Measurement for payment will be the number of cubic yards of Type I filter material acceptably placed within the specified pay limits.

1. Type I Filter Material required to fill voids resulting from over excavation outside specified lines and grades will not be measured for payment, unless such over excavation is authorized to remove unsuitable material where unsuitable condition is not a result of Contractor's operations.
2. Pay limits shall be the measured surface of foundation when approved for placement of fill and designated neat lines of fill surface. Quantities will be based upon project plan drawings.

C. Payment will be made at the contract unit price per cubic yard for TYPE I FILTER MATERIAL.

3.08 FILTER DRAIN PIPE

A. This work consists of furnishing and installing drainage pipe and cleanout. This work also includes the granular filter blanket in accordance with Sections 33 41 66 and 33 42 15.

B. Measurement for payment will be the number of linear feet of drainage pipe in place and the number of cubic yards of granular filter blanket materials, not to exceed limits shown on Drawings.

C. Payment for drainage pipe will be made at the contract unit price per linear foot for DRAINAGE PIPE.

D. Payment for granular filter blanket will be made at the contract unit price per cubic yard for FILTER DRAIN PIPE.

3.09 FILTER EMBANKMENT CAP

A. This work consists of excavating materials from embankment, stockpiling, placing, and compacting materials as specified, restoring stockpile areas, and other items necessary and incidental for construction of embankments in accordance with Section 31 20 00.

B. Measurement for payment will be the volume of earth fill within specified zones and pay limits measured and computed to nearest cubic yard by method of average cross-sectional end areas. Unless otherwise indicated, no deduction in volume will be made for embedded conduits and appurtenances.

C. Earth fill required to fill voids resulting from over excavation outside specified lines and grades will not be measured for payment, unless such over excavation is authorized to remove unsuitable material where unsuitable condition is not a result of Contractor's operations.

D. Pay limits shall be the measured surface of foundation when approved for placement of fill and designated neat lines of fill surface.

E. Payment for each type of earth fill will be made at the respective contract unit price per cubic yard for FILTER EMBANKMENT CAPEARTH.

3.10 ROCK SHELL REPLACEMENT

A. This work consists of restoring downstream rock shell defined in Section 31 24 00.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract unit price per cubic yard for ROCK SHELL REPLACEMENT.

3.11 HYDRAULIC SLUICE GATE

A. This work consists of furnishing and installing sluice gate, operators, hoses, casement pipes, pull box, portable operator, and anchorages in accordance with Sections 26 05 43, 33 42 15 and 35 22 28.

B. Measurement for payment will be the number of units acceptably installed.

C. Payment will be made at the contract unit price each for HYDRAULIC SLUICE GATE.

3.12 INTAKE STRUCTURE

A. This work consists of constructing the intake structure and pipe encasement as designated in the Drawings and in accordance with Section 03 30 00.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract lump sum price for CONCRETE OVERFLOW SPILLWAY.

3.13 AUXILIARY SPILLWAY GRADING

A. This work consists of constructing the intake structure as designated in the Drawings and in accordance with Section 31 20 00.

B. Measurement for payment for auxiliary spillway grading will be the square yards of spillway grading not to exceed limits shown on Drawings will be made at the contract unit price per square yard.

C. Payment for auxiliary spillway grading will be made at the contract unit price per square yard for AUXILIARY SPILLWAY GRADING.

3.14 AUXILIARY SPILLWAY ROCK BERMS

A. This work consists of furnishing and placing riprap in accordance with Section 31 37 00.

B. Measurement for payment will be the number of cubic yards of riprap in place.

C. Payment for riprap will be made at the contract unit price per cubic yard for AUXILIARY SPILLWAY ROCK BERMS.

3.15 PIPE FILTER DIAPHRAGM

A. This work consists of furnishing and placing Type I filter material for pipe diaphragm in accordance with Sections 31 24 00 and 33 41 66 and as shown on the Drawings.

B. Measurement for payment will be the number of cubic yards of Type I filter material acceptably placed within the specified pay limits.

1. Type I Filter Material required to fill voids resulting from over excavation outside specified lines and grades will not be measured for payment, unless such over excavation is authorized to remove unsuitable material where unsuitable condition is not a result of Contractor’s operations.
2. Pay limits shall be the measured surface of foundation when approved for placement of fill and designated neat lines of fill surface. Quantities will be based upon project plan drawings.

C. Measurement for payment will be the number of cubic yards of Type I filter material acceptably placed within the specified pay limits.

D. Payment will be made at the contract unit price per cubic yard for PIPE FILTER DIAPHRAGM.

3.16 OUTFALL PIPE

A. This work consists of constructing the outfall pipe as designated in the Drawings and in accordance with Section 33 42 15.

B. Measurement for payment will be the linear feet of pipe placed.

C. Payment will be made at the contract lump sum price for OUTFALL PIPE.

3.17 CIPP PIPE LINING

A. This work consists of lining of the steel penstock pipe in accordance with Section 33 01 36 and as shown on the Drawings. Any special equipment, personnel, or materials required for this work shall be incidental to this item.

B. Measurement for payment will be the number of linear feet of pipe acceptable lined.

C. Payment for fence will be made at the contract unit price per linear foot for CIPP PIPE LINING FENCE.

3.18 SITE RESTORATION

A. This work consists of placing topsoil, fertilizing, seeding, sodding, planting, and mulching in accordance with Sections 32 92 26 and 32 99 10.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract lump sum price for SITE RESTORATION.

3.19 EARLY WARNING SYSTEM

A. This work consists of furnishing and installing the early warning system in accordance with Section 40 70 10.

B. Measurement for payment will be as a complete unit of work acceptably performed.

C. Payment will be made at the contract lump sum price for EARLY WARNING SYSTEM.

END OF SECTION

SECTION 01 33 00

SUBMITTAL PROCEDURES

PART 1 GENERAL

1.01 SUMMARY

A. Submit items to A/E for review as required by the various Contract Documents. Refer to individual specification sections, General Conditions, Supplementary Conditions, and sections of Division 01 - General Requirements for submittal requirements.

1.02 GENERAL PROCEDURES

A. Follow the requirements for each submittal type as specified below.

B. Submittals shall be identified with project name, numbered consecutively, and bear the stamp of approval of Contractor as evidence of accuracy, compatibility, and conformance with contract requirements. Submittals not so stamped will be returned without being examined.

C. Give specific written notice of each variation that submittals may have from requirements of the Contract Documents.

D. Partial submittals will not be considered. Submit each portion of work complete in one submittal.

E. Products subject to submittal review shall not be used in the work until submittals have been reviewed and bear the stamp and signature of A/E. Submittals will only be reviewed for general conformance with the design concept of the project and general compliance with the information given in the Contract Documents. Contractor shall be responsible for confirming and correlating all quantities and dimensions, selecting fabrication processes and the means and methods of construction, coordinating its work with that of all other trades, and performing all work in a safe and satisfactory manner. Corrections or comments made on submittals shall not relieve Contractor from compliance with requirements of Drawings and Specifications and shall not be considered an order for extra work.

F. If information on previously reviewed shop drawings is altered, submit changes for review.

G. Maintain complete copies of all final submittals at the project site.

1.03 SHOP DRAWINGS

A. Required shop drawings are designated in the various specification sections. Submit shop drawings for review prior to fabrication, delivery, or installation in one of the following formats:

1. PDF electronic file. An annotated PDF electronic file will be returned to Contractor.

B. Each brochure of shop drawings shall contain an index of contents and shall consist of layout details, schedules, setting instructions, manufacturer's literature, and other data specifically prepared for the work. Reproductions of contract drawings may not be used without prior approval.

1.04 PRODUCT DATA

A. Required product data are designated in the various specification sections. Submit product data for review prior to delivery or installation in one of the following formats:

1. PDF electronic file. An annotated PDF electronic file will be returned to Contractor.

B. Product data shall consist of manufacturer's literature, illustrations, and brochures of catalog cuts; instructions for handling, storage, and installation; and specifications and design data. Where manufacturer's standard literature includes multiple products or options, identify the specific products and options as required for this project.

1.05 SAMPLES

A. Prior to fabrication, delivery, or installation, submit samples as designated in the various specification sections. Allow reasonable time for review and testing.

1. Submit samples in sufficient quantity and of adequate size to show quality, type, and extremes of color range, finish, and texture. Submit a minimum of two sets of appearance and color samples.

B. Label each sample stating material, description, project name, and Contractor's name. Expedite submittal of appearance and color samples following Notice to Proceed.

C. Submit samples with transmittal letter requesting review; prepay transportation charges. Samples shall become Owner's property, unless otherwise designated.

D. Samples will be reviewed for acceptability or selection of color, pattern, and texture only. Compliance with specifications is the responsibility of Contractor.

E. Order no materials subject to sample review until receipt of written notice of completion of review. Installed materials shall match reviewed samples. No review of samples shall be taken in itself to change contract requirements.

1.06 CERTIFICATES OF COMPLIANCE

A. Submit certificates of compliance as designated in the various specification sections in one of the following formats:

1. PDF electronic file.

B. Certificates shall be furnished by manufacturer, producer, or supplier of material or product and shall indicate that material or product conforms to or exceeds specified requirements. Include supporting reference data as appropriate. Certificates may be recent or previous test results on material or product, but must be acceptable to A/E.

1.07 PERMITS AND APPROVALS

A. Submit permits, code inspections, and agency approval documents as designated in the various specification sections in one of the following formats:

1. PDF electronic file.

1.08 TEST REPORTS

A. Submit test reports as designated in the various technical specifications in one of the following formats:

1. PDF electronic file.

1.09 OPERATION AND MAINTENANCE (O/M) MANUALS

A. Submit operation and maintenance manuals covering each item of equipment furnished or installed under the Contract. Submit individual preliminary O/M brochures in PDF electronic format within 30 days after completion of shop drawing or product data review. Submit final O/M manuals prior to substantial completion in the following formats:

1. PDF electronic file of entire manual, and
2. Two bound paper sets (unless otherwise specified in the various specification sections).

B. For each item of equipment, include the following information:

1. A/E-reviewed shop drawings and product data.
2. Installation and operating instructions.
3. Maintenance instructions and address of authorized service center.
4. Wiring diagrams and parts lists.
5. Test data and certifications.
6. Manufacturer's warranty information.

C. Designate correct model number where literature covers more than one model.

D. Write and furnish duplicate operation and maintenance instructions for items fabricated or assembled by Contractor.

E. Electronic Manual:

1. Submit manual in the form of a single PDF file for entire project.
2. Use electronic files prepared by manufacturer where available. Where scanning of paper documents is required, configure scanned file for minimum readable file size.
3. Group data according to specification section and organize with bookmarks. Bookmark both specification sections and individual products so that resulting bookmarks reflect a readily navigated document tree.
4. Include an overall table of contents of the O/M manuals furnished.

F. Paper Manuals:

1. Furnish data in 8-1/2 in. x 11 in. or 11 in. x 17 in. size; photographically reduce information if required. Place data into D-style, 3-ring hard cover binders; fold 11 in. x 17 in. sheets as required. Group data according to specification section and organize with tabbed index dividers on which the product name is typed.
2. Label binders as follows:

[Systems or Equipment Designation]
OPERATION AND MAINTENANCE MANUAL
[Project Name]
[Project Location]

3. Integrate general, mechanical, and electrical construction into same binder(s) when practicable. Individual subcontract O/M manuals will be acceptable provided they are placed in binder(s) as specified above.
4. Include an overall table of contents of the O/M manuals furnished.

1.10 PROJECT RECORD DOCUMENTS

A. Keep a current set of paper documents at project site that are marked to show all changes made during construction. Dimension underground and concealed work and utilities from permanent reference points; record vertical distances. Make and record measurements to the nearest 0.1 ft. Submit project record documents upon completion of Work.

PART 2 (NOT USED)

PART 3 (NOT USED)

END OF SECTION

SECTION 01 57 19

TEMPORARY ENVIRONMENTAL CONTROLS

PART 1 GENERAL

1.01 SUMMARY

A. Provide temporary environmental controls as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 DEFINITIONS

A. References to "Colorado DOT Std. Spec." shall mean Colorado Department of Transportation Standard Specifications for Road and Bridge Construction (available at https://www.codot.gov/business/designsupport/cdot-construction-specifications/2019-construction-specifications/2019-specs-book/2019-standard-specifications).

1.03 SUBMITTALS

A. Submit in accordance with Section 01 33 00: Submittals.

B. Inspection Reports: Submit one copy of weekly inspection reports for erosion and sediment controls.

C. Technical product literature for all commercial products to be used for sedimentation and erosion control.

D. Contractor's Sedimentation and Erosion Control Plan: A plan describing Best Management Practices (BMPs) in accordance with local, state, and federal regulations and permits and this Section and Drawings stamped and sealed by a professional engineer as required.

1.04 PERMITS

A. Owner will:

1. Submit a Colorado Discharge Permit System (CDPS) General Permit for Stormwater Discharges Associated with Construction Activities, COR400000 Permit Application at least 10 working days prior to the start of construction.
2. Provide Contractor with a copy of the notice of General Permit coverage.
3. File Notice of Termination after construction site has undergone final stabilization.
4. Apply for Mesa County Construction Stormwater Permit https://etrakit.mesacounty.us/etrakit3/.

B. Contractor shall:

1. Comply with the requirements of the General Permit and the Sedimentation and Erosion Control Plan and keep a copy of these documents at project site during construction.
2. Post a copy of the Certificate of Permit Coverage in a conspicuous place on the construction site.

PART 2 PRODUCTS

2.01 SILT FENCE

A. Posts: 2-inch by 2-inch; 4 feet 6-inch long, wood stakes, suitably durable for driving without cracking, as approved by the A/E.

B. Fabric: Woven, polypropylene, ultraviolet resistant material. Mirafi, Inc. Mirafi 100X, or approved equal.

C. Prefabricated commercial silt fence, if substituted for built-in-field fence: Mirafi Inc. "Envirofence" or approved equal.

2.02 EROSION BALES

A. Certified weed hay or straw certified under the North American Weed Free Forage Certification Program and the New Mexico State University Seed Certification (NMSUSC) Program. Each certified weed free erosion bale shall be identified with purple and yellow twine, and regional Forage Certification Program tag indicating the Regional Forage Certification Program Number unless otherwise specified by the program.

B. Erosion bales shall be inspected for and Regionally Certified as weed free based on the Regionally Designated Noxious Weed and Undesirable Plant List. Do not unload certified weed free erosion bales or remove their identifying twine, wire or tags until the A/E has inspected and accepted them. Provide a certificate of compliance showing the transit certificate number or a copy of the transit certificate as supplied from the forage producer.

2.03 EROSION LOGS

A. Curled aspen wood excelsior with a consistent width of fibers evenly distributed throughout the log and a seamless casing comprised of a photodegradable tube netting. Fungus free, resin free and free of growth or germination inhibiting substances. Furnish logs with the minimum diameter and length shown on approved erosion and sediment control plans.

2.04 VEHICLE TRACKING PAD

A. Constructed tracking pad complying with Colorado DOT Std. Spec. 208.02.

B. Use crushed natural aggregate with at least two fractured faces that meets the Colorado DOT Std. Spec. gradation requirements for vehicle tracking pads.

C. Geotextile fabric for tracking pads shall be Colorado DOT Std. Spec., Subsection 712.08, Class 2 fabric.

2.05 PERMANENT SEED

A. See Section 32 92 36.

PART 3 EXECUTION

3.01 EROSION CONTROL

A. General: Maintain erosion control measures to protect the project site and prevent sediment pollution of adjacent water courses and properties. At a minimum, provide erosion control measures as indicated on the Drawings.

B. Applicable Standards: Unless otherwise shown or specified, erosion control measures shall comply with:

1. CDPS General Permit for Stormwater Discharges Associated with Construction Activities, COR400000.
2. Colorado DOT Std. Spec.

C. Time Period: Install erosion control measures prior to start of construction and maintain them until final completion of work. Unless otherwise instructed, remove temporary erosion control measures prior to final application for payment.

D. Stripping: Strive to limit stripping of sod and vegetation to a period that will expose bare soil to the least possibility of erosion that construction requirements will allow.

E. Diversions: Construct and maintain dams, channels, flumes, sumps, surface roughening, and other temporary diversion and protective works to divert streamflow and other surface water through or around construction site and away from work while construction is in progress. Unless otherwise specified, a diversion must discharge into the same natural drainageway in which its head waters are located.

F. Sediment Barriers: Construct and maintain one or more sediment barriers to receive runoff leaving site.

1. Affix silt fence to ground and maintain in suitable structural condition to last until vegetation establishes.
2. Affix erosion logs to ground, either by wood stakes (recommended method) or by equivalent anchorage if frozen winter subgrade prevents driving stakes. Equivalent anchorages include using steel posts (removed after vegetation establishes) or erosion bales spaced at same intervals as planned stakes or other approved equivalent method.

G. Trackout Control: Prevent tracking of soils and sediments onto public and private streets by constructing and maintaining stabilized work surfaces and trackout controls in accordance with Colorado DOT Std. Spec. Check dual tire vehicles for picked up tracking pad materials prior to leaving site. Refresh and loosen tracking pad as needed to allow stones to contact full tire tread (at least up to start of sidewall) of exiting vehicles. If a vehicle or tire washing station is established, water from washing shall drain into a suitable sediment trap or settling device. Remove at the end of each workday soils and sediment reaching public and private streets not part of the construction site.

H. Re-establishment of Vegetation: Re-establish temporary or permanent vegetation on disturbed areas within the time limits allowed by applicable standards.

I. Sediment Deposits: Remove and dispose of sediment deposits when deposits reach one-half the volume capacity of sediment barrier, unless otherwise indicated.

3.02 EROSION CONTROL MONITORING AND REPORTING

A. Contractor shall conduct the following inspections:

1. Weekly inspections of implemented erosion and sediment controls.
2. Inspections of erosion and sediment controls within 24 hours after a precipitation event that produces 0.5 in. of rain or more during a 24 hour period.

B. Contractor shall prepare weekly written reports of all inspections that include:

1. Date, time, and exact place of inspection.
2. Name of individual who performed inspection.
3. An assessment of condition of erosion and sediment controls.
4. A description of any erosion and sediment control implementation and maintenance performed.
5. A description of the present phase of construction at site.

3.03 DUST CONTROL

A. Minimize dispersion of dust from construction operations by application of water or other dust control materials. Controls shall confine dust and dirt within the immediate area of project. Masonry and debris shall be thoroughly soaked during demolition and loading operations.

3.04 NOISE CONTROL

A. Provide noise control measures to limit the amount of noise and prevent nuisance. Properly equip all equipment with mufflers. Limit construction activities generating significant noise to normal working hours.

3.05 MATERIAL HANDLING AND SPILL PREVENTION

A. Hazardous materials shall be handled and stored in accordance with the recommendations of the manufacturer and Material Safety Data Sheets (MSDS). Containers or equipment leaking any contaminants shall be repaired, removed from the site, or utilize drip pans for containment purposes.

B. Spills of any contaminants shall be immediately reported to the Colorado Water Quality Control Division Hotline at 877.518.5608. Contractor shall maintain an emergency spill kit at the project site containing contaminant containment products and absorbent materials (inlet socks, dry absorbent materials, and similar products).

3.06 HAZARDOUS ENVIRONMENTAL CONDITIONS

A. If underground petroleum storage tanks, petroleum contaminated soils, or other hazardous environmental conditions are encountered, and are not identified to be part of the work, Contractor shall immediately stop all work in connection with the hazardous condition and shall notify Owner and A/E. (See the General Conditions of the Contract for specific procedures that may apply.)

END OF SECTION

SECTION 01 57 60

CONSTRUCTION DEWATERING

PART 1 GENERAL

1.01 SUMMARY

A. Provide construction dewatering as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

B. Work under this Section shall consist of removal of surface water and ground water as necessary to perform required work, including:

1. Building and maintaining temporary impounding works, channels, and diversions.
2. Furnishing, installing, and operating pumps, piping, and other facilities and equipment.
3. Removing temporary works and equipment when no longer required.

C. Contractor shall be responsible for design of dewatering system as specified below.

D. Construction dewatering is broken down into Phase 1 and Phase 2, as indicated on the Drawings. Phase 1 dewatering is to facilitate construction of the new outlet works, the embankment, and the downstream impact basin. Phase 2 dewatering is to facilitate construction of the overflow spillway weir and channel improvements.

E. Cofferdam (required for Phase 1 dewatering) will not protect the worksite from the 10-yr flood. Contractor shall monitor weather forecasts and current conditions and evacuate equipment and personnel from the drained reservoir should there be a risk of water levels in the lake rising above the specified construction pool.

1.02 SUBMITTALS

A. Dewatering Plan: Submit Dewatering Plan consisting of the following:

1. Cofferdam Cross Section: Submit cross section showing minimum and maximum cofferdam height, minimum and maximum grades near cofferdam, and distance from cofferdam section to nearest dam structure. Top elevations of cofferdams shown on the Drawings are to be considered minimum and may not be reduced unless Contractor obtains A/E permission and Contractor accepts all liability for workmanship lost and damages due to premature overtopping of cofferdam.
2. Location Map: Submit phased dewatering plan showing all stages and sequences of cofferdam construction and removal with adequate descriptions so that A/E can evaluate possible reductions to flood capacity of dam.
3. Cofferdam Drawing Bearing Professional Engineering Stamp: Submit an installation drawing bearing the stamp of a professional engineer who is registered in the state where the cofferdam is to be located. Stamped drawing(s) shall include height and depth of cofferdam, width of cofferdam materials (sheet size or base width), and minimum grade elevations near cofferdam. A/E approval of Contractor cofferdam drawing shall be considered only as review in conformance with dewatering intent and not considered as a full structural review or confirmation that cofferdam is suitable for intended purpose.

B. Permit Applications: After A/E's approval of Dewatering Plan, make application for required Contractor-furnished cofferdam and dewatering permits. Submit copy of applications to A/E.

C. Permits: Submit copy of permits obtained for project prior to starting the applicable work.

D. Make submittals in accordance with Section 01 33 00.

1.03 PERMITS

A. General: Rehabilitation of the dam, including cofferdams and diversions, shall be in accordance with Colorado Division of Water Resources Office of the State Engineer, U.S. Army Corps of Engineers (COE), Mesa County, and US Forest Service permits issued for project. The work is subject to inspection, review, and approval by these agencies.

B. Owner-Furnished Dam Rehabilitation Permit: Owner has obtained permits to rehabilitate dam from DSB, COE, and USFS. Refer to Appendices.

C. Contractor-Furnished Dewatering Permit: Contractor's method of accommodating seepage may require a Colorado Discharge Permit System (CDPS) General Permit COG080000 for Discharges from Short-Term Construction Dewatering Activities. Contact Colorado Department of Public Health and Environment for more details. Required dewatering permit(s) shall be obtained at Contractor's cost and no additional schedule allowance will be given to accommodate the permit review period.

D. Permit Compliance: Should Contractor's actions or construction not be in compliance with applicable permits, Contractor shall remedy situation as directed by Owner, and all costs associated with those actions shall be borne by Contractor.

1.04 SITE CONDITIONS

A. Information on flood flows and average flows is presented on the Drawings.

B. Owner will draw down the reservoir prior to construction.

C. Owner will operate the diversion gates upstream of the reservoir during construction.

PART 2 PRODUCTS

2.01 DESIGN REQUIREMENTS

A. Contractor shall be responsible for:

1. Protection of work area and safely passing stream flow for duration of construction.
2. Means and methods for dewatering work areas, including the actual dimensions, configurations, stability, and dewatering capacity of cofferdams and protective works.
3. All safety precautions and programs related to the work.

2.02 MATERIALS

A. Contractor shall furnish all materials for and shall construct and maintain, as it deems necessary, all cofferdams, channels, drains, sumps, and protective works for protection of work areas.

B. Contractor may select whatever materials it wishes for use in cofferdam, subject to permit requirements and the following:

1. Sheet piling may not be used within 10 ft of any permanent structure due to potential vibration damage.
2. Cofferdam top elevation shown on the Drawings may not be exceeded.
3. Cofferdam shall be equipped with a temporary 30-inch relief outlet as shown on the Drawings.

PART 3 EXECUTION

3.01 DIVERTING SURFACE WATER DURING PHASE 1 DEWATERING

A. Owner shall draw down the reservoir prior to construction and shall operate the existing bypass channel to divert normal river flows around the lake during construction. Owner estimates that the existing bypass channel can divert up to 50 cfs around the lake. River flows exceeding 50 cfs may flow into the lake and cause levels to rise.

B. If lake levels rise above the specified construction pool elevation shown on the Drawings, Contractor shall drain the lake using a temporary outlet. If the temporary outlet is not capable of draining the lake sufficiently, Contractor shall use pumping for additional release capacity.

C. Construct, maintain, and operate cofferdams, channels, flumes, sumps, and other temporary diversion and protective works to divert streamflow and other surface water through or around construction site and away from work while construction is in progress. Unless otherwise specified, diversions must discharge into the same natural drainageway in which its headwaters are located.

D. Surface water diversion procedures shall not create a condition where erosion or deposition of materials occurs in stream. Riprap or other means of protection shall be provided for erosion protection adjacent to all cofferdams where flows could occur.

E. Diversion works which are moved out of position by any cause during installation shall be righted or enlarged so as to provide necessary clearance.

F. As work area is dewatered, diversion works that are not watertight shall be plugged or sealed as much as practical to reduce infiltration of water into work area.

G. No shoring will be permitted in diversion works which will induce stress, shock, or vibration in permanent structure.

3.02 DIVERTING SURFACE WATER DURING PHASE 2 DEWATERING

A. Owner will operate the reservoir’s low-level outlet works with the gate fully open to pass stream flows through the lake and dam. During this phase, Owner shall operate the bypass gates upstream of the reservoir to minimize flow into the bypass channel and overflow spillway.

3.03 DEWATERING EXCAVATIONS AND WORK AREAS

A. Foundations, cutoff trenches, and other parts of construction site shall be dewatered and kept free of standing water or excessively muddy conditions for proper execution of construction work. Furnish, install, operate, and maintain wells, drains, sumps, pumps, and other equipment needed to perform dewatering as specified. Dewatering methods that cause loss of fines from foundation materials will not be permitted.

B. Maintain pumping operations to keep work area dry until all materials, equipment, and debris have been removed and diversion works is to be removed.

3.04 DEWATERING BORROW AREAS

A. Maintain borrow areas in drainable condition or otherwise provide for timely and effective removal of surface waters that accumulate, for any reason, within borrow areas.

3.05 REMOVAL OF TEMPORARY WORKS

A. Remove temporary works when no longer required; level and grade earth as required to restore appearance and to prevent obstruction to flow or any other interference with operation of or access to permanent works.

B. Unless otherwise noted, pipes and casings shall be removed from temporary wells and wells shall be filled to adjacent ground level with gravel or other approved material.

C. Construction dewatering material shall be removed from site and properly disposed of.

D. Contractor shall make its own arrangements for a disposal site and shall pay all costs involved.

3.06 REPAIR OF DAMAGES

A. Contractor shall repair, at its expense, any damage to foundations, structures, or other improvements caused by failure of any part of cofferdams or protective works.

END OF SECTION

SECTION 02 41 00

DEMOLITION

PART 1 GENERAL

1.01 SUMMARY

A. Provide demolition work as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 RELATED SECTIONS

31 05 10 Site Preparation.

1.03 PRE-DEMOLITION CONFERENCE

A. Contractor shall schedule a pre-demolition conference prior to beginning demolition to be attended by authorized representatives of A/E, Owner, and Contractor and by interested agencies.

B. Contractor shall give notice of conference to agencies listed under "Permits and Notification of Agencies" article, below.

1.04 SUBMITTALS

A. Demolition Work Plan: Submit a demolition work plan for A/E's information at least one week prior to start of work. Work plan shall include:

1. Work Sequence: Sequence of operation for demolition and removal work.
2. Noise and Dust Control Plan: Planned measures to minimize noise levels and control dust emissions from demolition activities.
3. Material Disposal Plan: Planned material disposal/recycling locations.

B. Permits: Submit copy of each application to and permit received from agencies having jurisdiction.

C. Disposal/Recycling Documentation: Submit copy of each disposal manifest or letter of acceptance for all disposed and recycled materials.

D. Make submittals in accordance with Section 01 33 00.

1.05 PERMITS AND NOTIFICATION OF AGENCIES

A. Demolition Permit: Obtain and pay for demolition permit from local jurisdiction.

B. Colorado Department of Public Health and Environment Notification: Submit and pay for Form DNA08 "Demolition Notification Application Form" to the Permit Coordinator at least 10 working days prior to demolition.

C. Utility and Agency Notifications: Make arrangements with the following utilities and agencies so that work may proceed with least disruption of traffic and services in area. Notify Owner, A/E, adjacent property owners, and each of the following agencies having jurisdiction three days prior to beginning work.

1. Department of Public Works.
2. US Forest Service.
3. Police Department.
4. Fire Department.

1.06 WORKER QUALIFICATIONS

A. Contractor is responsible for health and safety of its personnel during all demolition and construction work, including hazardous materials training, if required, in accordance with applicable laws and regulations.

1.07 EXISTING CONDITIONS

A. Prior to bidding, thoroughly inspect existing conditions.

B. Conditions existing at time of inspection for bidding purposes will be maintained by Owner in so far as practicable. However, variations within facilities may occur by Owner's removal and salvage operations prior to start of demolition work.

C. Structures to be demolished will be vacated and use discontinued prior to start of work. Owner assumes no responsibility for actual condition of structures to be demolished.

D. The existing low-level intake structure and outlet conduit will be submerged/buried at the time of bidding. Thus, these structures will not be accessible for inspection during bidding.

1.08 EXPLOSIVES

A. Use of explosives will not be permitted.

PART 2 PRODUCTS

2.01 FILL MATERIAL

A. Comply with the requirements of Section 31 24 00.

PART 3 EXECUTION

3.01 PROTECTION OF EXISTING IMPROVEMENTS

A. Comply with requirements of Section 01 01 00 / 31 05 10 and governing regulations.

B. Conduct operations to prevent damage to adjacent facilities. Provide interior and exterior shoring, bracing, or support to prevent movement, settlement, or collapse of adjacent facilities to remain and to prevent premature collapse of structures to be demolished. Maintain work and site in orderly condition to prevent accidents.

C. Protect survey monuments, reference points, benchmarks, and monitoring wells. Notify owner of disturbance.

3.02 SALVAGE

A. Items designated for salvage to Owner shall be removed in a workmanlike manner and neatly stored on-site. Items not designated for salvage to Owner shall become property of Contractor and shall be removed from site.

B. Salvaged items capable of disassembly shall be dismantled into individual components or subsections.

3.03 DEMOLITION

A. See Drawings for specific areas, depths, and limits of demolition, removal, salvage, and disposal. Restrict demolition activities to project limits.

B. Minimize dispersion of dust from demolition operations and dust generated by equipment traffic by application of water or other dust control materials. Controls shall confine dust and dirt within immediate area of demolition. Thoroughly soak masonry and debris during demolition and loading operations.

C. Crushing of concrete, brick, and other masonry materials will be allowed on site, provided the material is not contaminated with hazardous materials.

3.04 BELOW-GRADE STRUCTURES

A. Demolish and remove all foundations, footings, concrete slabs-on-ground, and all other below-grade construction within construction limits.

3.05 FILLING EXCAVATIONS AND VOIDS

A. Completely fill below-grade areas and voids resulting from demolition in accordance with the requirements of Section 31 24 00.

3.06 DISPOSAL OF DEMOLISHED MATERIALS

A. Remove from site and legally dispose of all debris, rubbish, and other materials resulting from demolition operations. Storage or sale of removed materials will not be permitted on site.

B. Keep all haul roads clean and free of debris. Take measures to avoid littering waste. No waste or litter shall buried in any open trench, excavation or fill area in the project site.

3.07 RESTORATION

A. Repair adjacent improvements damaged by demolition operations. Clean adjacent areas of dust, dirt, and debris caused by demolition operations. Return adjacent areas to condition existing prior to start of work.

B. Topsoil, fertilize, seed, and mulch exposed soils in accordance with Section 32 99 10, except areas designated to receive new construction.

3.08 CLOSEOUT

A. Deliver to Owner all designated salvage items; obtain receipts from Owner for materials delivered.

END OF SECTION

SECTION 03 30 00

CAST-IN-PLACE CONCRETE

PART 1 GENERAL

1.01 SUMMARY

A. Provide cast-in-place concrete work as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 SUBMITTALS

A. Shop Drawings: Submit shop drawings for fabrication, bending, and placement of concrete reinforcement. Comply with ACI 315.

B. Product Data: Submit product data for each product specified.

C. Mix Designs: Submit proposed mix designs to A/E ten days prior to beginning concrete work. Do not begin concrete production until mixes have been reviewed.

D. Delivery Tickets: Submit delivery ticket to A/E for each load of concrete delivered to project.

E. Test Reports: Testing agency shall submit copy of field and laboratory reports to A/E.

F. Make submittals in accordance with Section 01 33 00.

1.03 QUALITY ASSURANCE

A. Give A/E two working days notification of all planned concrete pours so that appropriate construction observation can be present at the project site.

B. Prior to placing concrete, request review of reinforcement steel by A/E.

1.04 CODES AND STANDARDS

A. Comply with the following codes and standards, except as otherwise designated:

1. ACI 301 Specifications for Structural Concrete.
2. ACI 304 Guide for Measuring, Mixing, Transporting and Placing Concrete.
3. ACI SP-2 Manual of Concrete Inspection.

1.05 TESTING

A. Contractor shall arrange and pay for services of a qualified testing agency acceptable to Owner and independent of Contractor.

B. Testing agency shall test concrete to measure slump, entrained-air content, temperature, and compressive strength to determine compliance with specifications. Furnish test apparatus and cylinders, perform on-site sampling and testing, submit samples, and perform laboratory tests.

C. On-site tests shall be performed under observation of A/E unless waived.

D. Slump, Air Content, and Temperature Tests:

1. Perform slump, air content, and temperature tests prior to concrete placement each day, whenever there is a change in consistency of concrete, and when concrete cylinders are prepared.

2. Test for slump in accordance with ASTM C143, air content in accordance with ASTM C231, and temperature in accordance with ASTM C1064.
3. If measured slump, air content, or temperature falls outside specified limits, immediately check another portion of same batch. In event of a second failure, concrete shall be rejected.

E. Compressive Strength Tests:

1. During progress of work, prepare one set of test cylinders per 50 cu yd or fraction thereof for each class of concrete placed each day.
2. Obtain samples in accordance with ASTM C172. Cast, identify, transport, and cure cylinders in accordance with ASTM C31. Test strength of cylinders in accordance with ASTM C39.
3. If 6x12 in. cylinders are used, a set shall consist of three cylinders for laboratory curing. Obtain one 7-day and two 28-day compressive strength tests.
4. If 4x8 in. cylinders are used, a set shall consist of four cylinders for laboratory curing. Obtain one 7-day and three 28-day compressive strength tests.
5. If the requirements specified in the "Cold Weather Placing" article below apply, in addition to the above laboratory cylinders, a set shall also include two extra 6x12 in. or three extra 4x8 in. cylinders for field curing and laboratory testing. Obtain 28-day compressive strength tests on field cured samples.
6. If test results indicate deficiencies, A/E may require additional tests and may order remedial work.

PART 2 PRODUCTS

2.01 CEMENTITIOUS MATERIAL

A. Cement: Portland cement, ASTM C150, Type I.

B. Fly Ash: ASTM C618, Class C.

2.02 AGGREGATES

A. Fine and coarse aggregates, ASTM C33, consisting of clean, hard, durable sand and crushed rock, crushed gravel, or gravel.

B. Coarse aggregate shall meet grading requirements for size number 67, 57, or 467. Maximum coarse aggregate size for each class of concrete shall be as indicated in Part 4 Schedules. Ratio of coarse aggregate to fine aggregate shall not be less than 1:1 nor more than 2:1.

2.03 WATER

A. Mixing water shall be potable, free of oil, acid, excessive alkalinity, organic matter, and salts.

2.04 ADMIXTURES

A. Air-entrained admixture shall conform to ASTM C260.

B. Water reducing admixture shall conform to ASTM C494, Class A.

C. Other admixtures which do not adversely affect strength and durability of concrete may be used with permission of A/E, if used in strict accordance with manufacturer's instructions. Care shall be exercised to assure that the admixture does not increase or decrease air content outside of allowable limits. Do not use salt or chemical anti-freeze admixtures.

2.05 FORMWORK

A. Forms for Exposed Finish Concrete: Construct forms for exposed concrete surfaces with water-resistant plywood, metal, metal-framed plywood-faced, or other acceptable panel type materials, to provide continuous, straight, smooth, exposed surfaces. Furnish in largest practicable sizes to minimize number of joints and to conform to joint system if shown. Provide form material with sufficient thickness to withstand pressure of newly placed concrete without bow or deflection. Rust-stained steel forms are not acceptable.

B. Forms for Unexposed Finish Concrete: Construct forms for concrete surfaces which will be unexposed in finished structure with plywood, lumber, metal, or other acceptable material. Provide lumber that is dressed on at least one side and two edges for tight fit.

2.06 REINFORCING STEEL

A. ASTM A615, Grade 60, new billet steel bars.

B. Provide metal chairs, stirrups, spacers, and hangers to support reinforcement and insure against displacement during placement of concrete. All supports shall be plastic protected (Concrete Reinforcing Steel Institute [CRSI], Class 1) or stainless steel protected (CRSI, Class 2).

2.07 REINFORCING ADHESIVE

A. General: Diameter and embedment depth of adhesive anchors shall be as indicated on the Drawings. Embedment depth into sound concrete shall develop the yield strength of reinforcing bar.

B. Epoxy Adhesive: Hilti “HIT-RE 500-V3”, Powers Fasteners “Pure110+”, Simpson “SET-XP”, or approved equal.

C. Acrylic Adhesive: Hilti “HIT-HY 200”, Powers Fasteners “AC100+ Gold”, Simpson “AT-XP”, or approved equal.

2.08 EXPANSION JOINT FILLER

A. Premolded joint filler, ASTM D1751, asphalt-saturated cellulosic fiber; 1/2 in. thickness by depth of concrete, unless otherwise shown.

2.09 EXPANSION JOINT SEALANT

A. Urethane, one or two part, air curing, elastomeric sealant complying with ASTM C920, Type S or M, Grade NS (vertical joints) or Grade P (horizontal joints), Class 25.

2.10 WATERSTOP

A. PVC Waterstop: PVC 6 in. x 1/4 in., split bulb, ribbed type, unless otherwise shown. Use poly (vinyl chloride) with no reclaimed material. Provide factory-fabricated corners, intersections, and directional changes.

B. Hydrophilic Waterstop: Coiled, rope-like, cold joint waterstop impregnated with sodium bentonite which swells upon contact with water; CETCO "Waterstop RX", Sika "Swellstop", W.R. Meadows "Waterstop EC Plus", or approved equal.

2.11 MOISTURE-RETAINING COVER

A. Waterproof paper, polyethylene film, or polyethylene-coated burlap complying with ASTM C171.

2.12 CURING COMPOUND

A. White, waterborne, membrane-forming curing compound, ASTM C309, Type 2, Class B, dissipating.

2.13 CONCRETE MIXTURES

A. Conform to minimum standards for class and usage in Part 4 Schedules.

B. Prepare design mixes for each type of concrete on the basis of compressive strength by methods recommended in ACI 301. Use an independent materials laboratory for preparing and reporting proposed mix designs.

C. Provide water-reducing admixture for all concrete work. Provide air entraining admixture as scheduled.

PART 3 EXECUTION

3.01 FORMWORK INSTALLATION

A. Design, construct, erect, brace, and maintain formwork according to ACI 301.

B. Form 3/4 in. chamfers at corners to produce uniformly straight lines and tight edge joints. Extend terminal edges to required limit and miter chamfer strips at changes in direction. Unexposed corners may be formed either square or chamfered.

C. Coat contact surfaces of forms with form-release agent, according to manufacturer's written instructions, before placing reinforcement.

3.02 EMBEDDED ITEM INSTALLATION

A. Place and secure anchorage devices and other embedded items required for adjoining work that is attached to or supported by cast-in-place concrete. Use setting drawings, templates, diagrams, instructions, and directions furnished with items to be embedded.

B. Adhesive for reinforcing bars shall not be installed into concrete that is less than 21 days old.

3.03 SLAB SUBGRADE PREPARATION

A. Sprinkle subgrade with water.

3.04 STEEL REINFORCEMENT INSTALLATION

A. Comply with CRSI's "Manual of Standard Practice" for fabricating, placing, and supporting reinforcement.

B. Clean reinforcement of loose rust and mill scale, earth, ice, and other foreign materials that reduce bond to concrete.

C. Position reinforcement steel to provide minimum concrete cover as indicated on the Drawings. Unless otherwise specified on Drawings, minimum reinforcement cover shall be 3 in.

3.05 JOINTS

A. General: Construct joints true to line with faces perpendicular to surface plane of concrete.

B. Construction Joints: Install construction joints so that strength and appearance of concrete is not impaired, at locations shown or as approved by A/E.

C. Isolation and Expansion Joints: Provide expansion joint filler to isolate slabs-on-grade from old concrete, walls, and other vertical surfaces, and where shown.

D. Contraction Joints in Slabs: Provide contraction (control) joints in slabs-on-grade to form panels of patterns as shown. If joint pattern is not shown, provide joints not exceeding 15 ft in either direction and located to conform to bay spacing wherever possible (at column centerlines, half bays, third-bays).

1. Form contraction joints by inserting 1/4 in. x 1/4 slab depth premolded plastic, hardboard, or fiberboard strip into fresh concrete until top surface of strip is flush with slab surface. Tool slab edges round on each side of insert. After concrete has cured, remove inserts and clean groove of loose debris.
2. Contraction joints in unexposed slabs may be formed by 1/8 in. x 1/4 slab depth saw cuts. Saw joints prior to formation of shrinkage cracks; achieve an even crisp joint.

E. Contraction Joints in Walls: Provide contraction (control) joints in walls as shown. If not shown, provide a contraction joint within 10 to 15 ft of wall corner and at a maximum spacing of 25 ft thereafter.

1. Total depth of inside and outside contraction joints shall be 1/4 of wall thickness.

3.06 WATERSTOP INSTALLATION

A. PVC Waterstop: Provide continuous waterstop where shown on the Drawings and anywhere non-horizontal concrete joints extend upstream to downstream. Splice PVC waterstops by heat sealing adjacent surfaces in accordance with manufacturer's recommendations. Do not expose waterstop to a direct flame which could cause charring. Lap splices are not permitted. Embed approximately half of the waterstop on each side of joint. Support and protect waterstop during construction and repair or replace damaged waterstop.

B. Hydrophilic Waterstop: Provide hydrophilic waterstop where shown on the Drawings. Install waterstop in accordance with manufacturer's recommendations. Place joint material to water side of reinforcement; comply with minimum cover requirements.

3.07 MIXING CONCRETE

A. "Ready-mix" concrete shall be produced, delivered and handled in accordance with ASTM C94. Concrete shall be deposited at job site within one hour after introduction of water in mix. Care shall be taken in transferring concrete from truck or mixer to avoid segregation of aggregates in mixture.

3.08 CONCRETE PLACEMENT, GENERAL

A. Place concrete as specified and in accordance with ACI 301. Maintain reinforcing steel in proper position.

B. Deposit concrete continuously or in layers of such thickness that no concrete will be placed on concrete which has hardened sufficiently to cause formation of seams or planes of weakness. If a section cannot be placed continuously, provide construction joints. Deposit concrete as near as practicable to final location to avoid segregation from rehandling or flowing. Do not subject concrete to any procedure which will cause segregation.

C. Screed concrete to proper level to avoid excessive skimming or grouting.

D. Do not use concrete which becomes non-plastic and unworkable, or does not meet required quality control limits, or which has been contaminated by foreign materials. Do not use retempered concrete. Remove rejected concrete from project site.

E. Concrete shall not be placed around castings, frames, joints, and other embedded items until they have been accurately adjusted and set to required alignment and grade. Prior to placing

of concrete, castings, frames, and embedded metal fixtures shall be painted on their contact surface with a heavy coat of asphaltic mastic or separated with expansion joint filler.

3.09 PLACING CONCRETE INTO FORMS

A. Deposit concrete in forms in horizontal layers not deeper than 18 in. and in a manner to avoid inclined construction joints.

B. Remove temporary spreaders in forms when concrete placing reaches elevation of spreaders.

C. Consolidate concrete in forms by mechanical vibrating equipment and supplement by hand-spading, rodding or tamping. Use vibrators designed to operate at a speed of not less than 6000 impulses per minute when submerged in concrete. Vibration of forms and reinforcing will not be permitted.

D. Do not use vibrators to move concrete inside forms. Insert and withdraw vibrators vertically at uniformly spaced locations not farther than visibly effective. Do not insert vibrators into lower layers of concrete that have begun to set. At each insertion, limit duration of vibration to time necessary to consolidate concrete and complete embedment of reinforcement and other items without segregation of mix.

3.10 PLACING CONCRETE SLABS

A. Deposit and consolidate concrete slabs in a continuous operation, within limits of construction joints, until panel or section is complete.

B. Consolidate concrete during placing operations using mechanical vibrating equipment. Thoroughly work concrete around reinforcement and other embedded items and into corners. Consolidate concrete placed in beams and girders of supported slabs, and against bulkheads of slabs on ground, as specified for formed concrete structures. Consolidate concrete in remainder of slabs by vibrating bridge screeds, roller pipe screeds, or other acceptable methods. Limit time of vibrating consolidation to prevent bringing an excess of fine aggregate to surface.

C. Bring slab surfaces to correct level with straight edge and strike off. Use bull floats or darbies to smooth surface, leaving it free of humps or hollows. Do not sprinkle water on concrete surface while in plastic state. Do not disturb slab surfaces prior to beginning finishing operations.

3.11 COLD WEATHER PLACING

A. Protect concrete work from physical damage or reduced strength caused by frost, freezing actions, or low temperatures, in compliance with ACI 306 and as specified below.

1. When air temperature falls to or is expected to fall below 40 deg F, uniformly heat water and aggregates before mixing to obtain a concrete mixture temperature of not less than 60 deg F (50 deg F for sections thicker than 31 in.) and not more than 90 deg F at point of delivery.
2. Do not use frozen materials or materials containing ice or snow. Do not place concrete on frozen subgrade or on subgrade containing frozen materials. Ascertain that forms, reinforcing steel, and adjacent concrete surfaces are entirely free of frost, snow and ice before placing concrete.
3. During seasons when atmospheric temperature may be expected to drop below 40 deg F, concrete shall be protected by covering with impermeable paper and insulated blankets. Retain covering for seven days.
4. During seasons when atmospheric temperature may be expected to drop below 40 deg F, prepare extra concrete cylinders for field curing in accordance with “Testing” article above. Store field-cured cylinders in a location specified by A/E and in accordance with ACI 318 Section 26.5.3.2.

3.12 HOT WEATHER PLACING

A. When hot weather conditions exist that would seriously impair quality and strength of concrete, place concrete in compliance with ACI 305 and as specified below.

1. Cool ingredients before mixing to maintain concrete temperature at time of placement below 80 deg F. Mixing water may be chilled, or chopped ice may be used to control temperature provided water equivalent of ice is calculated in total amount of mixing water.
2. Cover reinforcing steel with water-soaked burlap if it becomes too hot, so that steel temperature will not exceed the ambient air temperature immediately before embedment in concrete.
3. Spray forms, reinforcing steel, and subgrade just before concrete is placed.
4. Do not use set-control admixtures, unless approved by A/E.

3.13 FINISHING FORMED SURFACES

A. Provide standard rough finish to formed surfaces to be concealed in finish work or by other construction, unless otherwise designated. Standard rough form finish shall be the concrete surface having texture imparted by form facing material, with defective areas repaired and patched and fins and other projections exceeding 1/4 in. in height rubbed down with wood blocks.

B. Provide standard smooth finish for formed surfaces exposed-to-view or to receive a covering applied directly or bonded to concrete, such as waterproofing, dampproofing, or paint. Standard smooth finish shall be the as-cast concrete surface obtained with form facing material, with defective areas repaired and patched and fins and other projections on surface completely removed and smoothed.

3.14 FINISHING UNFORMED SURFACES

A. Thoroughly float surface after concrete has been struck off. Check and level surface plane to a tolerance not exceeding 1/4 in. in 10 ft when tested with a 10 ft straightedge placed on surface at not less than two different angles.

B. Exterior walks and ramps shall have a lightly broomed surface with grain perpendicular to direction of travel.

C. Edges shall be neatly trimmed with 1/4 in. radius edging tool.

3.15 CURING

A. General: Protect freshly placed concrete from premature drying and excessive cold or hot temperatures. Cure formed and unformed concrete for seven days or until 75 percent of the required 28-day compressive strength is obtained, whichever is less, but no less than three days regardless of the compressive strength obtained from testing.

B. Moisture-Retaining Cover: Apply moisture-retaining cover to wet cure flat surfaces not receiving curing compound and other surfaces where formwork is removed prior to end of curing period.

C. Curing Compound: Apply white-pigmented membrane curing compound to exterior slabs, walks, and ramps, unless otherwise indicated.

3.16 REMOVAL OF FORMS

A. Remove forms from cast-in-place concrete only after concrete has achieved sufficient strength to support itself and superimposed loads; but in no case in less time than stated below.

B. Forms not supporting weight of concrete, such as sides of walls and similar parts of the work, may be removed 24 hours after placing concrete, provided concrete is sufficiently hard to not be damaged by form removal operations, and provided that curing and protection operations are maintained.

C. Forms supporting weight of concrete, such as elevated slabs and other structural elements, may not be removed in less than 14 days, and not until concrete has attained design minimum 28-day compressive strength.

3.17 EXPANSION JOINT SEALING

A. Provide joint sealant where designated on Drawings.

B. Thoroughly clean joints, removing foreign matter such as dirt, dust, moisture, paint, lacquer, grease, form release agents, bond breakers, deleterious curing compounds, or other surface irregularities or treatments.

C. Install sealant in accordance with manufacturer's specifications and recommendations. Prime or seal joint surfaces as recommended by sealant manufacturer. Clean adjacent surfaces soiled by sealant operations.

3.18 CONCRETE SURFACE REPAIRS

A. Formed Surfaces: Request inspection of concrete surfaces immediately after removal of formwork. After inspection, repair and patch defective areas as soon as practicable.

1. Remove bulges and projections by chipping or tooling. Rub or grind surface after removal.
2. For patching, use dry-pack mortar consisting of one part portland cement to 2-1/2 parts fine aggregate passing a No. 16 mesh sieve and only enough water as required for handling and placing. Cut out honeycomb, rock pockets, voids over 1/4 in. diameter, and holes left by tie rods and bolts. Remove defects down to solid concrete but, in no case, to a depth of less than 1 in. Make edges of cuts perpendicular to concrete surface or slightly undercut to provide a key at edge of cut. Thoroughly clean, dampen with water, and brush-coat area to be patched with bonding agent. Place patching mortar before bonding agent has dried. Compact mortar in place and strike off slightly higher than surrounding surface. For exposed-to-view surfaces, patch shall match color of surrounding concrete.
3. For patching less than 1 in. in depth, Contractor may propose a proprietary patching product in lieu of removing concrete to minimum depth of 1 in. Submit proposed product to A/E for approval.

B. Unformed Surfaces: Repair surfaces that do not meet specified requirements.

1. Correct low and high areas. Submit proposed repair products to A/E for approval.
2. Fill cracks with an approved epoxy mortar which will match slab finish in density and performance. Grind filler smooth and even with adjacent surface, free of bumps or depressions at joint.

PART 4 SCHEDULES

4.01 CLASSES OF CONCRETE

Class	Min. Comp. Strength @ 28 days, p.s.i.	Max. Slump*	Max. Agg. Size	Min. Cement, Bags/ C.Y.	Max. Water-Cement Ratio	Air Content, % By Volume
AA	4500	4"	3/4"	6**	0.45	5-8%

* Value before pumping concrete.

** Fly ash shall be used to constitute between 15 to 30% by weight of total cementitious material.

4.02 USAGE SCHEDULE

A. Use class AA concrete for all work.

END OF SECTION

SECTION 03 62 00

NON-SHRINK GROUTING

PART 1 GENERAL

1.01 SUMMARY

A. Provide non-shrink grouting as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

B. Use non-shrink grout for bedding and grouting precast concrete, structural steel, railings, machinery, equipment, column bases, pipe sleeves, and as designated.

1.02 SUBMITTALS

A. Product Data: Submit manufacturer's product data and installation instructions in accordance with Section 01 33 00.

PART 2 PRODUCTS

2.01 NON-SHRINK GROUT

A. Grout shall be non-metallic, non-gas forming, non-shrink, pre-blended and ready-to-use requiring only the addition of water at project site.

B. Provide grout complying with ASTM C1107; Euclid "Euco N-S Grout", L & M "Crystex", W.R. Meadows "Sealtight 588", Sika "SikaGrout 212", or approved equal.

2.02 WATER

A. Clean, potable, and free from oil, acid, alkalines, and organics.

PART 3 EXECUTION

3.01 FORMING

A. Provide forms of sufficient strength and securely anchored and shored to withstand pressure of grout and conditions during placement. Fit forms closely together and seal joints. Provide adequate clearance for ease of grout placement.

B. Forming should allow for a minimum of 1 in. head of grout above bottom of base plates.

3.02 PREPARATION

A. Clean grout contact surfaces of oil, grease, scale, and other foreign matter. Chip away unsound concrete leaving surface level but rough. Remove grease, oil, dirt, and coatings from underside of base plates of machinery, rails, and bolts.

B. Saturate concrete contact area with water 12 hours prior to grouting. Before placing grout, remove excess or free standing water.

3.03 MIXING

A. Mix grout in a conventional mortar mixer or blend using propeller type mixer and electric drill. Position mixing equipment as close as practical to area to be grouted with adequate means of transporting grout for quick and easy placement.

B. Mix water and grout in accordance with manufacturer's instructions. Amount of water used should be minimum quantity to produce desired grout consistency.

C. Mix only that quantity of grout that can be placed within 30 minutes after mixing.

3.04 GROUTING

A. Follow established concreting procedures observing precautions for hot and cold weather concreting.

B. Place grout using most practical method; poured in place or pressure grouted by gravity or plunger. Completely fill space to be grouted; keep grout thoroughly compacted and free from air pockets. When practical, place grout from one side flowing to opposite side to avoid air entrapment.

C. Do not remove forms until grout has taken an initial set and will not slump. After removal, cut off excess grout and finish to a smooth surface.

D. Prevent rapid loss of water from grout during first 48 hours by covering with wet burlap or polyethylene sheet or by use of an approved membrane curing compound.

E. Remove shims or leveling devices after curing for 48 hours.

END OF SECTION

SECTION 05 50 00

METAL FABRICATIONS

PART 1 GENERAL

1.01 SUMMARY

A. Provide metal fabrications as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 COORDINATION

A. Furnish inserts and anchoring devices to be embedded in concrete for installation of miscellaneous metal work. Provide setting drawings, templates, and instructions for installation of anchorage devices. Coordinate delivery with related work to avoid delays.

B. See concrete section for installation of inserts and anchors.

1.03 SUBMITTALS

A. Shop Drawings: Submit shop drawings for fabrication, delivery, and erection of metal assemblies. Include product data, load tables, layouts, elevations, details of sections, connections, anchorage, and accessory items. Provide templates for anchors and bolts installed under other sections.

B. Welder/Weld Certifications: Submit certificate of AWS or ASME Welder at least 2 weeks before welding is scheduled to start. No welding shall be done until such certificates are received.

C. Mill Certificates: Furnish certified mill tests of chemical and physical properties of steel used in fabrication.

D. Testing Report: Submit a report documenting weld quality assurance testing.

E. Make submittals in accordance with Section 01 33 00.

1.04 QUALITY ASSURANCE

A. Establish and maintain a quality control program to assure compliance with these specifications and the drawings. Maintain records of quality control tests and inspections. Contractor shall be responsible for 100% visual inspection and necessary correction of all weld deficiencies.

B. Arrange and pay for services of an accredited testing laboratory approved by the Company representative to inspect weld quality. All defects revealed as a result of tests shall be rectified by Contractor to the satisfaction of the Company representative and at no additional cost to Company.

C. Nondestructive Examination of Welds: Completed welds and weld profiles shall be 100 percent visually examined and shall be in accordance with requirements of AWS D1.1.

D. Magnetic Particle Examination: Magnetic particle examination shall be made on all groove welds and a random 15 percent of fillet welds in accordance with AWS standards.

1.05 FIELD MEASUREMENTS

A. Take measurements prior to shop fabrication. Allow for trimming and fitting to make field adjustments. Correct defects resulting from failure to take proper measurements.

PART 2 PRODUCTS

2.01 STEEL

A. Plate: ASTM A572, Grade 50.

B. Other Shapes, Plates, and Bars: ASTM A36 or ASTM A572, Grade 50.

C. Tubing: ASTM A500, Grade C, cold-formed steel tubing.

2.02 STAINLESS STEEL

A. Sheet, Strip, and Plate: ASTM A240 or ASTM A666, Type 304 or Type 316.

B. Bars and Shapes: ASTM A276, Type 304 or Type 316.

C. Tubing: ASTM A554, Grade MT304 or Grade MT316.

D. Pipe: ASTM A312, Grade TP304 or Grade TP316.

2.03 EPOXY ADHESIVE ANCHORS

A. Description: Epoxy adhesive anchors with threaded rods, nuts, and washers.

B. Performance Requirements: Diameter and embedment depth of anchors shall be as indicated on the Drawings.

C. Adhesive: Adhesive shall be Hilti "HIT-RE 500-V3".

D. Rods, Nuts, and Washers: Anchor rods shall be Hilti HAS-E-55 Rod, ASTM F1554, Grade 55, zinc-plated. Anchor rod shall include nuts and washers supplied by rod manufacturer that are compatible with rod.

2.04 FASTENERS

A. Zinc-Plated Steel Fasteners: Regular hexagon-head bolts, ASTM F3125, Grade A325, Type 1; with hex nuts, ASTM A563; and, where indicated, flat washers, ASTM F436; with coating complying with ASTM B633 or ASTM F1941, Class Fe/Zn 5.

B. Stainless Steel Fasteners: Regular hexagon-head annealed stainless steel bolts, ASTM F593, with hex nuts, ASTM F594; and, where indicated, flat washers; Alloy Group 1 or Alloy Group 2.

2.05 FABRICATION

A. Use materials of designated type, size, and thickness or, if not shown, of required strength, stiffness, and durability. Work to field measurements and shop drawings, using proven details of fabrication and support. Miscellaneous framing and support members shall comply with AISC Specification.

B. Where exposed to view, use materials that are smooth and free of surface blemishes such as pitting, seam marks, roller marks, rolled trade names, and roughness.

C. Form work true to line and level with accurate angles and surfaces and straight sharp edges. Ease exposed edges to a radius of approximately 1/32 in. unless otherwise shown. Form bent-metal corners to smallest radius possible without causing grain separation or other impairment. Shearings and punchings shall be clean and true.

D. Weld corners and seams continuously; comply with AWS recommendations. Grind exposed welds smooth and flush; match and blend with adjoining surfaces. Weld discoloration of exposed surfaces is not acceptable.

E. Form exposed connections with hairline joints, flush and smooth, using concealed fasteners wherever possible. Use exposed fasteners of type shown or, if not shown, Phillips flathead (countersunk) screws or bolts.

F. Provide anchorage devices and fasteners for securing miscellaneous metal items to in-place construction, including threaded fasteners for concrete and masonry inserts, toggle bolts, through-bolts, lag bolts, wood screws, and other connectors as required. Cut, reinforce, drill, and tap miscellaneous metal work as required to receive hardware and connections required by other trades.

G. Preassemble and fit items to minimize field splicing and assembly. Disassemble units only as necessary for shipping and handling limitations. Clearly identify units for reassembly and installation.

2.06 SHOP PAINTING

A. Painting shall be in accordance with Section 09 90 00.

2.07 STAINLESS-STEEL FINISHES

A. Remove tool and die marks and stretch lines or blend into finish.

B. Provide dull satin finish No. 6, unless otherwise indicated.

C. When finishing is completed, passivate and rinse surfaces. Remove embedded foreign matter and leave surfaces chemically clean.

PART 3 EXECUTION

3.01 INSTALLATION

A. Install anchorage devices and fasteners for adequate support.

1. Expansion anchors shall not be installed into concrete that is less than 7 days old.
2. Adhesive anchors shall not be installed into concrete that is less than 21 days old.

B. Perform cutting, drilling, and fitting, as required. Set work accurately in location, alignment, and elevation, plumb, level, true, and free of rack, measured from established lines. Provide temporary bracing or anchors in formwork for items to be built into concrete.

END OF SECTION

SECTION 05 52 04

STEEL RAILINGS

PART 1 GENERAL

1.01 SUMMARY

A. Provide steel railings and handrails as shown and as specified. Comply with applicable provisions of Divisions 00 and 01 and Section 05 50 00.

1.02 SUBMITTALS

A. Shop Drawings: Submit shop drawings and product data in accordance with Section 01 33 00.

PART 2 PRODUCTS

2.01 STEEL PIPE

A. ASTM A53; Type E or S, Grade B, black finish, Sch. 80.

2.02 SAFETY RAILINGS

A. Fabricate welded steel railings from 1-1/2 in. round steel pipe, unless otherwise shown. Welds shall be ground smooth and flush.

B. Post spacing shall not exceed 4 ft, unless otherwise shown. Where not shown otherwise, top of top rail shall be 42 in. above floor or 34 in. above stair treads measured vertically at stair riser line.

C. Form simple and compound curves by bending pipe in jigs to produce uniform curvature for each repetitive configuration required; maintain cylindrical cross-section of pipe throughout entire bend without buckling, twisting or otherwise deforming exposed surfaces of pipe.

D. Provide wall returns at ends of wall-mounted handrails, except where otherwise indicated.

E. Close exposed ends of pipe by welding 3/16 in. thick steel plate in place or by use of prefabricated fittings.

2.03 BRACKETS, FLANGES, FITTINGS AND ANCHORS

A. Provide wall brackets, end closures, flanges, miscellaneous fittings and anchors for interconnections of pipe and attachment of railings and handrails to other work. Furnish inserts and other anchorage devices for connecting railings and handrails to concrete or masonry work.

2.04 PRIMER

A. Shop prime railing and accessories (except galvanized items) with one coat rust-inhibitive, lead and chromate free, alkyd or modified alkyd primer, unless otherwise designated in Section 09 90 00. Primer shall be compatible with finish coats specified.

2.05 NONSHRINK GROUT

A. Grout shall be non-shrink, non-metallic, non-gas forming, pre-blended and ready-to-use requiring only the addition of water at project site. Provide grout complying with CRD-C621;

Euclid "Euco N-S Grout", L & M "Crystex", Meadows "Sealtight 588", Sika "SikaGrout 212", Sonneborn "Sonogrout", or approved equal.

PART 3 EXECUTION

3.01 INSTALLATION

A. Installation shall be in accordance with shop drawings and manufacturer's recommendations. Perform cutting, welding, and fitting required for installation. Set the work accurately in location, alignment, and elevation. Install plumb, level, true, and free of rack.

B. Fit exposed connections accurately together to form tight hairline joints. Weld connections which cannot be shop-welded because of shipping size limitations. Grind joints smooth and touch-up shop paint coat.

C. Adjust railings prior to securing in place to ensure proper matching at butting joints and correct alignment throughout their length. Space posts and supports as shown. Plumb posts in each direction. Secure posts and rail ends to building construction as shown. Fasteners shall be tightened so completed railing is rigid and completely free of play at joints and attachments. For railing posts inserted in sleeves set in concrete, fill annular space between post and sleeve with non-shrink, non-metallic grout.

3.02 PROTECTION FROM ENTRAPPED WATER

A. For exterior use, provide drains from railing system to prevent damage caused by freezing of entrapped water. For interior installations subject to high humidity, make provision to drain water from railing system.

B. When posts are mounted into concrete or when bends or elbows occur at low points, drill 1/8 in. diameter weep holes at lowest elevations.

3.03 EXPANSION JOINT INSTALLATION

A. Provide expansion joints at locations indicated, or if not indicated, at intervals not to exceed 40 ft. Provide slip joint with internal sleeve extending 2 in. beyond joint on either side; fasten internal sleeve securely to one side; locate joint within 6 in. of posts.

END OF SECTION

SECTION 09 90 00

PAINTING AND COATING

PART 1 GENERAL

1.01 SUMMARY

A. Provide painting and coating as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 WORK INCLUDED

A. Work includes:

1. Shop priming and shop finish painting trash rack.

1.03 DEFINITIONS

A. Applicable abbreviations are:

1. DFT: Dry Film Thickness.
2. SSPC: The Society for Protective Coatings.

1.04 SUBMITTALS

A. Product Data: Submit manufacturer's specifications, including label analysis and application instructions for each material specified.

B. Color Charts: Furnish color charts for selection of paint colors.

C. Make submittals in accordance with Section 01 33 00.

PART 2 PRODUCTS

2.01 PAINT AND COATINGS

A. Provide products by one of the following or approved equal:

1. Tnemec.
2. Sherwin-Williams Co. (S-W).
3. Carboline.
4. PPG Protective & Marine Coatings.

B. Contractor shall submit a specific list of products he wishes to use if manufactured by a company other than that noted in Painting Schedule.

C. Refer to Drawings and painting schedule for finishes and coating systems to be applied to various surfaces.

PART 3 EXECUTION

3.01 SUBSTRATE EXAMINATION

A. Examine substrates and surfaces and conditions under which work is to be performed. Do not proceed with this work until unsatisfactory conditions have been corrected.

B. Do not paint over dirt, rust, scale, grease, moisture, scuffed surfaces, or conditions otherwise detrimental to formation of a durable paint film.

3.02 SURFACE PREPARATION

A. Perform preparation procedures for each substrate in strict accordance with paint manufacturer's instructions and as specified.

B. Remove all hardware, hardware accessories, and similar items in place and not to be painted, or provide surface-applied protection prior to surface preparation and painting operations. Remove mounted accessories if necessary for complete painting of items or adjacent surfaces. Following completion of painting, reinstall removed items.

C. Clean surfaces to be painted before applying paint or surface treatments. Remove oil and grease prior to mechanical cleaning. Program cleaning and painting so that contaminants from cleaning process will not fall onto wet, newly-painted surfaces.

3.03 MATERIAL PREPARATION

A. Prepare painting materials in accordance with manufacturer's directions. Mix materials before application to produce uniform density. Stir as required during application of materials. Do not stir surface film into material; remove film and, if necessary, strain material before using.

B. Store materials not in actual use in tightly covered containers. Maintain containers used in storage, mixing and application of paint in a clean condition, free of foreign materials and residue.

3.04 APPLICATION

A. Apply paint in accordance with manufacturer's directions. Use applicators and techniques best suited for type of material being applied.

B. The finished paint coating shall be free from holidays, pinholes, bubbles, runs, drops, ridges, waves, laps, unnecessary brush marks, and variations in color, texture, and gloss. All paint coats shall be applied in such manner as to produce an even, continuous film of uniform thickness. Edges, corners, crevices, seams, joints, welds, and other surface irregularities shall receive special attention to ensure that they receive an adequate thickness of paint.

C. Paints shall be applied only to surfaces that are above the dewpoint temperature and that are completely free of moisture. The temperature of the surfaces to be painted and of air in contact with them shall be not less than 50 deg. F. during paint application nor shall paint be applied if the surfaces can be expected to drop to 50 deg. F. before the film has dried to a reasonably firm condition. Paint shall not be applied to surfaces heated by direct sunlight or other sources to temperatures that will cause detrimental blistering, pinholing, or porosity of the film.

PART 4 SCHEDULES

4.01 PAINTING SCHEDULE

A. Coating Systems: Provide the following coating systems for the various substrates indicated. Named products are specified to establish a standard of type and quality. See "Paint and Coatings" article, above, for a complete list of acceptable manufacturers.

B. Ferrous Metal: Use for ferrous metal trash rack:

1. Wasser:

a. System Type: Moisture-cured urethane.
b. Surface Preparation: r SSPC SP-10 with a 2-3 mil blast profile.
c. Primer: One coat Wasser MC-Zinc at 3.0 mils DFT.
d. Finish: Two coats Wasser MC-Tar at 6.0 mils DFT per coat.
e. Total DFT: 15.0 mils.

2. Sherwin-Williams:

a. System Type: Moisture-cured urethane.
b. Surface Preparation: r SSPC SP-10 (with a 2-3 mil blast profile.
c. Primer: One coat S-W Corothane I Galvapac Zinc at 3.0 mils DFT.
d. Finish: Two coats S-W Corothane I Coal Tar at 6.0 mils DFT per coat.
e. Total DFT: 15.0 mils.

END OF SECTION

SECTION 26 05 43

UNDERGROUND CONDUIT AND PULL BOXES

PART 1 GENERAL

1.01 SUMMARY

A. Provide underground conduit and pull boxes for future use by Owner as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

PART 2 PRODUCTS

2.01 CONDUIT

A. Conduit shall be in accordance with CODOT Std. Spec., Section 613, for rigid nonmetallic conduit, Sch. 40.

2.02 PULL BOXES

A. Pull shall be in accordance with C)DOT Std. Spec., Section 604.

2.03 PULL BOXES COVER

A.. Furnish pull box entrance covers and frames as shown. Frames shall be installed as shown on drawings.

B. Cover: 1/4“ thick aluminum diamond plate, 36”x36”, 300 psf live load rating, mill finish, lifting mechanism with reinforced composite tubes and electro coated compression springs, automatic hold open arm, forged aluminum hinges with stainless steel pins, neoprene gasketed standard slam lock with keyed cylinder lock and underside release knob. Cover and frames shall be Bilco Type J-AL, Nystrom FDDP, Halliday Type WIR, or approved equal.

C. Frame: 1/4” thick extruded aluminum channel with continuous concrete anchor, with continuous EPDM debris gasket and 1-1/2” aluminum coupling for drain.

D. Guarantee: 10-year material.

PART 3 EXECUTION

3.01 CONDUIT INSTALLATION

A. Install underground conduits in accordance with CODOT Std. Spec., Section 613. Minimum cover shall be as indicated on the Drawing details.

B. Adequately compact trench backfill to density of surrounding soils. No rocks larger than 1 in. shall be present in backfill material.

C. Provide conduits with a nylon or polypropylene pull line.

3.02 PULL BOX INSTALLATION

A. Install pull boxes in accordance with CODOT Std. Spec., Section 604.

3.03 PULL BOX COVER INSTALLATION

A. Install pull boxes cover in accordance with manufacturers recommendations.

END OF SECTION

SECTION 31 05 10

SITE PREPARATION

PART 1 GENERAL

1.01 SUMMARY

A. Provide site preparation as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

B. Work includes, but is not limited to:

1. Protecting improvements, plants, and utilities.
2. Temporarily removing and replacing improvements.
3. Locating utilities and coordinating with utility companies.
4. Clearing and grubbing trees and vegetation.
5. Salvaging topsoil.
6. Performing site demolition and abandonments.

PART 2 (NOT USED)

PART 3 EXECUTION

3.01 PROTECTION

A. Protect improvements on site and on adjoining properties. Provide barricades, coverings, or other types of protection as necessary to prevent damage and to safeguard against injury. Restore to original condition improvements damaged by the work or improvements which required temporary removal during construction.

B. Protect existing vegetation indicated to remain against unnecessary cutting, breaking, bruising, or smothering by stockpiling excavated materials or parking of vehicles within drip line. Provide temporary fences, tree wells, barricades, or guards; repair or replace trees and vegetation damaged by construction operations.

C. Maintain survey monuments, reference points, and benchmarks; notify Owner of disturbance to markers.

D. No extra payment or time will be allowed for protection work that could have been suspected or anticipated by site inspection and interpretation of bidding documents prior to execution of contract.

3.02 LOCATING EXISTING UTILITIES

A. Location and description of underground utilities and structures shown on drawings are approximate and are based on records available to Owner or surface features indicating their existence. There may be other utilities within project area that are not shown.

B. Notify all affected utility companies of construction operations at least three working days before beginning work near their facilities. Do not begin excavation work until underground utility locations have been marked.

C. Use caution when excavating so that exact location of underground utilities, both known and unknown, may be determined. Provide adequate protection and support for utilities during construction operations.

D. If uncharted or incorrectly charted utilities are encountered during excavation work, or if proposed construction conflicts with existing utilities, give prompt notice and submit proposed solution to A/E for approval. Cooperate with Owner and public and private utility companies to keep their services and facilities in operation. Repair damaged utilities to satisfaction of utility owner.

3.03 SITE CLEARING

A. Remove trees, stumps, snags, shrubs, brush, heavy growths of grass, weeds and other vegetation, improvements, rubbish and debris, and obstructions that interfere with proposed construction; remove items only as necessary for completion of work.

B. Cut brush and vegetation flush with ground. Grub out stumps, roots having a diameter of 2 in. or larger, and root clusters to a depth of at least 2 ft below subgrade elevation for pavements, structures, and embankments and 6 in. below ground surface in other areas.

C. Carefully and cleanly cut roots and branches of trees indicated to be left standing, where such roots and branches obstruct new construction. Cut back roots a minimum of 1 ft from concrete work, paving, and structures and to a depth of not less than 2 ft below structures, foundations, and embankments.

3.04 TOPSOIL STRIPPING

A. Topsoil shall include all friable, fertile, loam soil suitable for grass and plants, found at surface, reasonably free of subsoil, clay lumps, stones, objects over 2-in. diameter, weeds, large roots, root clusters, and other objectionable material.

B. Strip topsoil from project area to whatever depths encountered; prevent intermingling with underlaying subsoil or other objectionable material. Remove heavy growths of grass from areas before stripping topsoil.

C. Where trees are indicated to remain, terminate stripping a sufficient distance from such trees to prevent damage to root system.

D. Stockpile topsoil in storage piles in areas where designated. Construct storage piles to freely drain surface water. Cover or sprinkle water on storage piles to prevent windblown dust.

3.05 DEMOLITION

A. Remove structures, pavements, utilities, and other improvements within construction limits as shown and as required for construction.

B. If removing only a portion of pavement, curb, gutter, sidewalk, or similar surface improvements, remove the improvement to an existing joint or saw cut the improvement to provide a smooth straight joint.

C. Owner shall have first right to retain all useful salvage. All items not retained by Owner and construction debris shall become property of Contractor.

3.06 DEBRIS DISPOSAL

A. Remove debris and excess materials from site and legally dispose of it; do not burn debris.

END OF SECTION

SECTION 31 20 00

EARTH MOVING

PART 1 GENERAL

1.01 SUMMARY

A. Provide earth moving as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 RELATED SECTIONS

31 05 10 Site Preparation.
31 23 33 Trenching and Backfilling.
31 24 00 Embankments.
33 41 66 Drain Fill.
33 42 15 Piping and Accessories.

1.03 CLASSIFICATION

A. Excavation of materials encountered under this work will be unclassified without regard to type, difficulty to remove, or suitability for use in construction.

1.04 SUBMITTALS

A. Test Reports: Submit reports for laboratory and field tests required under "Testing" article. Test reports for footing, slab, and pavement subgrades shall be submitted prior to placing concrete or paving materials. Make submittals in accordance with Section 01 33 00.

1.05 TESTING

A. Contractor shall arrange and pay for soil sampling and testing by a qualified testing agency, acceptable to Owner and independent of Contractor.

B. Test subgrade and fill materials for gradation in accordance with ASTM C136 for conformance with ASTM D2487 gradation limits. Test materials for liquid limit and plasticity index in accordance with ASTM D4318.

C. Provide one optimum moisture-maximum density curve for each type of soil encountered in subgrade and fills under foundations, structure slabs, and paved areas; determine maximum densities in accordance with ASTM D1557.

D. During course of work, testing agency shall inspect and approve subgrades and fill layers before further construction work is performed on each layer. Perform field density tests in accordance with standard, recognized procedures. Take tests as follows:

1. Footing Subgrade: Perform at least one field test for every 10,000 sq ft of structure area, but in no case less than three tests, to verify that the bearing capacity indicated on the Drawings is present.
2. Structure Slabs and Paved Areas: Perform at least one field density test on fill subgrade for every 2000 sq ft of structure slab or paved area, but in no case less than three tests. In each compacted fill layer, perform at least one field density test for every 2000 sq ft of overlaying structure slab or paved area, but in no case less than three tests.
3. Foundation Wall Backfill: Perform at least two field density tests at locations and elevations as directed.

E. If in opinion of A/E, based on reports of testing agency and inspection, subgrade or fills which have been placed are below specified density, provide additional compaction and testing at no additional cost to Owner.

1.06 PROTECTION

A. Protect existing improvements, utilities, trees and shrubs, and reference marks in accordance with Section 31 05 10.

1.07 BLASTING

A. Use of explosives is not permitted.

PART 2 PRODUCTS

2.01 SOIL MATERIALS, GENERAL

A. Soil materials shall be free of organic matter, debris, frozen soils, ice, and other objectionable materials. Rock particles larger than maximum size specified shall be removed prior to placement of soil.

B. Select existing material from required excavations may be used for fill or backfill if it meets the specified product requirements. If necessary, furnish additional approved material from suitable off-site sources.

2.02 GRANULAR FILL

A. Select soils complying with ASTM D2487 soil classification groups GW (well-graded gravel), GP (poorly-graded gravel), SW (well-graded sand), or SP (poorly-graded sand). Aggregate shall pass a 1-1/2-in. sieve and not more than 35% shall be retained on a No. 10 sieve. Maximum 5% by weight shall pass a No. 200 sieve.

2.03 STRUCTURAL FILL

A. Select soils complying with ASTM D2487 soil classification groups GW, GP, SW, or SP; or these groups in combination with groups GM, GC, SM, or SC (dual symbol soils). Aggregate shall pass a 1-1/2-in. sieve and not more than 35% shall be retained on a No. 10 sieve. Maximum 12% by weight shall pass a No. 200 sieve; plasticity index shall not exceed 5.

2.04 GENERAL SITE FILL

A. Select, natural, free draining soils complying with ASTM D2487 soil classification groups GW, GP, SW, SP, GM, GC, SM, SC, or combinations thereof, and suitable for compaction. Maximum aggregate size shall be 1/2 specified lift thickness.

2.05 GRANULAR BEDDING AND BACKFILL

A. Select soils suitable for use as Granular Fill, except coarse aggregate shall pass a 3/4-in. sieve.

PART 3 EXECUTION

3.01 PREPARATION

A. Prepare site for work in accordance with Section 31 05 10.

B. Layout and stake lines and grades as required to complete the work.

C. Layout of work is to be performed and certified in writing by a licensed surveyor.

3.02 EXCAVATION FOR STRUCTURES

A. Excavate to achieve necessary dimensions, lines, and grades. Conform to elevations and dimensions shown within a tolerance of plus or minus 1 in., and extending a sufficient distance from footings and foundations as required for bracing and supports, concrete formwork, installation of services, other required construction, and for inspection.

B. For footings and foundations, take care not to disturb bottom of excavation. Excavate to final grade just before concrete is placed. Trim bottoms to required lines and grades to leave solid, undisturbed base to receive granular fill, base course, or concrete as shown.

3.03 TRENCHING

A. Excavate trenches so that pipe can be laid safely and accurately to required line and grade. Hand excavate for bells, fittings and projections to allow for proper jointing and to ensure that pipe rests evenly along barrel and is not resting on bell.

B. In sand and gravel soils, bottom of trench may be shaped to fit bottom 1/3 of pipe. In silt or clay soils, bottom of trench shall be 4 in. below pipe barrel and 3 in. below bell. In rock, bottom of trench shall be 6 in. below pipe barrel. Under foundations and footings, bottom of trench shall be 8 in. below pipe. Provide Granular Bedding as specified below.

C. Trench widths in ordinary soil shall be limited at top of pipe to not less than a 6 in. clearance on either side of barrel to allow for installation of bedding material between pipe and trench wall. Maximum trench width at top of pipe shall be outside pipe diameter plus 24 in. (30 in. minimum). Trench above top of pipe may be sloped, stepped or vertical to comply with state and federal regulations regarding trenches.

D. Minimum trench width in rock shall not be less than that for ordinary soil. Maximum trench width shall be outside pipe diameter plus 18 in. for an unsheathed trench, and outside pipe diameter plus 24 in. for sheathed trench.

3.04 UNAUTHORIZED EXCAVATION

A. Unauthorized excavation consists of removal of materials beyond indicated elevations or side dimensions without specific direction of A/E. Unauthorized excavation, as well as remedial work, shall be at Contractor's expense. Notify A/E prior to backfilling if unauthorized excavations are made.

B. Under footings, foundations, underpinning, equipment bases, and retaining walls, fill unauthorized excavation by extending indicated bottom elevation of footing or base to excavation bottom, without altering required top elevation. Lean concrete or compacted fill may be used to bring elevations to proper position when approved by A/E.

C. Elsewhere, backfill and compact unauthorized excavations as specified for authorized excavations of same classification, unless otherwise directed.

3.05 STABILITY OF EXCAVATIONS

A. Maintain sides and slopes of excavations in a safe condition until completion of backfilling. Slope sides of excavations to angle of repose of material excavated; otherwise, shore and brace where sloping is not possible either because of space restrictions or stability of material excavated. Take precautions to prevent slides or cave-ins when excavations are made in locations adjacent to backfilled excavations, and when sides of excavations are subjected to vibrations from traffic, machinery, or any other source. Comply with applicable codes and ordinances.

3.06 SHORING AND BRACING

A. Carry down shoring and bracing as required as excavation progresses. Maintain shoring and bracing while excavations are open.

B. Provide and maintain shoring and bracing, such as sheet piling, uprights, stringers and cross-braces, in good serviceable condition. Use timbers that are sound and free of large or loose knots.

C. Provide permanent steel sheet piling or pressure treated timber sheet piling wherever subsequent removal of sheet piling might permit lateral movement of soil under adjacent structures. Cut off tops as required and leave permanently in place.

3.07 DEWATERING

A. Perform earthwork in a manner to prevent surface water and ground water from flowing into excavations. Promptly remove water from excavations using pumps, sumps, and dewatering system components necessary to convey water away from excavations. If underground springs are encountered, notify A/E before proceeding.

B. Convey water removed from excavations and rain water to collection or run-off areas. Provide and maintain temporary drainage ditches and other diversions outside excavation limits for each structure. Do not use foundation or utility trench excavations as temporary drainage ditches.

C. Provide filter material, trash screens, and other devices around pumps and intakes to avoid pumping or discharging sediment from construction site.

3.08 STOCKPILING

A. Stockpile excavated materials meeting the requirements for fill and backfill where directed until required for the work. Place, grade, and shape stockpiles for proper drainage. Locate stockpiles a sufficient distance from edge of excavations, even though such excavations may be sheeted and braced, to prevent such material from falling or sliding into excavations and to prevent cave-ins.

3.09 COLD WEATHER PROTECTION

A. Protect excavation bottoms against freezing when atmospheric temperature is less than 35 deg F by covering with dry insulating materials of sufficient depth to prevent frost penetration.

3.10 SUBGRADE EXAMINATION AND PREPARATION

A. Examine subgrade prior to placing fill. Remove organic materials and debris subject to rot or corrosion. Plow, strip, or break-up sloped surfaces steeper than 1 vertical to 4 horizontal so that fill material will bond with subgrade.

B. In structure and pavement areas, proof-roll exposed subgrade in overlapping passes in a perpendicular grid pattern with a fully-loaded tandem-axle dump truck weighing not less than 10 tons, or other equipment of similar size and weight, to compact subgrade and detect areas which must be undercut or improved. Inform A/E of unsuitable, unconsolidated subgrade soils.

C. After subgrade soil is stable, scarify top 6 to 8 in., moisture condition, and compact surface to density specified in Part 4 Schedules.

D. Reconstruct subgrades damaged by freezing temperatures, frost, rain, accumulated water, or construction activities, as directed by A/E, without additional compensation.

3.11 ADDITIONAL EXCAVATION (OVER EXCAVATION)

A. If unsuitable bearing materials, such as poorly compacted fill, existing foundations, rubble, debris, or organic deposits, are encountered at required subgrade elevations, carry excavations deeper and replace excavated material with properly compacted Structural Fill as directed by A/E.

B. Where over excavation below footing subgrade is required, widen over excavation beyond footing edges at least 1 ft for each 1 ft of over excavation depth.

C. Removal of unsuitable material and its replacement as directed will be paid for as extra work, unless a pay item is included in the Bid Schedule. Do not proceed with extra or unit price work until authorized.

3.12 FILLING AND BACKFILLING, GENERAL

A. Do not place fill until required subgrade preparation has been examined and approved by testing agency.

B. Backfill excavations as promptly as work permits, but not until completion of the following:

1. Acceptance by A/E of construction below finish grade including, where applicable, dampproofing, waterproofing, and perimeter insulation.
2. Inspection, testing, approval, and recording locations of underground utilities.
3. Removal of concrete formwork.
4. Removal of trash and debris.
5. Removal of temporary shoring and bracing, and backfilling of voids with satisfactory materials. Cut off temporary sheet piling driven below bottom of structures and remove in manner to prevent settlement of the structure or utilities, or leave in place if required.
6. Placement of permanent or temporary horizontal bracing on horizontally supported walls.

C. Place fill or backfill in approximately horizontal layers; do not exceed the maximum lift thickness specified in Part 4 Schedules before compaction. Spread piles and windrows uniformly.

D. Adjacent to structures, place fill or backfill to prevent damage and allow structures to assume loads gradually and uniformly, at approximately the same rate on all sides of structure. Adjacent to earth-retaining structures, do not place fill or backfill until concrete has reached specified 28-day compressive strength (minimum 14 days). Do not travel heavy equipment over cast-in-place concrete work until it has reached specified 28-day compressive strength (minimum 14 days), unless otherwise approved.

3.13 SOIL FILL

A. Place and compact fill materials in layers to required elevations as follows:

1. Under turf and planted areas: Use General Site Fill.
2. Under footing, foundation, building slab, pavement, and walk areas: Use Structural Fill.
3. For upper 6 in. immediately under building slabs and walks: Use Granular Fill.
4. For backfill behind retaining walls: Use Granular Fill.

B. Do not place soil fill on frozen subgrades.

3.14 TRENCH BEDDING AND BACKFILL

A. Trenches dug in sandy or gravelly materials may use undisturbed earth for bedding provided surface is shaped to conform to pipe. Provide Granular Bedding in all other trenches from subgrade to a point supporting bottom 1/3 of pipe for rigid pipe and to springline (mid-height) for flexible pipe. Place and compact bedding so that it fills and supports pipe haunch area.

B. Immediately after installation of pipe, provide tamped Granular Backfill up to a minimum depth of 1 ft above pipe. Take special care in placing and tamping initial backfill material so alignment and grade of pipe is not disturbed nor pipe damaged.

C. Backfill more than 1 ft over pipe shall meet material requirements for area in which pipe is located.

3.15 GRADING

A. Grade areas within project limits to achieve cross sections, lines, and elevations indicated. Slope grades to direct water away from structures and to prevent ponding. Finish surface to be reasonably smooth and free from irregular surface changes. Provide a smooth transition between adjacent existing grades and new grades.

B. Finish subgrades to required elevations within the following tolerances:

1. Turf and Unpaved Areas: Plus or minus 1 in.
2. Pavements and Walks: Plus or minus 0.5 in.
3. Structure Slabs: Tolerance of 0.5 in. when tested with 10 ft straightedge.

3.16 CONTROL OF MOISTURE CONTENT

A. During placement and compaction, maintain moisture content of materials within optimum range.

B. Apply water to fill materials by sprinkling materials at borrow site or after placement on fill if necessary. Obtain uniform moisture distribution by discing, blading or other approved methods prior to compaction of layer.

C. If material is too wet when deposited on fill, remove or dry it to specified moisture content prior to compaction.

D. If top surface of a preceding layer of compacted fill becomes too dry to permit suitable bond, scarify and moisten it by sprinkling to an acceptable moisture content prior to placement of next layer of fill.

3.17 COMPACTION

A. Compact each layer of soil material to not less than the percentage of maximum density specified in Part 4 Schedules.

B. Provide compaction equipment required to obtain specified compaction. Compaction by travel of grading equipment is not considered adequate for uniform compaction. Small vibratory compactors are required wherever fill is placed adjacent to foundation walls, footings, and piers. Pipe bedding and initial backfill shall be hand or mechanically tamped.

3.18 MAINTENANCE

A. Where completed compacted areas are disturbed by subsequent construction operations or adverse weather, scarify surface, re-shape, and compact to required density prior to further construction.

B. Where settling is measurable or observable at excavated areas during general project warranty period, remove surface (pavement, lawn or other finish), add fill or backfill material, compact, and replace surface treatment. Restore appearance, quality, and condition of surface or finish to match adjacent work, and eliminate evidence of restoration to greatest extent possible.

3.19 DISPOSAL OF EXCESS AND WASTE MATERIALS

A. Remove excess excavated material and trash, debris, and other waste materials and legally dispose of them off-site.

PART 4 SCHEDULES

4.01 COMPACTION SCHEDULE

Material Type	Usage	Lift Thickness (1)	Compaction (2)
Granular Fill	Below concrete slabs.	6"	95%
	Other designated areas.	8"	95%
Structural Fill	Under foundations.	8"	95%
	Below concrete slabs.	8"	95%
	Below pipe and conduit.	8"	90%
General Site Fill	Unpaved areas 10 ft. or less outside structure line.	8"	95%
	Unpaved areas more than 10 ft. outside structure line.	12"	90%
Granular Bedding	Below pipe and conduit.	6"	90%
Granular Backfill	Initial backfill around pipe and conduit.	6"	90%

(1) Place manually compacted materials in maximum 4 in. layers.

(2) Percent of maximum density determined in accordance with ASTM D1557 (Modified Proctor test).

END OF SECTION

SECTION 31 23 33

TRENCHING AND BACKFILLING

PART 1 GENERAL

1.01 SUMMARY

A. Provide trenching and backfilling for piping, conduits, and other buried utilities as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 RELATED SECTIONS

31 05 10 Site Preparation.
31 24 00 Embankments

1.03 CLASSIFICATION

A. Excavation of materials encountered under this work will be unclassified without regard to type, difficulty to remove, or suitability for use in construction.

1.04 SUBMITTALS

A. See Section 31 24 00 Embankments, for pipe bedding material.

1.05 TESTING

A. See Section 31 24 00 Embankments, for pipe bedding material Testing.

1.06 PROTECTION

A. Protect existing improvements, utilities, trees and shrubs, and reference marks in accordance with Section 31 05 10.

1.07 BLASTING

A. Use of explosives is not permitted.

PART 2 PRODUCTS

A. The following Products apply to trenching and backfill where pipe placement will not be backfilled with material specified in Section 31 24 00 Embankments.

2.01 SOIL MATERIALS, GENERAL

A. Soil for fill and backfill shall be free of organic matter, debris, frozen soils, ice, and other objectionable materials. Rock particles larger than maximum size specified shall be removed prior to placement of soil.

B. Select existing material excavated from site may be used if it meets requirements specified. If necessary, furnish additional approved material from suitable off-site sources.

1. It is anticipated that select on-site materials will be suitable for bedding and initial backfill. Off-site materials may also be necessary. Contractor shall make such investigations as it deems necessary to determine extent of work.

2.02 WASHED STONE

A. Clean, hard, tough, durable, 1-1/2 in. washed stone, crushed rock, crushed gravel, or gravel free from adherent coatings and soft, flat, or elongated particles.

2.03 GRANULAR BASE, BEDDING, AND BACKFILL

A. Select soils complying with ASTM D2487 soil classification groups GW (well-graded gravel), GP (poorly-graded gravel), SW (well-graded sand), or SP (poorly-graded sand). Aggregate shall pass a 3/4-in. sieve and not more than 35% shall be retained on a No. 10 sieve. Maximum 5% by weight shall pass a No. 200 sieve.

2.04 BACKFILL

A. Previously excavated soils, free of aggregate larger than 3 in., and organic material, subject to approval by A/E.

PART 3 EXECUTION

3.01 PREPARATION

A. Provide site preparation, including clearing, topsoil removal, and pavement removal, as required in accordance with Section 31 05 10.

3.02 TRENCHING

A. Excavate trenches so that pipe can be laid safely and accurately to required line and grade. Hand excavate for bells, fittings and projections to allow for proper jointing and to ensure that pipe rests evenly along barrel and is not resting on bell.

B. Excavation shall extend to bottom of pipe bedding. Unnecessary excavation below this required level shall be filled with compacted sand, gravel, crushed stone or concrete, as approved by A/E.

C. In sand and gravel soils, bottom of trench may be shaped to fit bottom 1/3 of pipe. In silt or clay soils, bottom of trench shall be 4 in. below pipe barrel and 3 in. below bell. In rock, bottom of trench shall be 6 in. below pipe barrel. Under foundations and footings, bottom of trench shall be 8 in. below pipe. Provide Granular Bedding as specified below.

D. Trench widths in ordinary soil shall be limited at top of pipe to not less than a 6 in. clearance on either side of barrel to allow for installation of bedding material between pipe and trench wall. Maximum trench width at top of pipe shall be outside pipe diameter plus 24 in. (30 in. minimum). Trench above top of pipe may be sloped, stepped or vertical to comply with state and federal regulations regarding trenches.

E. Minimum trench width in rock shall not be less than that for ordinary soil. Maximum trench width shall be outside pipe diameter plus 18 in. for an unsheathed trench, and outside pipe diameter plus 24 in. for sheathed trench.

F. During trenching and backfilling work in areas with moisture sensitive soils, do not mix wet or ground water laden soils with dryer soils. Keep wet and dry soils separate to minimize the amount of soil drying required.

3.03 STABILITY OF TRENCHES

A. Slope sides of trenches to angle of repose of material excavated; otherwise, provide sheathing and bracing where sloping is not possible either because of space restrictions or stability of material excavated. Comply with applicable codes and ordinances.

B. Maintain sides and slopes of trenches in a safe condition until completion of backfilling. Take precautions to prevent slides or cave-ins when trenches are made in locations adjacent to backfilled trenches, and when sides of trenches are subjected to vibrations from traffic, machinery, or any other source.

3.04 SHEATHING AND BRACING

A. Provide tight sheathing where necessary to protect nearby structures and pavements, or when trench size must be confined. Notify A/E of unforeseen condition that requires use of sheathing. Sheathing shall be driven unless soil conditions allow sets to be placed after excavating. If placed after excavating, voids between trench wall and sheathing shall be immediately filled with sand.

B. Removal of sheathing shall not take place until trench is backfilled. Compact backfill by flooding or jetting after sheathing is removed. Obtain written approval to leave some or all of sheathed sets to remain in place; cut off and remove upper portion within 2 ft of surface.

3.05 WET TRENCH CONDITIONS

A. Contractor shall attempt to dispose of ground water or surface drainage entering trench by employing ordinary dewatering techniques such as use of sump pumps, sump pits adjacent to pipe alignment, dikes, and similar methods. Dispose of or divert water along existing drainage ways. Do not place water so that it ponds on roadway subgrade or adjacent private property.

B. Allowing water to flow into pipe being laid will not be permitted, except for storm sewer after joints have set. Install temporary stopper or plug on upper end of pipe if there is danger of sand or debris being washed into pipe.

C. Provide filter material, trash screens, and other devices around pumps and intakes to avoid pumping or discharging sediment from construction site.

3.06 UNSTABLE TRENCH BOTTOM

A. When trench bottom is unstable because of ground water, A/E may require extra excavation to remove unstable material. Provide Washed or Crushed Stone foundation followed by Granular Bedding as required in "Bedding and Initial Backfill" article.

3.07 POOR SUBSOIL MATERIALS

A. Notify A/E whenever muck, sawdust, bark, or other material is encountered which would not form a suitable and permanent base; A/E may order it removed and replaced with Granular Base up to bottom of normal trench section.

3.08 BEDDING AND INITIAL BACKFILL

A. Trenches dug in sandy or gravelly materials may use undisturbed earth for bedding provided surface is shaped to conform to pipe. Provide Granular Bedding in all other trenches from subgrade to a point supporting bottom 1/3 of pipe for rigid pipe and to springline (mid-height) for flexible pipe; see Details. Place and compact bedding so that it fills and supports pipe haunch area.

B. Immediately after installation of pipe, provide tamped Granular Backfill or Type I and Type II filter material up to a minimum depth of 1 ft above pipe. Take special care in placing and tamping initial backfill material so alignment and grade of pipe is not disturbed nor pipe damaged.

3.09 TRENCH BACKFILL

A. Backfill more than 1 ft over pipe within the embankment shall be Type I or Type II filter material as shown or previously excavated soil from the embankment and meeting the requirements of the "Backfill" article of Part 2 of this Section. All other trench backfill over initial backfill zone may be previously excavated soil subject to approval by A/E.

B. Backfilling above initial backfill zone in areas where settlement is not critical may be done from top of trench by mechanical means. In no case shall backfill material be dropped from such a height or in such a volume that its impact will cause dislocation or damage to piping.

C. When backfilling in freezing temperatures, cover pipe and tamp backfill around pipe using only loose thawed material. Do not place frozen material in trench within 2 ft of top of pipe, nor around manholes and other structures.

3.10 COMPACTION

A. See Section 31 24 00 and Section 33 41 66 for compaction requirements.

3.11 DISPOSAL OF EXCESS AND WASTE MATERIALS

A. Dispose of excess excavated materials and trash, debris, and other waste materials at legal disposal site.

END OF SECTION

SECTION 31 24 00

EMBANKMENTS

PART 1 GENERAL

1.01 SUMMARY

A. Provide earth embankments as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 RELATED SECTIONS

01 57 60 Construction Dewatering.
31 05 10 Site Preparation.
31 20 00 Earth Moving.
31 23 33 Trenching and Backfilling.
33 41 66 Drain Fill.

1.03 SUBMITTALS

A. Submit the following information for embankment fill materials two weeks prior to start of construction:

1. Source of proposed materials.
2. Grain size analysis in accordance with ASTM C136 and, if more than 15% passes the #200 sieve, use ASTM C117 to demonstrate acceptability of source.
3. Optimum moisture-maximum density curve for fill materials in accordance with ASTM D1557.
4. Direct shear tests performed at 95% Modified Proctor density, near optimum moisture in accordance with ASTM D3080.

B. A/E will review submittals for conformance with the specified requirements and the design parameters. A/E will also confirm material compatibility with drain fill.

C. During construction, submit test reports for field tests.

D. Make submittals in accordance with Section 01 33 00.

1.04 TESTING

A. General: Contractor shall arrange and pay for laboratory and field testing of fill material by a qualified testing agency, acceptable to Owner and independent of Contractor.

B. Laboratory Testing:

1. Perform at least two sets of laboratory tests for each 1,000 cu yd of compacted material used from each source.
2. Fill materials shall be tested for gradation in accordance with ASTM C136 and ASTM C117 for conformance with ASTM D2487 gradation limits, and for liquid limit and plasticity index in accordance with ASTM D4318. Optimum moisture-maximum density curve for fill material shall be determined in accordance with ASTM D1557.
3. Results of laboratory tests shall be received prior to any material placement.

C. Field Testing:

1. Perform a minimum of one density test for each vertical foot of fill for every 100 lin ft of embankment fill.
2. Field density tests shall be in accordance with ASTM D6938 or ASTM D1556.

3. Where soil materials do not conform to type or density specified, soil shall be replaced or reworked to conform. Cost of extra tests for replaced or reworked areas shall be paid for by Contractor.

1.05 PROTECTION

A. Protect existing improvements, utilities, trees and shrubs, and reference marks in accordance with Section 31 05 10.

PART 2 PRODUCTS

2.01 SOIL MATERIALS, GENERAL

A. Soil materials shall be free of organic matter, debris, frozen soils, ice, and other objectionable materials. Rock particles larger than maximum size specified for each type of material shall be removed prior to placement of soil. If not otherwise specified, rock particles shall be no larger than 1/2 the specified lift (layer) thickness.

B. Obtain fill materials from required excavations and grading within site limits, and from designated borrow areas.

2.02 EMBANKMENT FILL

A. Select, natural, free draining soils complying with ASTM D2487 soil classification groups CL or SC, or combinations thereof, and suitable for compaction. Maximum aggregate size shall be 1.5 in. No greater than 20% of particles shall be retained on the No. 4 sieve and no greater than 50% of particles shall be retained on the No. 200 sieve.

2.03 COBBLE AND ROCK SHELL

A. This material is existing on site and consists of well graded riprap from sand size material to 24-inch diameter riprap. If importing of additional Cobble Rock Shell material is needed, it needs to match a similar gradation to existing material.

PART 3 EXECUTION

3.01 FOUNDATION PREPARATION

A. Foundations for embankment fill shall be stripped in accordance with Section 31 05 10 to remove vegetation and topsoil. If shown, or required to remove unsuitable materials, provide excavation work.

B. Except as otherwise specified, grade earth foundation surfaces to remove surface irregularities and scarify parallel to axis of fill or otherwise acceptably score and loosen to a minimum depth of 2 in. Control moisture content of loosened material as specified for embankment fill, and compact and bond surface materials with first layer of fill as specified for subsequent layers of fill.

C. Foundation and abutment surfaces shall be not steeper than 1 horizontal to 1 vertical, unless otherwise specified. Fill test pits and other cavities with material conforming to specifications for earth fill.

D. Keep earth abutment surfaces free of loose, uncompacted earth in excess of 2 in. in depth normal to slope and at a moisture content that embankment fill can be compacted against them to affect a good bond between fill and abutments. Clear rock foundation of loose materials by hand or other effective means. Keep foundations and abutments free of standing water when fill is placed.

E. Occasional rock outcrops in foundations for embankment fill, except in dams and other structures designed to restrain movement of water, shall not require special treatment if they do not interfere with compaction of foundation and initial layers of fill or bond between foundation and fill.

3.02 PLACEMENT

A. Do not place fill until required excavation and foundation preparation have been inspected and approved. Do not place fill upon frozen surface; no snow, ice, or frozen material shall be incorporated in fill.

B. Place fill in approximately horizontal layers; do not exceed maximum loose layer thickness specified. Spread piles and windrows uniformly. Spreading and compacting equipment shall travel approximately parallel to centerline of embankment.

C. Place fill to be hand compacted or compacted by manually directed power tampers in layers not to exceed maximum thickness specified for manually compacted fill.

D. Adjacent to structures, place fill to prevent damage and allow structures to assume loads gradually and uniformly at approximately the same rate on all sides of structure. Do not travel heavy equipment over cast-in-place concrete work until cured a minimum of 14 days, unless otherwise approved.

E. Earth fill in dams, levees, and embankments designed to retain water shall meet the following requirements:

1. Distribute materials throughout each zone uniformly, free from lenses, pockets, or layers differing substantially in texture or graduation.
2. Scarify layers too hard and smooth for proper bond with succeeding layer; scarify parallel to axis of fill to a depth of 2 in.
3. Maintain top surfaces of fills approximately level during construction, except provide a crown or cross-slope of not less than 2 percent for drainage. If the work requires fill to be placed higher at parts of an embankment, maintain top surface of each part level as specified above.
4. Place fill in continuous layers from abutment to abutment, except where openings to facilitate construction or to allow stream flow are authorized. Route equipment travel approximately parallel to embankment centerline.
5. Construct embankments required to be built at different levels so slopes of bonding surfaces between adjacent levels of embankment are not steeper than 3 horizontal to 1 vertical. Strip bonding surface of loose material and scarify, moisten, and recompact at specified moisture content and density to insure good bond with new fill.

3.03 CONTROL OF MOISTURE CONTENT

A. During placement and compaction of fill, maintain moisture content of materials being placed within the specified range.

B. Apply water to fill materials by sprinkling at excavation site or during placement of fill if necessary. Obtain uniform moisture distribution by discing, blading, or other approved methods prior to compaction of layer. If material is too wet when deposited on fill remove or dry it to specified moisture content prior to compaction.

C. If top surface of preceding layer of compacted fill or a foundation or abutment surface in zone of contact with fill becomes too dry to permit suitable bond, scarify and moisten it by sprinkling to an acceptable moisture content prior to placement of next layer of fill.

3.04 COMPACTION

A. Compact each layer of fill to a mass density not less than the percent of maximum density specified in 4.01 Embankment Compaction Schedule.

B. Manually compact fill adjacent to structures to density of surrounding fill by means of manually directed power tampers or plate vibrators. Do not operate heavy equipment within 2 ft of any structure; do not operate vibrating rollers within 5 ft of any structure. Compaction by means of drop weights will not be permitted.

C. Do not pass compacting equipment over cast-in-place concrete until cured 14 days (7 days for precast concrete with a concrete cradle) or over conduits until backfill has been placed above structure to a height equal to one-half the clear span width of structure or pipe or 2 ft, whichever is greater.

D. Compacting of fill adjacent to structures may be started when concrete has attained design strength, determined by test cylinders (ASTM C31). If concrete strength is not determined by tests, passage of heavy equipment and compaction of fill adjacent to structures may not be started until the following curing periods have elapsed:

Structure	Minimum Curing Period
Retaining walls, counterforts, and wing walls	14 days
Cast-in-place conduits and risers (forms removed)	14 days
Cast-in-place concrete conduits and risers (with inside forms in place)	7 days
Walls backfilled on both sides simultaneously	7 days
Footings	4 days
Precast conduits, bedded	1 day

3.05 REMOVAL AND REPLACEMENT OF DEFECTIVE FILL

A. Fill placed at densities lower than the specified minimum density, or at moisture contents outside the specified acceptable range or otherwise not conforming to requirements shall be removed and replaced by acceptable fill. Replacement fill and foundation preparation shall conform to all requirements.

3.06 DISPOSAL OF EXCESS AND WASTE MATERIALS

A. Remove excess excavated material and trash, debris, and other waste materials and legally dispose of them off-site.

PART 4 SCHEDULES

4.01 EMBANKMENT COMPACTION SCHEDULE

Zone	Material Type	Layer Thickness (Max) Machine Compaction	Layer Thickness (Max) Hand Compaction	Moisture Content	Compaction Density *
Embankment	Fill	8"-10"	N/A	+/- 3% of optimum	95%

*ASTM D1557 (Modified Proctor density)

END OF SECTION

SECTION 31 37 00

RIPRAP

PART 1 GENERAL

1.01 SUMMARY

A. Provide loose rock riprap, including geotextile filter fabric, as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 SUBMITTALS

A. Product Data:

1. Borrow site is shown on the drawings. Provide access to sources to enable A/E to inspect and obtain samples once the riprap is produced. Do not deliver riprap until reviewed by A/E.
2. Submit fabric product data. Include material samples, certification of physical properties, and installation procedures.

B. Make submittals in accordance with Section 01 33 00.

1.03 TESTING

A. A/E may perform tests to verify that riprap and completed work meet specified requirements. However, these tests are not intended to provide Contractor with information it may need to assure that materials and workmanship meet requirements of specifications, and their performance will not relieve Contractor of responsibility of performing its own tests for that purpose.

PART 2 PRODUCTS

2.01 RIPRAP

A. Riprap shall consist of hard, dense, durable stone, angular in shape and resistant to weathering. Rounded stone or boulders shall not be used as riprap material. The stone shall have a specific gravity of at least 2.5. Each piece shall have its greatest dimension not greater than three times its least dimension.

B. Riprap gradation shall comply with requirements of Colorado DOT Std. Spec., Section 506, Table 506-2.

2.02 RIPRAP BEDDING

A. Material used for riprap bedding shall meet the requirements for Drain Gravel in Section 33 41 66 – Drain Fill.

PART 3 EXECUTION

3.01 SUBGRADE PREPARATION

A. Grade subgrade surfaces to lines and grades as shown with an allowance for riprap. Remove organic materials. Compact soft subgrade soils. When fill to achieve subgrade lines is required, provide granular materials.

3.02 PLACING DRAIN GRAVEL RIPRAP BEDDING

A. Place riprap bedding at the locations, thicknesses, lines, and grades shown on the Drawings.

B. General: Surfaces to receive bedding materials shall be smooth and firm, free from deleterious materials, and shall be brought to the lines and grades shown on the Drawings. Prepare the surfaces that are to receive bedding materials, by rolling and trimming as necessary to enable a uniform lift of bedding of the specified thickness to be placed thereon. Surface preparation will include, but not be limited to, bringing all low spots up to the lines and grades shown on the Drawings with compacted fill and removing all material projecting above lines and grades shown on the Drawings.

C. Placement: Place the bedding materials in a manner that minimizes segregation and results in uniform lifts of bedding materials of the thicknesses shown on the Drawings. Place riprap bedding materials from the bottom of the slope working up the slope.

D. Moisture condition the materials as necessary to control dust and to minimize segregation.

E. Compaction is not required for the bedding materials; however, bedding materials shall be spread in such a manner as to form a smooth, uniform layer under the riprap.

3.03 EQUIPMENT-PLACED ROCK RIPRAP

A. Place riprap at the locations, thicknesses, lines, and grades shown on the Drawings.

B. Stones with typical stone dimensions that are equal to D50 and larger shall be placed at the top surface with faces and shapes matched to minimize voids and form as smooth a surface as practical. Dumping and backhoe placement alone is not sufficient to ensure a properly interlocked system. The material may be machine-placed and then arranged as necessary by use of an excavator with a multi-prong grappling device or by hand to interlock and form a substantial bond.

C. Hand placement will be required where necessary to correct obvious irregularities and to prevent damage to adjacent improvements and wherever equipment placement methods are unsatisfactory.

3.04 HAND-PLACED RIPRAP

A. Riprap shall be securely bedded with larger rocks firmly in contact one to another. Spaces between larger rocks shall be filled with smaller rocks and spalls. Smaller rocks shall not be grouped as a substitute for larger rock. Flat slab rock shall be laid on edge.

END OF SECTION

SECTION 32 92 36

NATIVE SPECIES SEEDING

PART 1 GENERAL

1.01 SUMMARY

A. Provide native species seeding as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

B. Types of work required include, but are not limited to, the following:

1. Preparing, topsoiling, seeding, and mulching areas designated on Drawings to receive native species seeding.
2. Maintaining native species seeding for a 1-year period.

1.02 RELATED SECTIONS

31 05 10 Site Preparation: For topsoil removal.

1.03 SUBMITTALS

A. Product Data: Submit product labels for seed mixtures proposed for use on project.

B. Make submittals in accordance with Section 01 33 00.

1.04 WORK SEASONS

A. Conduct seeding during favorable weather conditions. Seeding will only be performed between September 1 and when ground freezes, and when the ground thaws and June 1, unless approved by Owner's Representative ""ground thaw" shall be defined as the earliest date in a new calendar year in which seed can be buried 1/2" into the surface soil (topsoil) through normal drill seeding methods. "ground freeze" shall be defined as that time during the fall months in which the topsoil, due to freeze conditions, prevents burying the seed 1/2" into the topsoil through normal drill seeding operations. Comply with seed supplier's recommendations for planting conditions.

PART 2 PRODUCTS

2.01 TOPSOIL

A. Loam, sandy loam, silty clay loam, or clay loam humus-bearing surface soil; 100% passing the 2 in. sieve; neither excessively acid, nor excessively alkaline; reasonably free of subsoil, clay lumps, brush, and weeds; and free of extraneous matter harmful to plant growth.

B. Reuse topsoil salvaged from within work area. If necessary, obtain topsoil to supplement insufficient quantities at site from naturally well-drained local sources; do not obtain from bogs or marshes.

2.02 NATIVE SEED

A. General: Deliver in bags tagged and labeled to show percentage of purity and germination. Seed shall have been tested within 1-year prior to date of seeding and shall conform to latest State and Federal seed laws.

B. Upland Areas: Upland seed mixture shall be High Altitude Native Grass Mix by Beauty Beyond Belief Wildflower Seed (Boulder, Colorado, 303.530.1222, www.bbbseed.com), or approved equal.

C. Wetland Areas: Wetland seed mix shall be Montane Wet Meadow Mix by Western Native Seed (Coaldale, CO, 719-942-3935, westernnativeseed.com), or approved equal.

2.03 MULCH

A. Field or marsh hay or straw of oats, barley, rye, or triticale.

B. Mulch shall be inspected for and Regionally Certified as weed free based on the Regionally Designated Noxious Weed and Undesirable Plant List for Colorado, Wyoming, Montana, Nebraska, Utah, Idaho, Kansas, and South Dakota

C. Mulch materials shall not contain excessive moisture which prevents uniform feeding through mulching machine.

2.04 FERTILIZER

A. Fertilizer as recommended by seed provider.

PART 3 EXECUTION

3.01 FINISH GRADING

A. Disturbed area shall be graded to be reasonably smooth; fill all washes and gullies to conform to required lines and grades.

3.02 TOPSOIL PLACEMENT

A. After completion of finish grading, place minimum of 4 in. of topsoil over areas indicated to be seeded.

B. Smooth grade topsoil to eliminate irregularities. Remove rocks and hard soil clods. Finished topsoil grade shall be 1 in. below adjoining grade of any surfaced area.

3.03 FERTILIZING

A. Apply fertilizer to prepared topsoil according to seed provider's recommendations.

3.04 SEEDING

A. Subgrade shall be final graded to the design elevation leaving the surface in a roughened condition. Shallow potholes and low ridges can remain following grading to promote variety of soil moisture regime conditions. In no case will a smooth, compacted seed bed common to a typical construction site be allowed to remain as the final grade. Re-work previously prepared areas that have become compacted or damaged by rains or traffic.

B. Apply native seed mixture using a drop spreader or broadcast spreader using method and application rate recommended by seed provider.

C. Do not sow immediately following rain, when ground is too dry, when ground is frozen or untillable, or during windy periods.

D. Do not seed areas in excess of that which can be mulched on same day.

3.05 MULCHING

A. Apply straw mulch uniformly in all seeded areas at rate of 1-1/2 tons per acre to a loose depth of 1 to 2 in. Anchor mulch using non-asphalt-based tackifier or mulch nets installed and stapled according to manufacturer's recommendations.

B. Mulch shall be applied within 4 hours after seeding. Areas not mulched within 4 hours after seeding or prior to precipitation or damaging winds on site shall be reseeded with the specified seed mix and re-mulched at the Contractor’s expense.

C. Avoid applying mulch in windy conditions.

3.06 HYDROMULCHING

A. At Contractor's option, mulch may be applied by hydromulch method at a minimum rate of 1700 lb per acre.

B. Mix components in water using equipment specifically designed for hydromulch application. Continue mixing until uniformly blended into homogeneous slurry suitable for hydraulic application. Include nonasphaltic tackifying agent in mixture.

3.07 POST PLANTING MAINTENANCE

A. Maintain seeded areas for 1-year following planting.

B. Follow seed provider recommendations for maintenance and care during the 1-year period following planting.

3.08 ESTABLISHMENT AND REPLACEMENT

A. Seeded areas which fail to become established during the 1-year maintenance period shall be reseeded and maintained to ensure healthy growth. Because native seed mixtures are difficult to assess the first year of growth, satisfactory establishment of the cover crop and general erosion control by vegetation shall constitute establishment.

END OF SECTION

SECTION 33 01 36

CURED-IN-PLACE PIPE LINING

PART 1 GENERAL

1.01 SUMMARY

A. Provide cured-in-place lining as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

B. Existing 30-inch steel outlet pipe to be lined is deteriorating, but is generally believed to be intact with no obstructions. Pipe has not been televised by Owner.

C. Cured-in-place lining shall be installed over the entire length of the 30-inch steel outlet pipe, both existing and newly installed, from the intake structure to the outfall structure.

D. The pipe shall be televised before and after installation of the cured-in-place lining.

1.02 RELATED SECTIONS

33 42 15 Piping and Accessories.

1.03 SUBMITTALS

A. Product Data: Submit product data for lining materials.

B. Installer Qualifications: Submit documentation of installer qualifications as specified in "Quality Assurance" article below.

C. Work Procedures: Submit proposed procedures for review at least 10 days prior to beginning lining operations.

D. Inspection Report: Submit report of inspection performed following lining. Include pressure test results and video recording of inspection. Video shall be recorded on a standard definition DVD suitable for use in a computer DVD drive.

E. Make submittals in accordance with Section 01 33 00.

1.04 QUALITY ASSURANCE

A. Liner installer shall have a minimum of 20,000 linear feet of pipe lining experience with the specified product.

1.05 PATENTS

A. Contractor shall warrant to Owner that the methods, materials and equipment used herein, where covered by patents, are furnished in accordance with applicable licenses and that the prices included on Bid Form include applicable royalties and fees in accordance with such license. Warranty shall include defense against claims from infringement of patent and shall save harmless Owner and his Representatives from loss on account thereof.

PART 2 PRODUCTS

2.01 LINING MATERIAL

A. General: Pipe lining system must be recommended by the manufacturer for the type of application shown on the Drawings. A/E approval of pipe lining system required.

B. Size: Liner shall be properly sized to diameter and length as shown.

C. Structural Capacity: Pipe lining system must be capable of supporting no less than 58 ft of overburden material with an estimated unit weight of 125 lb per cu ft (pcf). The pipe lining shall also be capable of supporting 200 lb per sq (psf) of surcharge load located above the overburden.

PART 3 EXECUTION

3.01 INSTALLATION OF LINER

A. Install liner according to manufacturer’s recommendations.

B. Finished lining shall be continuous over entire length of an insertion run between the low-level intake structure (upstream) and the outfall structure (downstream) and shall be as free as commercially practicable from visual defects such as foreign inclusions, dry spots, pinholes, and delamination. Lining shall be impervious and free of any leakage from pipe to surrounding ground or from ground to inside of lined pipe.

C. Defects which will affect integrity or strength of lining in the foreseeable future or warranty period shall be repaired at Contractor's expense in a manner mutually agreed by Owner and Contractor.

3.02 INSPECTION

A. Upon completion, televise pipe and prepare a written report of findings.

END OF SECTION

SECTION 33 41 66

DRAIN FILL

PART 1 GENERAL

1.01 SUMMARY

A. Provide drain fill for drainage blankets and as riprap bedding as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 RELATED SECTIONS

01 57 60 - Construction Dewatering.
31 20 00 - Earth Moving.
31 24 00 - Embankments.

1.03 SUBMITTALS

A. Drain Fill Source: At least two weeks prior to start of construction, submit source location for drain fill materials.

B. Test Report: Submit grain size analysis of proposed drain fill to demonstrate acceptability of source.

C. Make submittals in accordance with Section 01 33 00.

1.04 TESTING

A. A/E will perform tests to verify drain fill in place. These tests are not intended to provide Contractor with information needed to assure proper execution of work and test performance will not relieve Contractor of responsibility of performing tests for that purpose.

B. At least one grain size analysis will be performed for each type of drain fill for each 100 lin ft along the embankment where drain fill is placed. Contractor shall provide A/E free access to placed material for purpose of obtaining samples for testing. Samples will be obtained from in-place, compacted drain fill.

1.05 DELIVERY, STORAGE, AND HANDLING

A. Drain fill aggregates shall be stored and handled by methods that prevent segregation of particle sizes or contamination by other materials.

PART 2 PRODUCTS

2.01 DRAIN FILL MATERIALS

A. Drain fill aggregates shall be sand, gravel, crushed stone or mixtures thereof, composed of clean, hard, durable mineral particles free from organic matter, clay balls, soft particles, excessive fine-grain soils, or other substances that would interfere with their free-draining properties.

1. Not more than 15 percent, by weight, shall be flat, elongated particles.
2. Not more than 5 percent of material finer than a No. 4 sieve shall be crushed limestone.
3. Material passing the No. 200 shall be non-plastic.

B. Drain fill shall comply with the following gradation by weight:

Sieve Designation	Percent Passing Type 1[1]	Type 2[2]	Drain Gravel[3]
1"			
3/4"		100	
1/2"			100
3/8"	100	90-100	85-100
No. 4	95-100	20-55	10-30
No. 8	80-100	5-30	0-10
No. 16	50-85	0-10	0-5
No. 30	25-60		
No. 50	5-30	0-5	
No. 100	0-10		
No. 200	0-5		

[1] ASTM C33 Fine Aggregate
[2] ASTM D448 No. 89
[3] ASTM No. 8

PART 3 EXECUTION

3.01 PREPARATION

A. Foundation surfaces and trenches shall be clean and free of organic matter, loose soil, foreign substances, and standing water when drain fill is placed. Earth surfaces upon or against which drain fill will be placed shall not be scarified.

B. Contaminated drain fill near edge of each excavation section shall be removed prior to continuation of placement of drain fill.

C. Do not prepare any more base than can be satisfactorily covered that same working day.

3.02 PLACEMENT

A. Drain fill shall not be placed until subgrade has been inspected and approved by A/E. Drain fill shall not be placed over or around pipe or drain tile until installation of pipe or tile has been inspected and approved.

B. Drain fill shall be placed uniformly in layers not more than 12 in. deep before compaction. When compaction is accomplished by manually controlled equipment, layers shall be not more than 8 in. deep. Material shall be placed in a manner to avoid segregation of particle sizes and to ensure continuity and integrity of all zones. No foreign materials shall be allowed to become intermixed with or otherwise contaminate drain fill.

C. Traffic shall not be allowed to cross over drains at random. Equipment crossovers shall be maintained, and number and location of such crossovers shall be established and approved prior to beginning of drain fill placement. Each crossover shall be cleaned of all contaminating materials and shall be inspected and approved by A/E before additional drain fill is placed.

D. Damage to foundation surface or to sides or bottoms of trenches occurring during placement of drain fill shall be repaired before drain fill placement is continued.

E. Upper surface of drain fill constructed concurrently with adjacent zones of earth fill shall be maintained at an elevation at least 1 ft above upper surface of adjacent fill.

F. Drain fill over or around pipe or drain tile shall be placed in a manner to avoid displacement of pipe or tile in line or grade.

3.03 CONTROL OF MOISTURE

A. Moisture content of drain fill materials shall be controlled. When addition of water is required, it shall be applied in such a way as to avoid excessive wetting of adjacent earth fill.

3.04 COMPACTION

A. Type 1: Each lift of Type 1 drain material shall be compacted by a minimum of 3 passes over the entire surface with the track of a crawler-type tractor or tire of front-loader equipment weighting at least 30 tons.

B. Type 2 and Drain Gravel: No compaction of Type 2 drain material and Drain Gravel will be required beyond that resulting from placing and spreading operations.

END OF SECTION

SECTION 33 42 15

PIPING AND ACCESSORIES

PART 1 GENERAL

1.01 SUMMARY

A. Provide piping and accessories as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 RELATED SECTIONS

31 20 00 Earth Moving.
31 23 33 Trenching and Backfilling.
33 01 36 Cured-in-Place Pipe Lining.
33 41 66 Drain Fill.
40 70 10 Instrumentation.

1.03 SUBMITTALS

A. Product Data: Submit product data for pipe, fittings, and other piping accessories.

B. Test Reports: Submit report for leakage testing.

C. Record Drawings: Accurately record locations of pipe and field changes on a set of Drawings. Prior to final application for payment, deliver record drawings to A/E.

D. Make submittals in accordance with Section 01 33 00.

1.04 DELIVERY, STORAGE, AND HANDLING

A. Carefully unload and store pipe to prevent chipping, cracking, or damage to surface coatings. Pipe shall not be skidded upon ground. Repair damaged coatings.

PART 2 PRODUCTS

2.01 HDPE PIPE

A. ANSI/AWWA C906, DR 32.5, high density, perforated or non perforated pipes. Joints shall consist of an integral bell and spigot and shall be water tight according to requirements of ASTM D3212 (10.8 psi pressure test).

B. Pipe perforations shall meet the following specifications

C. Capacity of each perforation: 0.08 gpm

D. Hole Size: 5/32” or 3.9688 mm

E. Hole Spacing: 8 holes equally spaced around pipe circumference, every 2” of pipe

2.02 STEEL PIPE

A. Steel pipe conforming to AWWA C200 and having the following minimum wall thickness:

Diameter	Minimum Wall Thickness
30"	0.375"

B. Joints shall be in accordance with AWWA C200. Joints must be welded. Pipe ends for field-welded joints shall be beveled in accordance with AWWA C206.

C. Steel pipe shall have an interior lining of primer and coal-tar enamel (AWWA C203), cement mortar (AWWA C205), or near-white blast cleaned (SSPC SP-10) and painted with one 7-mil coat of coal-tar epoxy, unless otherwise noted.

D. Exterior of all pipe shall receive two coats of coal tar epoxy, 7 mils dry thickness per coat, applied to commercial blast cleaned surface (SSPC SP-6).

2.03 PVC PLASTIC PIPE

A. Poly (vinyl chloride) (PVC) plastic pipe, Sch. 40 or 80, type 1, grade 1, meeting requirements of ASTM D1785, unless otherwise designated. Joints shall be solvent cement type or threaded type (Sch. 80 only) using teflon tape.

B. Provide Sch. 40 or 80, ASTM D2466 or D2467, socket-type fittings, in size and classification the same as connecting pipe, suitable for solvent cement joints, unless otherwise noted.

C. Solvent cement shall be a compatible solution of type 1, grade 1, unplasticized PVC plastic compound in accordance with ASTM D2564, free-flowing and free of lumps, undissolved particles or foreign matter that will adversely affect ultimate joint strength or chemical resistance.

2.04 GALVANIZED STEEL PIPE

A. Steel pipe, welded or seamless, conforming to ASTM A53; Sch. 40, unless otherwise designated.

B. Provide welded or flanged joints for piping 2-1/2 in. diameter and larger and threaded joints for piping 2 in. diameter and smaller, unless otherwise designated. Steel welding fittings shall be ANSI B16.9, std. wt.; steel flanges shall be ANSI B16.5, Class 150; and malleable iron threaded fittings shall be ANSI B16.3, Class 150; unless otherwise designated.

C. Where designated, plastic coating and wrap shall comply with requirements of American Gas Association.

2.05 DUCTILE IRON PIPE

A. Ductile iron pipe shall conform to AWWA C151, Class 52 unless otherwise shown. Pipe sections shall be straight and true circular sections with inner and outer surfaces concentric. Pipe sections shall be at least 16 ft long.

B. Fittings shall conform to AWWA C110 or C153, 250 psi working pressure.

C. Pipe and fittings shall have standard thickness cement mortar lining and interior bituminous coating conforming to AWWA C104. Apply bituminous seal coat on exterior of pipe and fittings conforming to AWWA C151. Coatings shall be smooth, tough and tenacious and impervious to water without tendency to scale off, and shall not be brittle.

D. Pipe joints shall be either push-on type (slip joint) or mechanical joint conforming to AWWA C111.

2.06 RODENT GUARD

A. Rodent guard to be fabricated from galvanized metal with bolt on band and expanded metal screen.

2.07 DUCTILE IRON SEWER ADAPTER

A. Ductile iron body with rubber gasket.

2.08 PIPE STRAP

A. Heavy-duty, 2-hole type, galvanized.

2.09 EXPANSION ANCHORS

A. Description: Wedge type, torque-controlled expansion anchors with washers.

B. Acceptable Manufacturers: Anchors shall be Hilti, ITW Ramset/Red Head, Powers Fasteners, Simpson Strong-Tie, Wej-it, or approved equal.

C. Materials: Provide stainless steel anchors of diameter indicated on the Drawings.

1. Stainless Steel Anchors: ASTM F593 stainless steel bolts and ASTM F594 stainless steel nuts, Alloy Group 1 or Alloy Group 2.

2.10 FLEXIBLE CONDUIT SEAL

A. Description: Provide a flexible, foam-based conduit seal to prevent water from flowing into bottom of the 8-inch PVC pipe used to house the hydraulic lines. Seal must be removable without causing damage to the PVC carrier pipe or surrounding concrete.

B. Required sealing head: 50 ft.

C. Required operating temperature: 20 to 100 ºF.

D. Acceptable manufacturers: Duraline Hydra-Seal S-60 or approved equal.

PART 3 EXECUTION

3.01 LINE AND GRADE

A. Provide staking as required to install pipe and drainage structures to line and grade as shown on Drawings.

3.02 LAYING OF PIPE

A. Where practicable, begin at lowest point of proposed pipe line; lay with bell end or receiving groove edge upstream in direction of laying.

B. Cut in and connect to existing pipe and drainage structures as required. When connection to an existing pipe is required, contact A/E if adjustments to inverts are needed.

C. Pipe shall be laid immediately following the trench preparation and bedding provisions of Section 31 20 00 / 31 23 33.

D. Exercise care when handling pipe. Ropes, slings, or other devices must be used for lowering pipe into trench. Only pipe which is suitable for use is to remain on site. Damaged or broken pipe shall be immediately separated from acceptable pipe.

E. Lay pipe uniformly to line and grade on a prepared bed providing even support along entire barrel. Excavate bell holes in bedding material so pipe will rest on barrel and not on bell. As work progresses, interior shall be cleared of dirt and debris. Do not lay pipe where water is above bedding material except where A/E determines that foundation is stable, pipe will not be displaced upward, and joint construction will not be affected by water.

F. Each pipe shall be bedded by hand or by equally careful means to 12-in. cover before laying subsequent pipes. Fill space between pipe and trench wall in 6-in. layers and manually compact. Pipe sizes larger than 15-in. diameter may require mechanical compaction of bedding material.

G. When work is not in progress, water may be allowed to flow into newly laid pipe if provisions are made to prevent dirt from washing into pipe.

3.03 JOINTING

A. Joint materials and methods shall conform to manufacturer's recommendations.

B. Pipe joints shall be clean before joints are made. No joints shall be made under water.

3.04 ALIGNMENT AND GRADE

A. Contractor shall view the HDPE drain pipe after installation and backfill using a televised inspection. If defects are found due to failure of installation or materials, Contractor shall promptly correct defects at no additional cost to Owner.

3.05 FIELD CUTTING PERFORATIONS

A. Field drill perforations in HDPE pipe if manufactured product does not meet specifications.

3.06 SECURING PIPE TO CONCRETE

A. Secure pipe to concrete as shown on the Drawings using straps and expansion anchors. Required strap spacing and anchor embedment depths are indicated on the Drawings. Install expansion anchors according to manufacturer’s instructions. Expansion anchors shall not be installed into concrete that is less than 7 days old.

3.07 INSTALLING FLEXIBLE CONDUIT SEAL

A. Install at location shown on Drawings according to manufacturer’s instructions.

END OF SECTION

SECTION 35 22 28

STAINLESS STEEL SLIDE GATES

PART 1 GENERAL

1.01 SUMMARY

A. Provide stainless steel slide gates as shown and as specified. Comply with applicable provisions of Divisions 00 and 01.

1.02 RELATED SECTIONS

03 30 00 – Cast-in-Place Concrete.
33 42 15 – Piping and Accessories.

1.03 SUBMITTALS

A. Shop Drawings: Submit shop drawings for slide gate and operator. Include hydraulic requirements for operator.

B. Certification: Submit manufacturer's certification that slide gate and operator comply with the specified requirements.

C. O/M Manuals: Submit operation and maintenance manuals for slide gate and operator.

D. Make submittals in accordance with Section 01 33 00.

1.04 FIELD MEASUREMENTS

A. Field verify dimensions prior to gate fabrication.

PART 2 PRODUCTS

2.01 ACCEPTABLE MANUFACTURERS

A. Subject to compliance with the specified requirements, provide products by one of the following manufacturers, or approved equal:

1. Golden Harvest.
2. Hydro-Gate.
3. Rodney Hunt.
4. WACO Products.
5. Whipps.

2.02 DESIGN REQUIREMENTS

A. Design gates to comply with AWWA C561 and the following conditions.

B. Gates shall have self-adjusting seals. Gates that utilize adjustable wedges are not acceptable.

C. To compensate for debris strikes, gate bottom edge shall be designed to resist a static load of 1,000 lb per lin ft that is applied 3 in. above gate bottom lip.

D. Gates shall be operable at variable gate open positions in increments of 0.10 ft to regulate flow. Gate shall be able to remain in any closed, partially open, or fully open position for long periods of time without excessive vibrations.

E. Gates shall not vibrate excessively during opening, closing, or static positioning.

2.03 SLIDE GATES AND ACCESSORIES

A. Provide fabricated stainless steel slide gates complying with AWWA C561 and 4.01 Stainless Steel Slide Gate Schedule.

B. Gates designated for flush-bottom closure shall have a seal mounted either on gate slide or on gate frame. Full length of contacting member shall be accurately machined to make an effective seal when gate is closed.

C. Provide stainless steel hydraulic lines per gate manufacturers recommendation.

2.04 MANUAL OPERATORS

A. A hand-operated hydraulic pump shall be provided as back up operator.

2.05 PORTABLE HYDRAULIC OPERATOR

A. One underwater (fully submerged) hydraulic operating system shall be provided to operate the gate. The system shall be designed to operate the gate smoothly and uniformly and hold the gate in the desired position. The gate shall be operated manually with a provided portable, gasoline-powered hydraulic pump mounted to a hand cart.

B. Connections to hydraulic lines will be accessible via a locked valve box located at the top of the dam embankment.

C. The hydraulic power unit (HPU) shall be designed to operate using environmentally-friendly (confirm with Owner prior to constructing HPU) hydraulic fluid such as Mobil 224H.

D. Gate assembly shall have indicator device at operator location (top of dam) indicating position of gate in 0.10 ft increments.

2.06 GROUT AND FRAME LEAKAGE SEAL

A. Grout furnished for installation of embedded components shall be as recommended by gate manufacturer for the site conditions.

B. Frame to concrete connection shall be free of leakage. Acceptable leakage seals include a layer of grout between frame and existing concrete, rubberized membranes, or injectable waterstops. Design and furnishing of leakage seal materials shall be responsibility of gate manufacturer.

2.07 FINISH

A. Ferrous metal surfaces shall receive surface preparation and high-solids epoxy protective coating or equivalent suitable for outdoor exposure before shipment to site.

B. Machined surfaces, tapped holes, and threads shall receive protective coat of grease.

PART 3 EXECUTION

3.01 GATE INSTALLATION

A. Install gates in accordance with shop drawings and manufacturer's recommendations and in a manner that will prevent leakage around seats and binding of gates during operation.

B. Surfaces of metal against which concrete will be placed shall be free from oil, grease, loose mill scale, loose paint, surface rust, and other debris or objectionable coatings.

C. Anchor bolts, thimbles, and frames shall be secured in true position and in forms and held in alignment during placement of concrete. Mechanical anchors shall not be installed into concrete that is less than 7 days old. Adhesive anchors shall not be installed into concrete that is less than 21 days old.

D. Concrete surfaces against which seals will bear or against which flat frames or plates are to be installed shall be smooth and uniform.

E. Install frame and anchorages in a manner that prevents leakage between frame and concrete buttress and secures gate for all load conditions.

F. When a gate is attached to a wall thimble, mastic or resilient gasket shall be applied between gate frame and thimble in accordance with recommendation of gate manufacturer.

3.02 LIFT INSTALLATION

A. Gate stems, stem guides, and gate operators shall be carefully aligned so that stem is parallel to guide bars or angles on gate frame after installation.

3.03 OPERATIONAL TESTS

A. Clean, lubricate, and otherwise service gates and operators in accordance with manufacturer's instructions. Operate each gate several times throughout its full range. Test functioning of each operator. Adjust as necessary to insure satisfactory operation of gate system.

B. Should any gate vibrate excessively during opening, closing, or static positioning (see "Design Requirements" article above), manufacturer shall promptly attend a meeting onsite with A/E to discuss cause and solutions for vibrations.

C. A certified field technician from the gate manufacturer shall inspect the final installation.

3.04 LEAKAGE TEST

A. Perform a leakage test on all slide gates. Maximum permissible leakage at normal pool shall be 0.10 gallons per minute per foot of seating perimeter in compliance with AWWA C561, 5.2.2. Gate shall not leak more than 2 gallons per minute along any one foot length of seal perimeter. Adjust gates to meet permissible leakage.

PART 4 SCHEDULES

4.01 STAINLESS STEEL SLIDE GATE SCHEDULE

Quantity	Size (in.)* w x h	Stem*	Type*	Frame Type	Max. Unbalanced Head (ft) Seating	Unseating	Wall Thimble	Operation
1	36x36	HC	FB	Flange Frame	55	10	No	M and P

*Abbreviations:

Size: w - clear opening width, h - clear opening height

Stem: HC - hydraulic cylinder

Type: FB - flush bottom

Operation: M - manual P - portable hydraulic operator

END OF SECTION

SECTION 40 70 10

INSTRUMENTATION

PART 1 GENERAL

1.01 SUMMARY

A. Provide a complete system of instrumentation as shown and as specified, except as designated elsewhere. Responsibility under this Section consists of system design, installation, start-up and testing, and instruction of Owner's authorized personnel. Coordinate with related work of other sections; see below. Comply with applicable provisions of Divisions 00 and 01.

B. Equipment shall be of latest and most modern design; components shall be current models at time of bidding. Equipment shall have overall performance, precision, and accuracy as guaranteed by manufacturer; units shall be stable, vibration free, properly operating in accordance with manufacturer's recommendations. Contractor shall be responsible for all details as required, including any undesignated items necessary for a complete installation, properly installed and ready for operation. Contractor shall be fully responsible for additional costs resulting from unauthorized deviations from these specifications.

C. System shall be the product of one (1) single approved Vendor who has an experience record in real-time early warning systems satisfactory to the Owner and A/E based on established systems. Vendor shall provide instrumentation and electronics as shown on the Drawings and real-time satellite data transmittal services to allow for the Owner and other stakeholders to keep informed of meteorological and hydrological conditions at the dam. Vendor shall provide a web-based interface from where the real-time data can be viewed. Vendor shall also provide capabilities for automated data monitoring and the issuing of alarms when specified data thresholds are met or exceeded. See the early warning system feasibility study provided in the Appendices.

1.02 RELATED SECTIONS

01 33 00 Submittal Procedures.
01 57 60 Construction Dewatering.
01 60 00 Product Requirements.
33 42 15 Piping and Accessories.

1.03 SHOP DRAWINGS

A. Submit shop drawings for instrumentation in accordance with Section 01 33 00 and instructions of individual Sections. Furnish the following information:

1. Manufacturer's product data and specifications.
2. Shop drawings showing general dimensions, openings, connections, components, wiring schematics, templates, required appurtenances, and construction details.
3. Weight of major components.
4. Parts suppliers, service crews, and repair facilities.
5. Recommended installation procedures.

B. Submit systems complete in one submittal; individual segments of systems will not be reviewed.

C. Review of shop drawings by A/E will not relieve Contractor of responsibility for providing materials, workmanship, and design which meet specifications and operating conditions.

D. See the Drawings for the two (2) general locations and layout for the proposed instrumentation stations. Contractor shall submit shop drawings for the concrete foundations needed to accept the selected vendor's instrumentation pedestals.

1.04 OPERATION AND MAINTENANCE (O/M) MANUALS

A. Submit O/M manuals in accordance with Section 01 33 00 and as specified in individual sections; include the following:

1. A/E-reviewed shop drawings and product data.
2. Installation and operating instructions.
3. Maintenance instructions and address of authorized service center.
4. Wiring diagrams and parts lists.
5. Test data and certifications.
6. Manufacturer's warranty information.

1.05 MAINTENANCE CONTRACT

A. Where designated, supplier/manufacturer shall offer to the Owner for consideration a 5-year renewable maintenance-service contract at an additional cost. Submit maintenance-service contracts for review with shop drawings.

1.06 DELIVERY, STORAGE, AND HANDLING

A. Deliver instrumentation to job site at appropriate time for installation. Equipment items shall be packaged in cartons, crated, or affixed to pallets with protective wrappings. Exercise care to prevent damage from handling.

B. Protect stored components in accordance with manufacturer's instructions. Store mechanical and electrical components in a warm, dry location; store fabrications off ground, under cover, and away from damp or corrosive surfaces. Keep equipment dry at all times. Equipment suppliers shall periodically inspect to insure proper storage of equipment.

1.07 WARRANTY

A. System manufacturer shall warranty for 1 year that all equipment is free from defects in design, materials and workmanship. Furnish and install replacement parts for defective components at no additional cost.

B. Unless otherwise designated, warranty provisions shall begin with date of Substantial Completion or with Owner's acceptance for beneficial use, whichever is earlier, but not prior to receipt of manufacturer's report of startup services (where designated). Beneficial use shall commence when product has been fully installed, tested and adjusted in accordance with Contract Documents (and in the case of operating equipment, has been certified by manufacturer to be ready for operation), is ready to be used for intended application, and is accepted by Owner for use.

PART 2 PRODUCTS

2.01 ACCEPTABLE MANUFACTURERS

A. Vendor shall select system components from established manufacturers of instruments with proven track records of long-term, successful reliability in conditions like those at Hogchute Dam. All products shall be manufactured in accordance with the specifications by one of the named manufacturers or approved equal; substitutions shall be submitted in accordance with Section 01 60 00 and Instructions to Bidders.

B. The system shown on plan sheets is designed by OneRain Incorporated. Contractors may use a OneRain system or approved equal.

2.02 PROTECTION

A. All infrastructure shall be powder coated to prevent corrosion and to improve aesthetics.

B. Instruments and electronics shall be protected from weather and tampering.

1. Each site shall use traffic-style infrastructure capable of withstanding high winds manufactured by Pelco or an approved equal.
2. Batteries shall be housed within locking, watertight, stainless steel, National Electrical Manufacturer Association (NEMA) rated enclosures.
3. Electronic components shall be housed within locking, watertight, plastic, NEMA rated enclosures. Plastic NEMA rated enclosures shall be housed in locking 1/4”-thick steel enclosures to prevent damage from firearms.
4. Antennas, sensors, and solar panels shall be mounted in such a way as to prevent tampering and damage from firearms.

2.03 EQUIPMENT IDENTIFICATION

A. Each item of equipment shall have a nameplate, securely attached, with manufacturer's designations and characteristics.

2.04 BUBBLER

A. Provide bubbler-type water level sensor to measure depth of water in reservoir.

B. Operating ambient temperature: -40 to +158 ºF (-40 to 60 ºC).

C. Operating altitude: 9,900 ft.

D. Operating humidity: 0 to 95 percent.

E. Required depth measurement: 0 to 60 ft.

F. Bubbler gas delivery: Constant-flow bubbler system with built-in compressor. User-programmable bubbling rates of 30 to 120 bubbles per minute.

G. Purge pressure: 20 to 90 PSI, user selectable.

H. Power supply: 10.8 to 16.5 V DC.

I. Output interface: SDI-12 compatible with data logger.

J. Accuracy: 0.01 ft or 0.1% of measured depth, whichever is greater.

K. Desiccators: The bubbling mechanism and the non-submersible transducer must be equipped with a desiccating system.

L. Enclosure: NEMA4.

M. Manufacturers: FTS or approved equal.

2.05 PRESSURE TRANSDUCER

A. Provide temperature-compensated pressure transducer-type water level sensors to measure depth of water in reservoir (as a backup to the bubbler-type sensor) and in the stream below the dam (as a primary sensor).

B. Operating temperature: 23 to +113 ºF (-5 to +45 ºC).

C. Operating altitude: 9,900 ft.

D. Operating humidity: 0 to 95 percent.

E. Required depth measurement: 0 to 60 ft.

F. Power supply: 12 to 24 V DC.

G. Output interface: SDI-12 compatible with data logger.

H. Accuracy: 0.05% of measured depth.

I. Materials: Corrosion-resistant metal.

J. Manufacturers: OTT or approved equal.

2.06 TIPPING BUCKET

A. Provide tipping bucket-type rain gauge with 0.01-inch measurement resolution at a rate of 0 to 25 inches per hour.

B. Required operating temperatures 32 to 140 ºF.

C. Diameter: 8 inches.

D. Switch rating: 200 V DC max.

E. Output: SDI-12 compatible with data logger.

F. Materials: UV-resistant plastic or corrosion-resistant metal, shock and vibration resistant.

G. Manufacturer: Design Analysis Associates, WaterLog Series, Model H-340 or approved equal.

2.07 DATA LOGGER (PRIMARY STATION)

A. Provide a primary data logger to record measured data from the tipping bucket, pressure transducer, and bubbler installed at the top of dam. The primary data logger shall also receive data from the secondary station via a 900 MHz spread-spectrum radio feed. The primary station shall broadcast recorded data from the site using a satellite transmitter.

B. Manufacturer: Campbell Scientific CR1000X Measurement and Control Data Logger or approved equal.

2.08 DATA LOGGER (SECONDARY STATION)

A. Provide a secondary data logger to record measured data from the pressure transducer installed at the secondary station downstream of the dam. The secondary data logger shall transmit recorded data to the primary station on top of the dam via a 900 MHz spread-spectrum radio feed.

B. Manufacturer: Campbell Scientific CR300-RF407 Series Measurement and Control Data Logger or approved equal.

2.09 APPURTENANCES

A. Battery: Campbell Scientific PS200 12 V Power Supply with Charging Regulator or approved equal. Provide two (2) batteries at each station for power redundancy.

B. Solar Panel: Campbell Scientific SP10 Solar Panel or approved equal.

C. 900 MHz Antenna for Primary Station: Campbell Scientific 900 MHz 6dBd Omni Antenna w/Type N Female and Mounting Hardware or approved equal.

D. 900 MHz Antenna for Secondary Station: Campbell Scientific 900 MHz 6dBd Yagi Antenna with Type N Female and Mountain Hardware or approved equal.

E. Satellite Transmitter: ORBCOMM ST 6100 or approved equal.

F. Conduit: For buried applications, provide PVC Plastic Pipe. For non-buried applications, provide Galvanized Steel Pipe. See Section 33 42 15 – Piping and Accessories.

G. Underground Enclosure Assembly: Precast polymer concrete construction with open bottom. Equipped with stainless steel hex-head cover bolts.

2.10 STATION MARKERS

A. Post: 1.375” deep carbon-steel heavy duty u-channel post, 6-foot tall. Green baked enamel finish. Uline (1-800-295-5510) Model No. H-4585 or approved equal.

B. Plate: aluminum plate, 1/8-inch minimum thickness, sharp edges and corners removed. Plate is coated with Label on side facing the access road on top of dam.

C. Label: non-metalized microprismatic lens reflective coating for production of reflective work zone signs. ASTM D4956 Type III or IV. White “3M High Intensity Prismatic Reflective Sheeting Series 3930” or approved equal.

D. Marker: self-adhesive exterior-rated waterproof and fade-resistant vinyl letters. Letters shall be all black with no background (stand-alone). Coordinate labeling names and numbers with A/E prior to ordering labels. Marker is located on side facing the gravel road.

E. Mounting hardware: each plate requires two sets of stainless steel 3/8-inch standard thread stainless bolt, nut, front washer, and back washer. Owner will consider alternative mounting solutions that are rust-free and stable against winter wind forces on plate.

PART 3 EXECUTION

3.01 INSTALLATION, GENERAL

A. Install equipment and systems in accordance with shop drawings and manufacturer's recommendations.

B. Installation of primary station instrumentation and electronics must take place during Phase 1 dewatering, when the lake is drawn down and the intake structure is dewatered by cofferdam.

3.02 VENDOR SERVICES

A. Supplier of instrumentation and electronics shall provide services of a factory-trained service representative to:

1. Review the Drawings of required appurtenant structures (e.g., foundations, piping, etc.), providing feedback if the design of any of these structures requires modification to ensure compatibility with Vendor’s instrumentation and electronics.
2. Meet with Contractor onsite prior to construction to identify final locations of primary and secondary stations.
3. Install instrumentation and electronics.
4. Check and adjust equipment prior to operation.
5. Check integral equipment supplied by other manufacturers.
6. Observe field tests of equipment.
7. Train Owner's operator(s) in operation of equipment.

B. Notify A/E and Owner when this initial service will be performed.

C. After start-up of equipment, service representative shall furnish a letter to A/E and Owner confirming that the installation is in accordance with manufacturer recommendations, necessary alignments and adjustments have been made, and equipment is operating properly.

D. In addition to initial services, manufacturer shall provide for 1-day inspection trips after 1 month and 6 months of operation to inspect and adjust equipment.

3.03 SYSTEM TESTS

A. Prior to acceptance, conduct an operational test, under observation of A/E, to demonstrate that installed equipment and systems meet purpose and intent of Specifications. Performance shall be demonstrated throughout operating range.

B. Test shall demonstrate that equipment and systems are not defective electrically, mechanically, or otherwise, and are in a safe and satisfactory operating condition.

3.04 CLEANING

A. Clean components and systems for intended use and in accordance with Division 01.

3.05 STATION MARKERS

A. Coordinate marker name and numbering system with Owner.

B. The intent of the crest stationing posts is to visually indicate crest movement by allowing sighting along the tops of each post from end to end of the embankment. Therefore, it is important to align posts carefully so that the tops of each post are in a straight line and level.

C. Assemble the plates with label on one side and marker on same side as label, carefully keeping markers centered and neat. Avoid placing marker near future bolt locations.

D. Drive stationing post and label systems at 100-ft increments of stationing, as shown on plans, noting that posts along crest of dam shall be located at downstream breakline of embankment crest. All posts along the stations shall be in a straight line and have the same top elevation. All labels shall face access road.

END OF SECTION

Appendix E

Slope Stability and Seepage Modeling

Slope Stability Summary

Hogchute (Carson) Dam Improvements (DAM ID 420127)
Mesa County, Colorado

References

1) Design Standards No. 13, Embankment Dams, Chapter 4: Static Stability Analysis, Phase 4 (Final), United States Department of the Interior, Oct 2011
- Appendix D, Table 5-1, Average Engineering Properties of Compacted Soils from the Western United States.
2) *Slope Stability* , Engineer Manual EM 1110-2-1902, Table 3-1, US Army Corps of Engineers, 31 Oct 2003.
3) Improvements to Water Supply System, Hogchute Reservoir, Plan Profile and Sections, RJ Tipton and Associates Inc., May 24, 1947.
4) *Inflow Design Flood for Carson Dam, Mesa County, Colorado, Carson Lake Hydrology Report Addendum* , June 16, 2020.
5) Geotechnical Investigation, Carson Lake, Huddleston-Berry Engineering and Testing, November 4, 2019.
6) Geotechnical Data Report, Hogchute Dam (AKA Carson Lake) DAM ID 420127, RJH Consultants, January 2019.
7) PE Civil Reference Manual, Michael R Lindeburg, 16th ed, Sept. 2019, Table 35.11, page 35-23, - Typical Permeabilities.
8) Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.
9) Geotechnical Design Parameters, Carson Lake, Huddleston-Berry Engineering and Testing, August 27, 2020.

Analysis

Horizontal Datum is NAD83, Vertical Datum is NAVD88. City of Grand Junction performed survey in winter 2017. Nominal Crest Elev. 9902 ft.
Maximum Normal Water Elevation is 9895 ft. Current normal Water Elevation is 9885 ft., spillway crest elevation is 9885 ft.
GeoStudio SEEP/W and SLOPE/W computer program was used.
Critical section span the main embankment from borings B-101 to B-103 at crest and B-104 to B-105 at the downstream toe.
Headwater EL 9895 ft and Tailwater EL. is 9840.5 ft (NAVD 88) (Ref. 6).
Headwater EL 9895 ft (Ref. 6) and average tailwater from piezometer data is 9840.5 ft (NAVD88).
Evaluate for IDF (5,930 cfs) headwater EL 9900.40 ft and tailwater EL. 9848.5 ft (NAVD88). (Ref. 4)
Rapid drawdown from normal pool El. 9895 ft. to pipe intake structure El. 9855 ft.
Embankment Fill dimensions are 3:1 to 2.5:1 H:V upstream 2:1 H:V downstream per Ref. 6. Based on Ref. 3, dimensions of embankment construction vary per section.
Use continuous 500 pcf surcharge for equipment on top of slope.

Boring Logs

Soil data taken from seven borings, B-101 to B-105 in Ref. 6.
Top B-1 EL. 9902.2 feet, Top B-2A/B EL. 9902.1 feet, Top B-3 EL. 9901.8 feet, Top B-4 EL. 9846.1 feet, Top B-105A EL. 9865.7 and Top B-5B EL 9866.9 feet.
From Ref. 6, page 6: "At some locations , the SPT sampler encountered refusal (50 blows for less than 6 inches of penetration) prior to advancing 18 inches; therefore, SPT N-values and the associated samples could not be obtained at these locations."
Therefore correlation of strata identified in Ref. 6 with Ref. 3 drawing was used to determine model layers for the relative engineering properties.

BOREHOLE STRATA (S to N)

B-101 (9902.2')	B-102 (9902.1')	B-103 (9901.8')	B-104 (9846.1')	B-105A (9865.7')
Lat: 38.995296	**Lat: 38.995677**	**Lat: 38.996095**	**Lat: 38.995285**	**Lat: 38.995632**
Long: -108.109759	**Long: -108.109800**	**Long: -108.109881**	**Long: -108.110292**	**Long: -108.110160**
Total Depth: 77.5'	**Total Depth: 48.0'**	**Total Depth: 30.0'**	**Total Depth: 33.0'**	**Total Depth: 73.5'**
1'-8': GC	1'-26': GC	1'-14.5': SC	0'-6': GP-GM Colluvium	0'-9': SC Colluvium
8'-15.5': SC	26'-31': SC	14.5-17': GC	6'-25': CL Colluvium	9'-50': CL Colluvium
15.5'-23': GC	31'-38.5': GC	17'-23': SC	25'-30': SC Colluvium	50'-60.3': SC Colluvium
23'-51': SC	38.5'-40.5: GP-GC	23'-27': CL Colluvium	30'-33': CL Colluvium	60.3'-65': CL Colluvium
51'-77.5': CL Colluvium	40.5'-46': SC	27'-30': SC Colluvium		65'-73.5': SC Colluvium
	46'-48': GP-GC			

AYRES

Blow Count Correlation

Embankment Fill: Ref. 6, pg. 15 "In our opinion (RJH), the SPT results were likely influenced by larger gravel or cobbles within the embankment fill and are not reliable to correlate with material density"

Colluvium: Ref. 6, pg. 17 "In our opinion (RJH), the SPT results were likely influenced by larger gravel, cobbles or boulders within the colluvium and are not reliable to correlate with material density"

Layer Classification

Model layers are presented below based on borings, drawings and laboratory testing. See Ref. 1, 3, 5, and 6 for classification.

Thickness, Soil Classification & Index Laboratory Test Results

Embankment Fill: (GC to SC), Medium Dense to Dense Clayey Sand with Gravel to Clayey Gravel with Sand

Colluvium: (CL to SC), Sandy Lean Clay with Gravel/Sand to Clayey Sand with Gravel

Riprap: (Cobbles to Boulders) Cobble and Fill as indicated in Ref. 3 and 6.

Soil Properties

(Footnotes are the reference identification at top of page.)

1) Embankment Fill (GC to SC)								
	Dry In-Situ Unit Weight, γ (pcf)	**Moist/Saturated In-situ Moisture (%)**	**Moist In-situ Unit Weight, γ (pcf)**	**Saturated Unit Weight, γ (pcf)**	**Maximum Dry Unit Weight, ASTM D698 γ (pcf)**	**Optimum Moisture (%)**		
Slope Stability	105.1[6]/109.6[6]	10.8[6]/14.3[6]	116[6]	125[6]	130[6]	10.4[6]		
	Average Unit Weight, γ (pcf)	**Average moisture content, %**	**Average Moist Unit Weight, γ (pcf)**	**Average Friction Angle, ϕ (degrees)**				
Ref. 1 - SC	115.6	14.2	132.0	33.9				
Ref. 1 - GC	111.1	15.9	128.8	27.5				
Ref. 1 - GP	127.5	6.5	135.8	41.4				
		Average	**132.2**	**34.3**				
	Permeability, Kx (cm/sec)	Compressibility (psf)	Saturated Water Content (%)	Residual Water Content (%)	Anisotropy Ratio (Ky/Kx)~	10% finer than particle size distribution (mm)	60% finer than particle size distribution (mm)	Liquid Limit
Seepage Stability	0.0000027[6] (In Situ)	NA	29[6]	25.4[6]	0.30	0.003[5]	1.895[5]	28[6]

2) Colluvium (CL to SC)								
	Dry In-Situ Unit Weight, γ (pcf)	**Saturated In-situ Moisture (%)**	**Effective Friction Angle, ϕ (degrees)**	**Effective Cohesion, c' (psf)**	**Saturated Unit Weight, γ (pcf)**			
Slope Stability	106.7[6] (ave)	22.4[6] (ave)	36[6]	0.0	130[6]			
	Permeability, Kx (cm/sec)	Compressibility (psf)	Saturated Water Content (%)	Residual Water Content (%)	Anisotropy Ratio (Ky/Kx)	10% finer than particle size distribution (mm)	60% finer than particle size distribution (mm)	Liquid Limit
Seepage Stability	0.0031[6]	6000[6]	25.2[6]	23.3[6]	0.52[6]	<0.001[6]	1.42[6]	27[6]

3) Riprap (Cobbles to Boulders)						
	Min. Unit Weight, γ (pcf)	**Max. Unit Weight, γ (pcf)**	**Min. Friction Angle, ϕ (degrees)**	**Max. Friction Angle, ϕ (degrees)**	**1/3 Moist Unit Weight*, γ (pcf)**	**1/3 Friction Angle*, ϕ (degrees)**
Ref. 1	130.9	144.5	38.0	43.7	135.4	39.9
				Average	**135.4**	**39.9**
	Permeability, Kx	Compressibility (psf)	Saturated Water	Residual Water Content	Anisotropy Ratio	Modeling Sample
Seepage Stability	0.025[7]	NA	16[7]	11.4[1]	1.0	Gravel

*Reference 2 (*Slope Stability* , EM 1110-1-1902, COE, 1 January 2003, Appendix D, Page D-7 (a)) for the selection of effective stress parameters:
"The failure envelope for effective stresses from Consolidated-Undrained tests is, for practical purposes, the same as the failure envelope from consolidated-drained (CD or S) tests."
A conservative consideration is to evaluate as drained conditions with no effective cohesion efforts and provided average values of criteria to avoid outlier influence.

~FERC requirements of gravity dams anisotrophic conditions are determined by Creep Theory: The horizontal permeability (kh) is 3 times as great as the vertical permeability (kv).

Seismic Horizontal Coefficient determined by 2/3 of Peak Ground Acceleration from Ref. 9 (PGA = 0.1593).

Although the soil is not purely sand, reduce the friction angle by 1 to 2 degrees to account for only saturated conditions (*Soil Mechanics In Engineering Practice* , Terzaghi and Peck, John Wiley and Sons, 4th Printing 1949, page 83).

Based on the above data, engineering judgement was used to choose the following soil parameters.

1) Embankment Fill (GC to SC)
γs = 125 pcf
γm = 116 pcf
c = 0 psf Saturated
ϕ = 34° ϕ = 32°

3) Riprap (Rock Fill)
γs = 135 pcf
γd = 125 pcf
c = 0 psf Saturated
ϕ = 39° ϕ = 38°

2) Colluvium (CL to SC)
γs = 130 pcf
γd = 106 pcf
c = 0 psf Saturated
ϕ = 36° ϕ = 34°

Piezometer Data Note: Average tailwater EL 9840.5 ft is similar to B-104 readings.
B-101 Piezometer Readings September 11 to 22nd 2018: 9865.4 ft to 9863.1 ft
B-102 Piezometer Readings September 11 to 22nd 2018: 9866.9 ft to 9863.9 ft
B-103 Piezometer Readings September 11 to 22nd 2018: 9879.3 ft to 9879.2 ft
B-104 Piezometer Readings September 22nd 2018: 9840.5 ft
B-105 Piezometer Readings September 17 to 22nd 2018: 9863.7 ft to 9855.9 ft
Use For Lowered Condition Model: Headwater Pool EL. 9885 & Tailwater EL. 9840.5 ft
Use For Normal Pool Condition Model: EL. 9895 ft & Tailwater EL. 9840.5 ft
Use For IDF Condition Model: EL. 9900.4 ft & Tailwater EL. 9848.5 (Ref. 8: HEC-HMS Computation)

Results

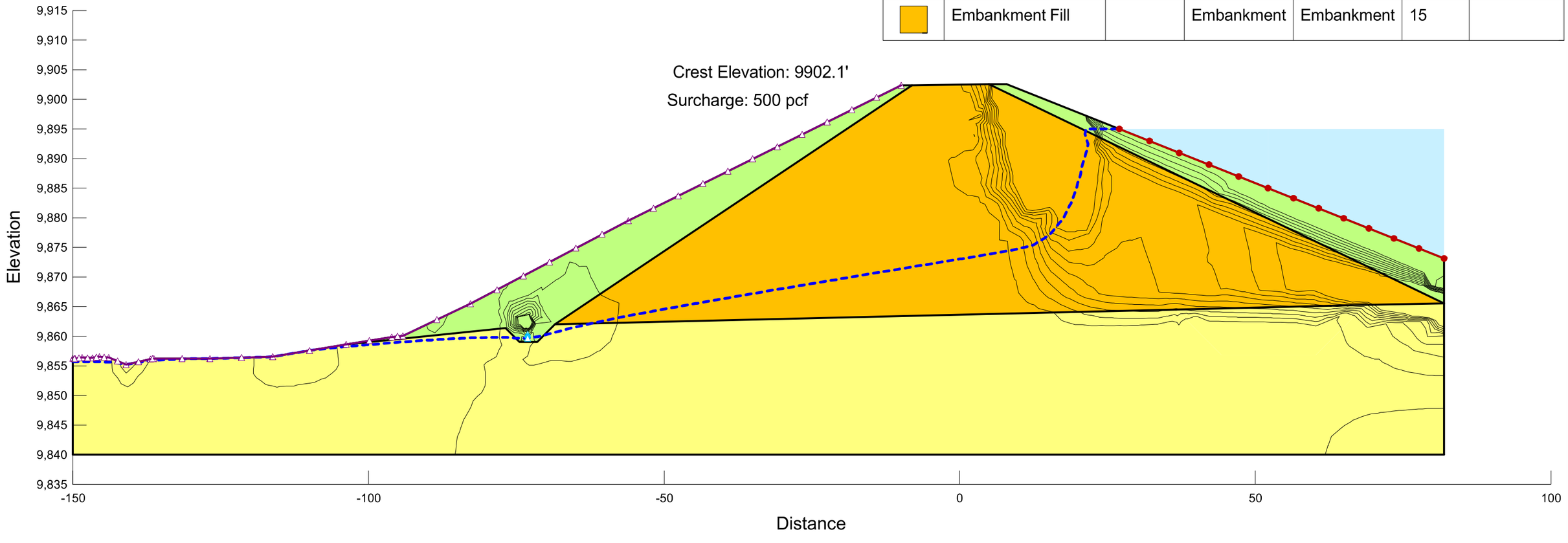
Embankment Stability Results

Load Case	Upstream	Downstream	USCOE Required Safety Factor	Bureau Required Safety Factor
Station 20+60				
Steady-state seepage normal pool	1.9	1.5	1.5	1.5
Maximum surcharge pool	---	1.4	1.4	1.2
Rapid drawdown from maximum storage pool	1.3	---	1.1	1.2
Seismic Steady-state seepage normal pool	1.2	1.1	1.0	1.0
Station 21+50				
Steady-state seepage normal pool	1.6	1.5	1.5	1.5
Maximum surcharge pool	---	1.3	1.4	1.2
Rapid drawdown from maximum storage pool	1.2	---	1.1	1.2
Seismic Steady-state seepage normal pool	1.1	1.2	1.0	1.0
Station 22+50				
Steady-state seepage normal pool	1.7	1.6	1.5	1.5
Maximum surcharge pool	---	1.6	1.4	1.2
Rapid drawdown from maximum storage pool	1.2	---	1.1	1.2
Seismic Steady-state seepage normal pool	1.2	1.3	1.0	1.0

Project: DSB - Carson Dam, Station 20+60
Seepage Stability
2/25/21 Existing Conditions
Seepage Material, Normal Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Nominal Water Level 9895	Hydraulic	Water Total Head	9,895 ft
	Zero Pressure	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Embankment Fill		Embankment	Embankment	15	



Kind: Slope/W
Method: Morgenstern-Price

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Hogchute Dam Embankment

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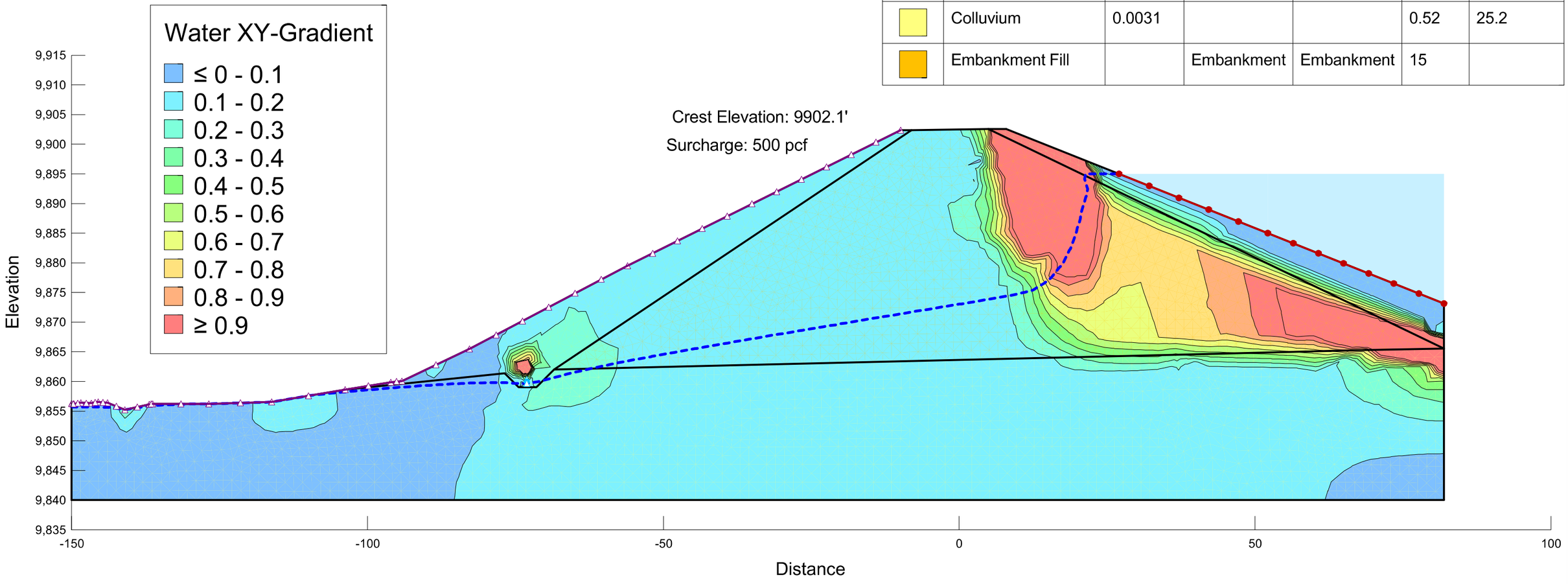
Project: DSB - Carson Dam, Station 20+60
Seepage Stability
2/25/21 Existing Conditions
Seepage Gradient, Normal Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Nominal Water Level 9895	Hydraulic	Water Total Head	9,895 ft
	Zero Pressure	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Embankment Fill		Embankment	Embankment	15	



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Method: Morgenstern-Price

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Hogchute Dam Embankment

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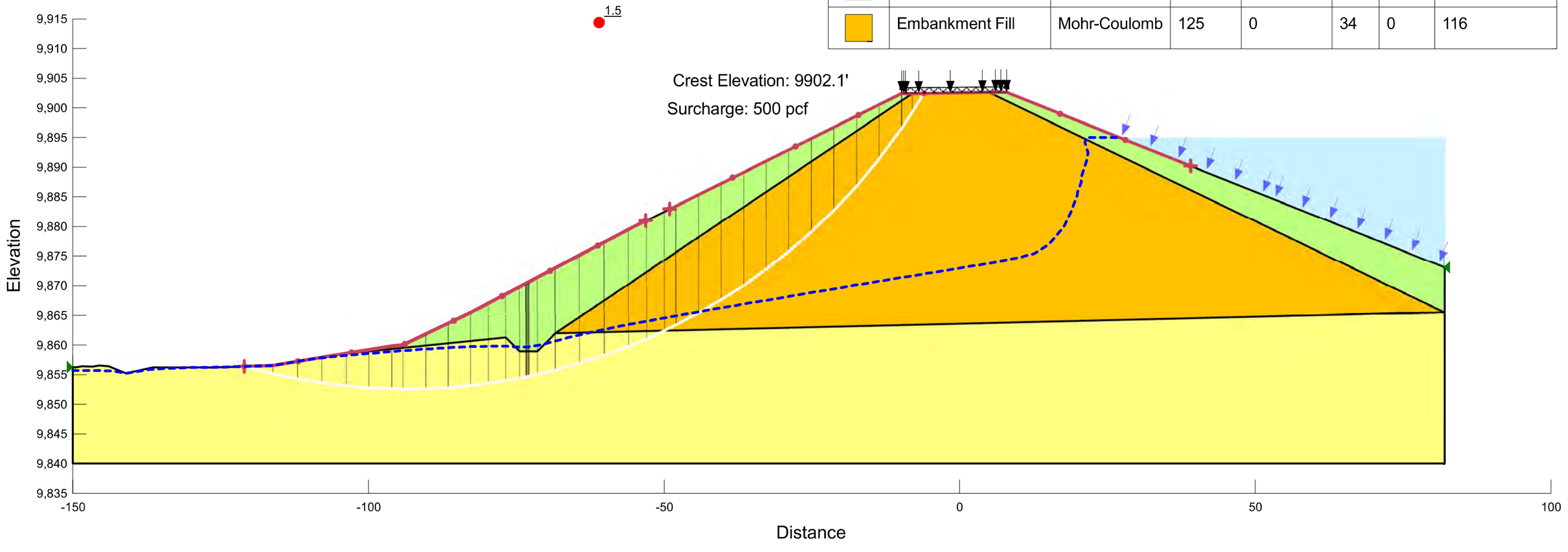
Project: DSB - Carson Dam, Station 20+60
Slope Stability
2/25/21 Existing Conditions
Downstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116



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Method: Morgenstern-Price

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Hogchute Dam Embankment

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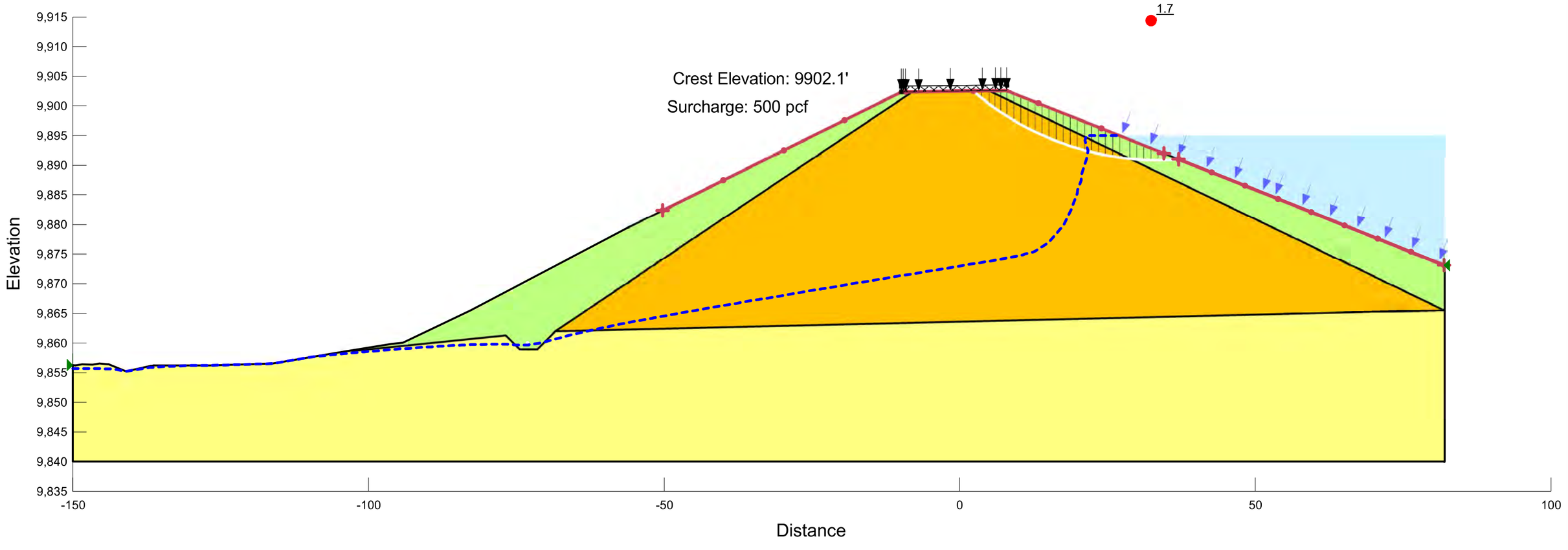
Project: DSB - Carson Dam, Station 20+60
Slope Stability
2/25/21 Existing Conditions
Upstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116



Kind: Slope/W
Method: Morgenstern-Price

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Hogchute Dam Embankment

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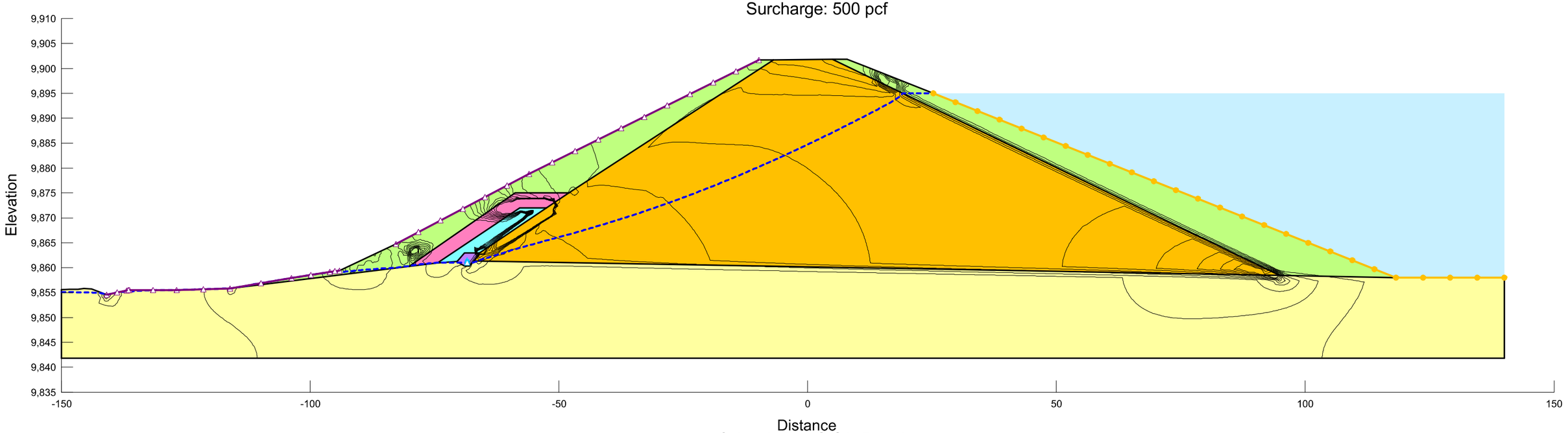
Project: DSB - Carson Dam, Station 20+60
Seepage Stability
2/25/21 Proposed Conditions
Seepage Material, Normal Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	9895.0 Nominal Reservoir Level Pressure	Hydraulic	Water Total Head	9,895 ft
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Zero pressure Head	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Compacted Embankment		Embankment	Embankment	0.3	
	Embankment Fill		Embankment	Embankment	0.3	
	Type 1 Filter Material		Type 1 Filter	Type 1 Filter	0.3	
	Type 2 Filter Material		Type II Material	Type II Material	0.3	

Crest Elevation: 9902.1'

Surcharge: 500 pcf



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Method: Morgenstern-Price

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Hogchute Dam Embankment

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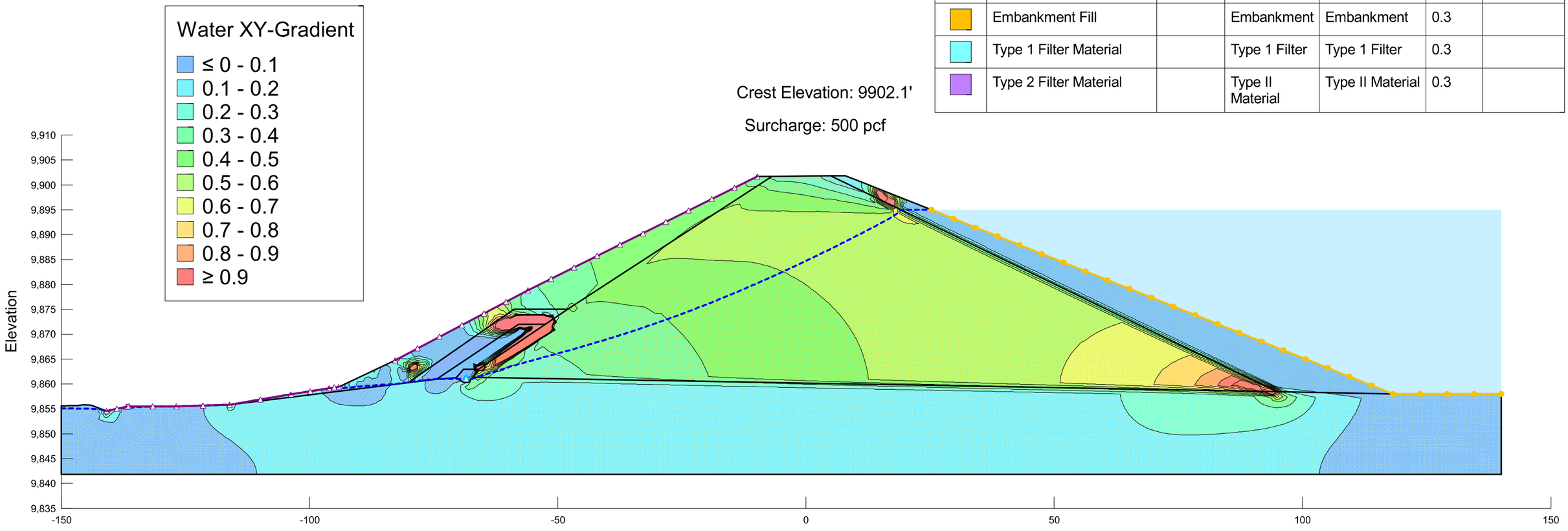
Project: DSB - Carson Dam, Station 20+60
Seepage Stability
2/25/21 Proposed Conditions
Seepage Gradient, Normal Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	9895.0 Nominal Reservoir Level Pressure	Hydraulic	Water Total Head	9,895 ft
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Zero pressure Head	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Compacted Embankment		Embankment	Embankment	0.3	
	Embankment Fill		Embankment	Embankment	0.3	
	Type 1 Filter Material		Type 1 Filter	Type 1 Filter	0.3	
	Type 2 Filter Material		Type II Material	Type II Material	0.3	



Kind: Slope/W
Method: Morgenstern-Price

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Hogchute Dam Embankment

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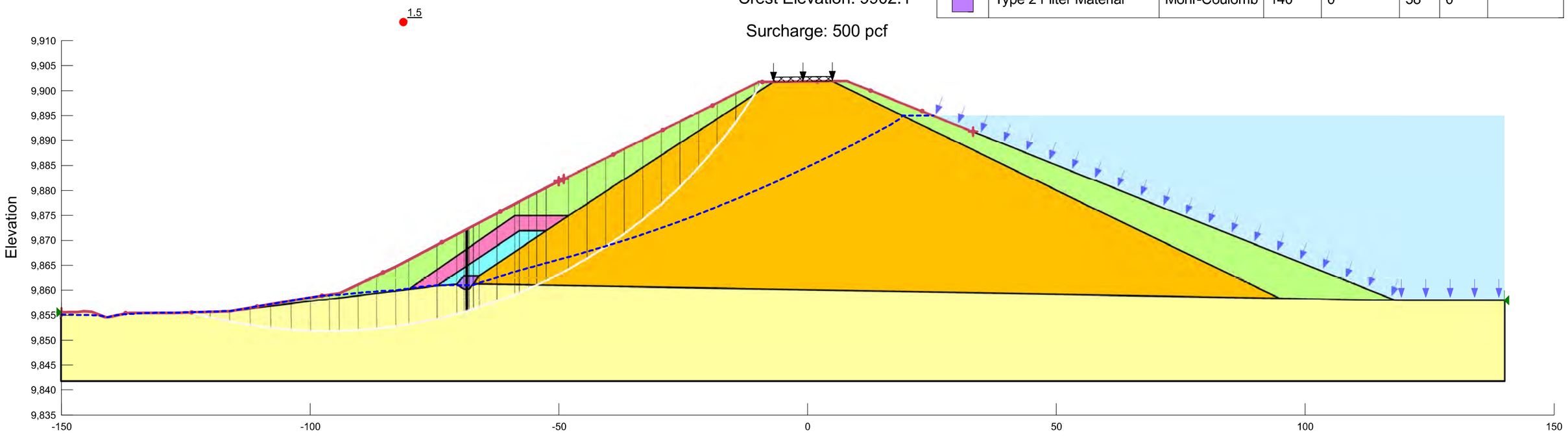
Project: DSB - Carson Dam, Station 20+60
Slope Stability
2/25/21 Proposed Conditions
Downstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'

Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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Hogchute Dam Embankment

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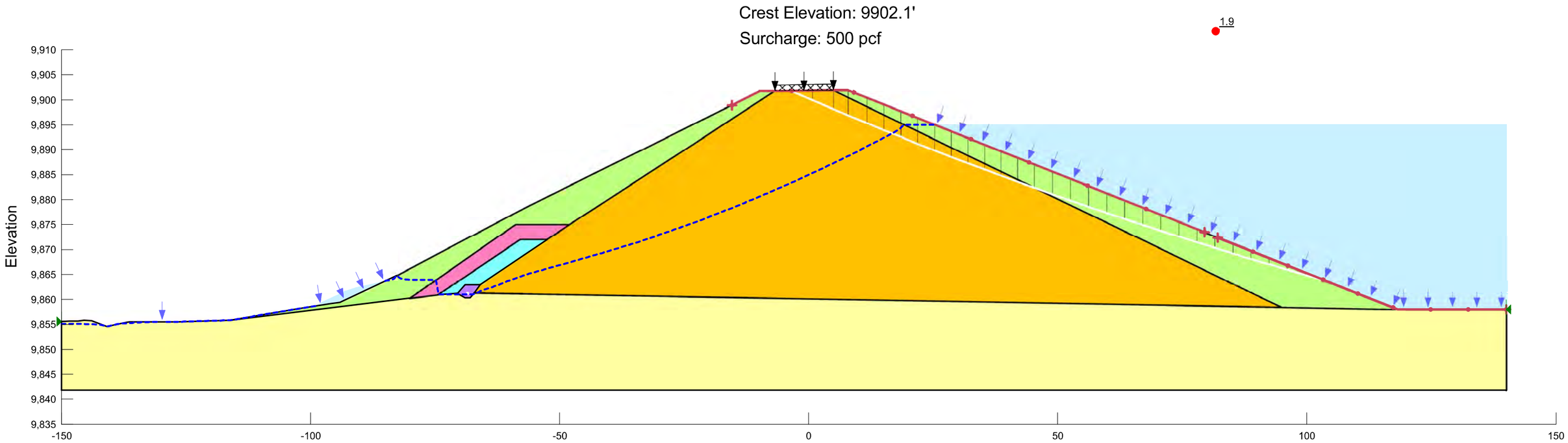
Project: DSB - Carson Dam, Station 20+60
Slope Stability
2/25/21 Proposed Conditions
Upstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	



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Method: Morgenstern-Price

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Hogchute Dam Embankment

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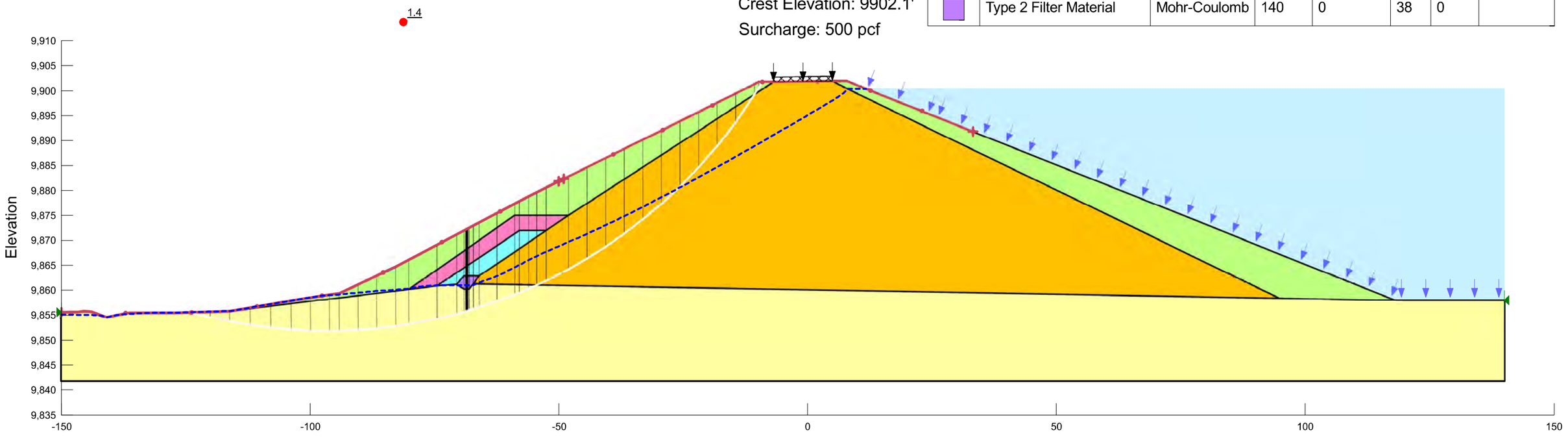
Project: DSB - Carson Dam, Station 20+60
Slope Stability
2/25/21 Proposed Conditions
Downstream Slope, IDF Head Water Pool EL. 9900.4'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'
Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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Hogchute Dam Embankment

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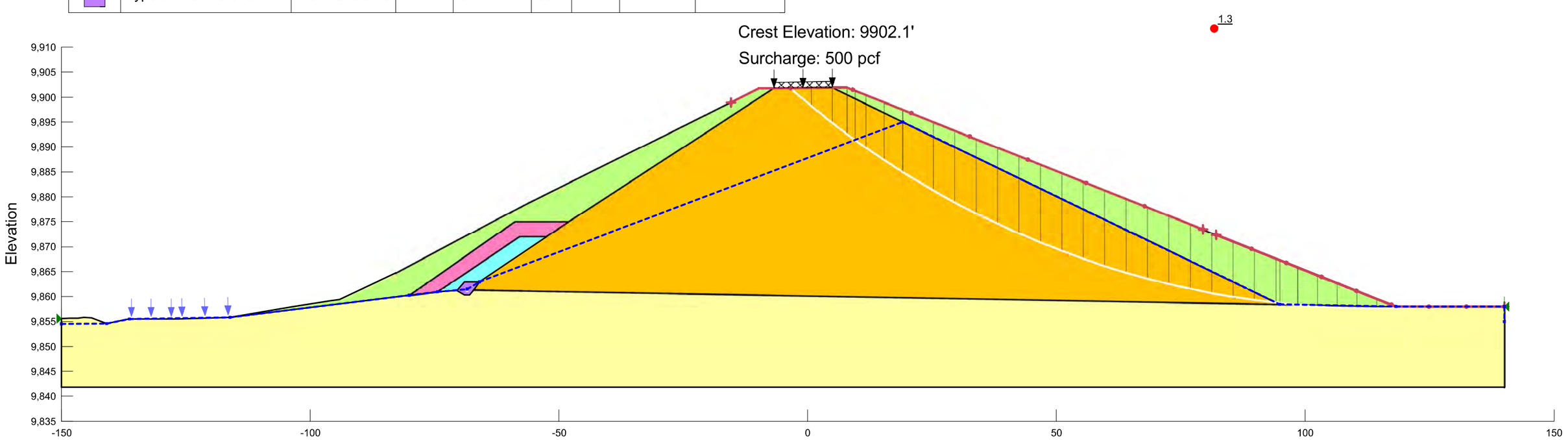
Project: DSB - Carson Dam, Station 20+60
Slope Stability
2/25/21 Proposed Conditions
Upstream Slope, Rapid Drawdown Head Water Pool EL. 9855'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)	Piezometric Line
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125	1
	Colluvium	Mohr-Coulomb	130	0	34	0	106	1
	Compacted Embankment	Mohr-Coulomb	130	0	34	0		1
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116	1
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125	1
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0		1



Kind: Slope/W
Method: Morgenstern-Price

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Hogchute Dam Embankment

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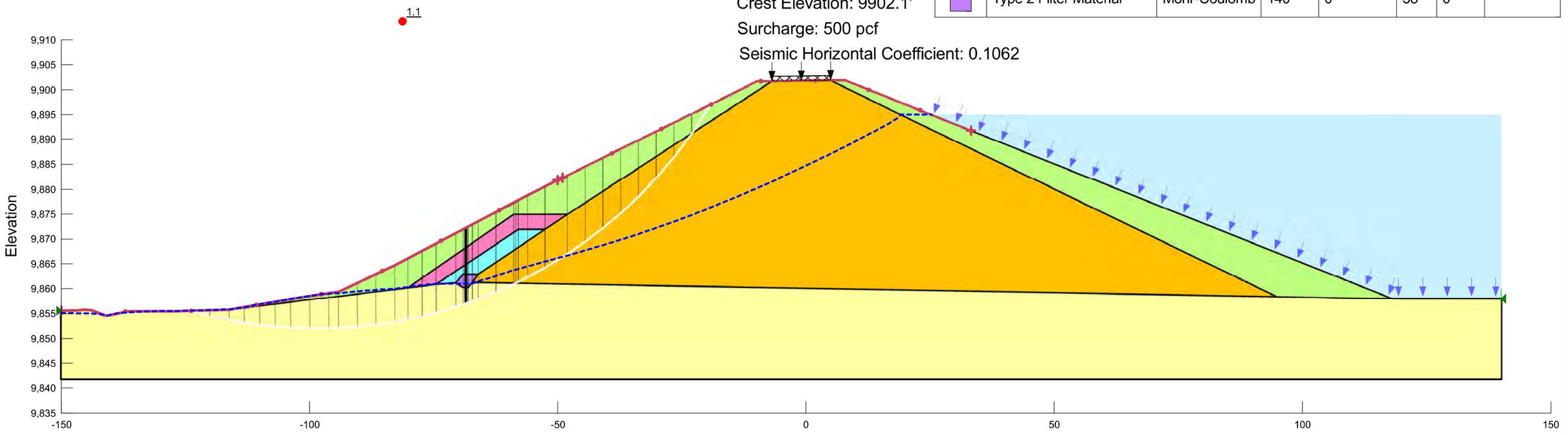
Project: DSB - Carson Dam, Station 20+60
Seismic Stability
2/25/21 Proposed Conditions
Downstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'
Surcharge: 500 pcf
Seismic Horizontal Coefficient: 0.1062



Kind: Slope/W
Method: Morgenstern-Price

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Colorado Dam Safety Branch
Hogchute Dam Embankment

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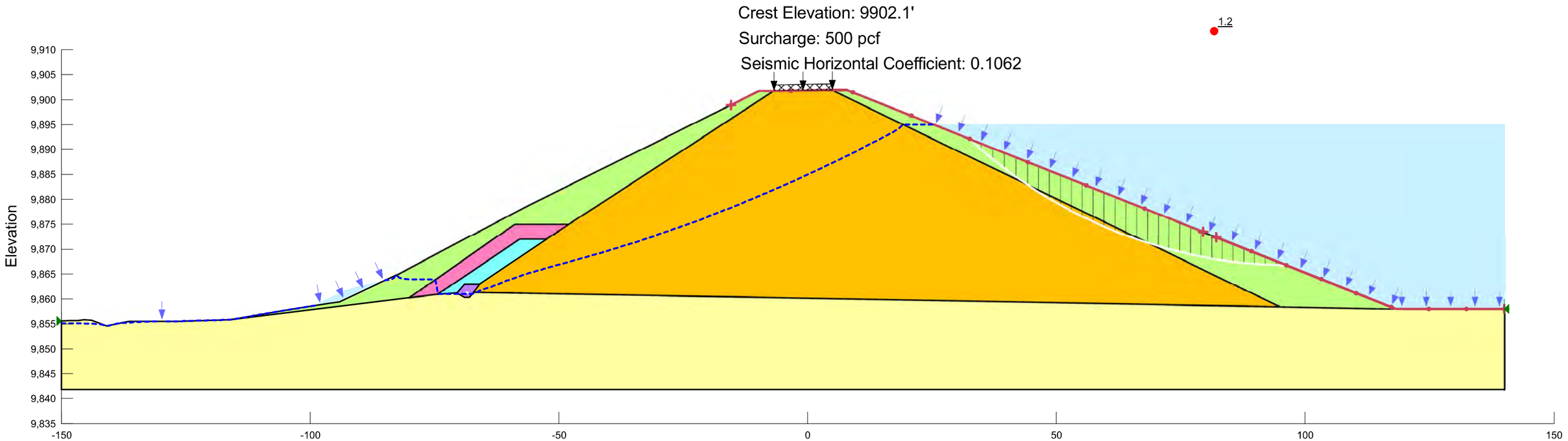
Project: DSB - Carson Dam, Station 20+60
Seismic Stability
2/25/21 Proposed Conditions
Upstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	



Kind: Slope/W
Method: Morgenstern-Price

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February 21, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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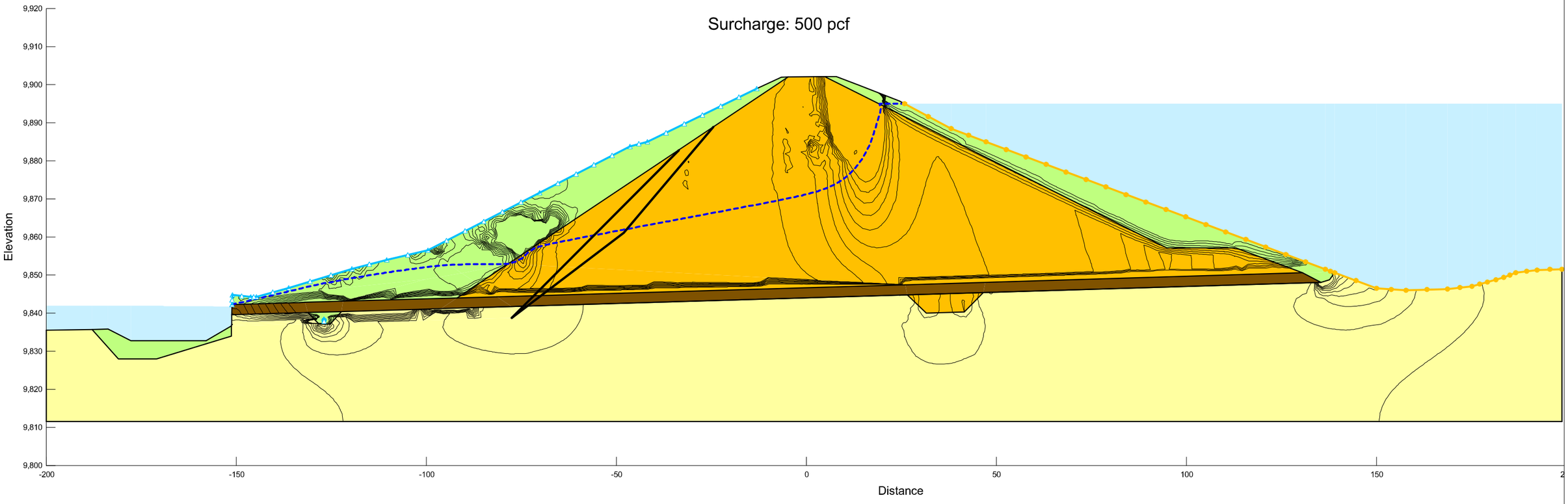
Project: DSB - Hogchute Dam, Station 21+50
Seepage Stability
2/25/21 Existing Conditions
Seepage Material, Normal Head Water Pool EL. 9895'

Color	Name	Category	Kind	Parameters
	9895.0 Nominal Reservoir Level Pressure	Hydraulic	Water Total Head	9,895 ft
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Zero Boundary Condition	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Embankment Fill		Embankment	Embankment	15	
	Outlet Conduit	1			0.001	80

Crest Elevation: 9902.1'

Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2020

Colorado Dam Safety Branch
Hogchute Dam Embankment

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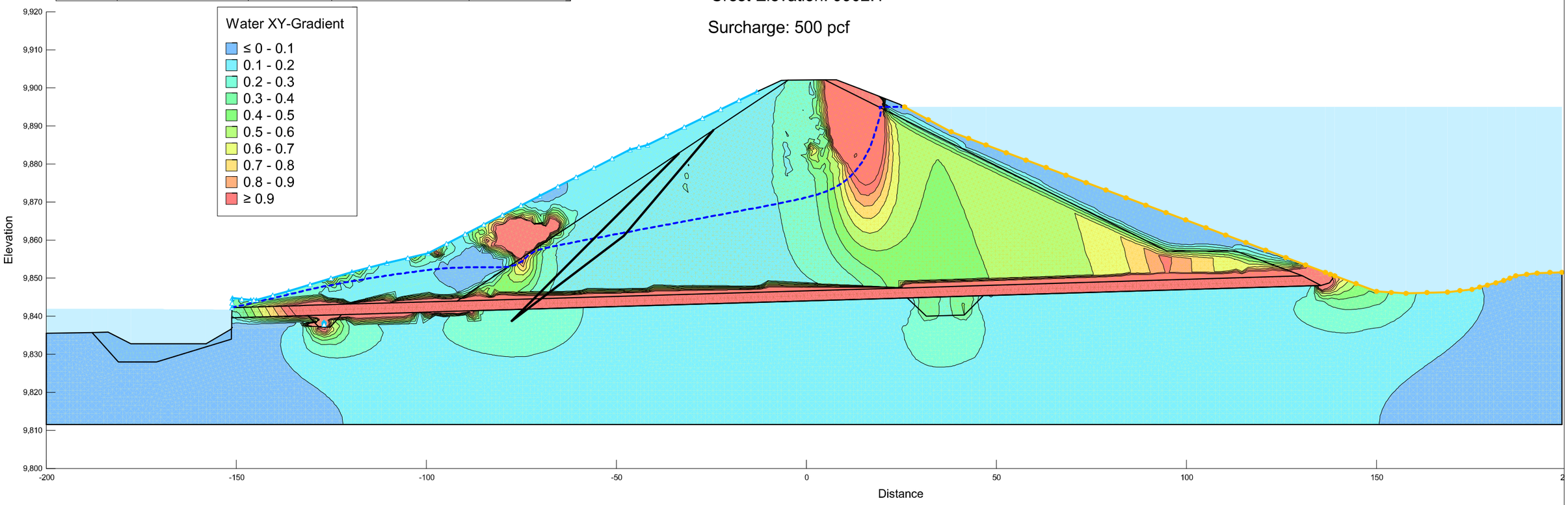
Project: DSB - Hogchute Dam, Station 21+50
Seepage Stability
2/25/21 Existing Conditions
Seepage Gradient, Normal Head Water Pool EL. 9895'

Color	Name	Category	Kind	Parameters
	9895.0 Nominal Reservoir Level Pressure	Hydraulic	Water Total Head	9,895 ft
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Zero Boundary Condition	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Embankment Fill		Embankment	Embankment	15	
	Outlet Conduit	1			0.001	80

Crest Elevation: 9902.1'

Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2020

Colorado Dam Safety Branch
Hogchute Dam Embankment

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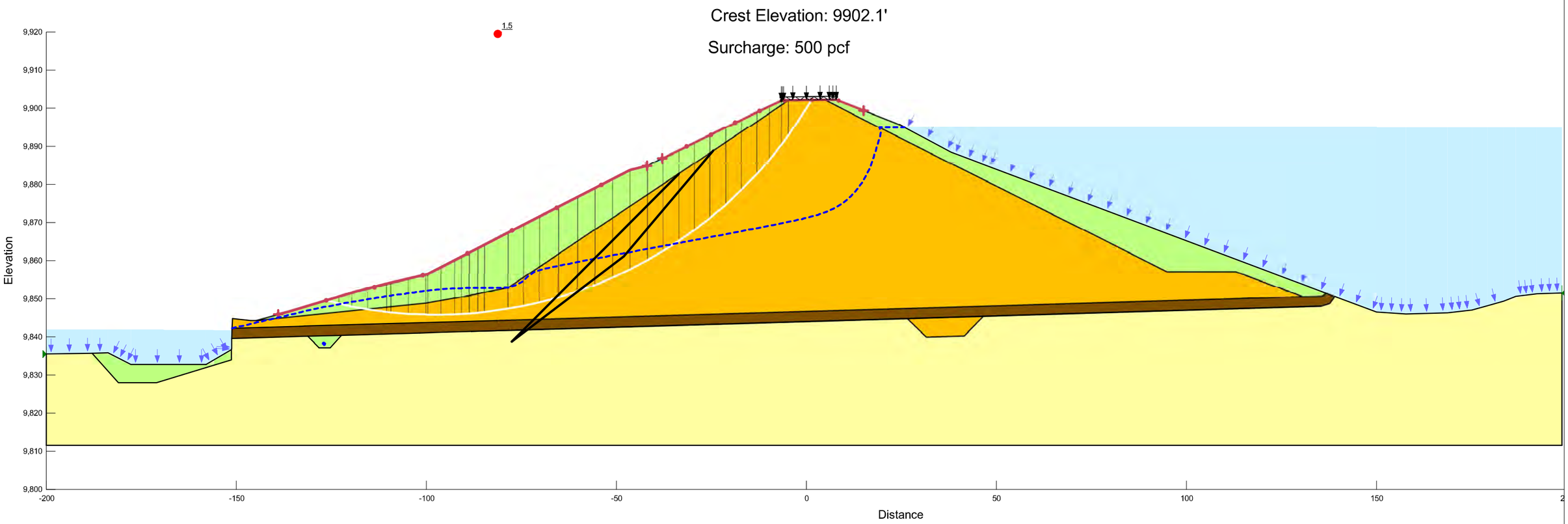
Project: DSB - Hogchute Dam, Station 21+50
Slope Stability
2/25/21 Existing Conditions
Downstream Slope, Normal Head Water Pool EL. 9895'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Outlet Conduit	High Strength	157.5				



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2020

Colorado Dam Safety Branch
Hogchute Dam Embankment

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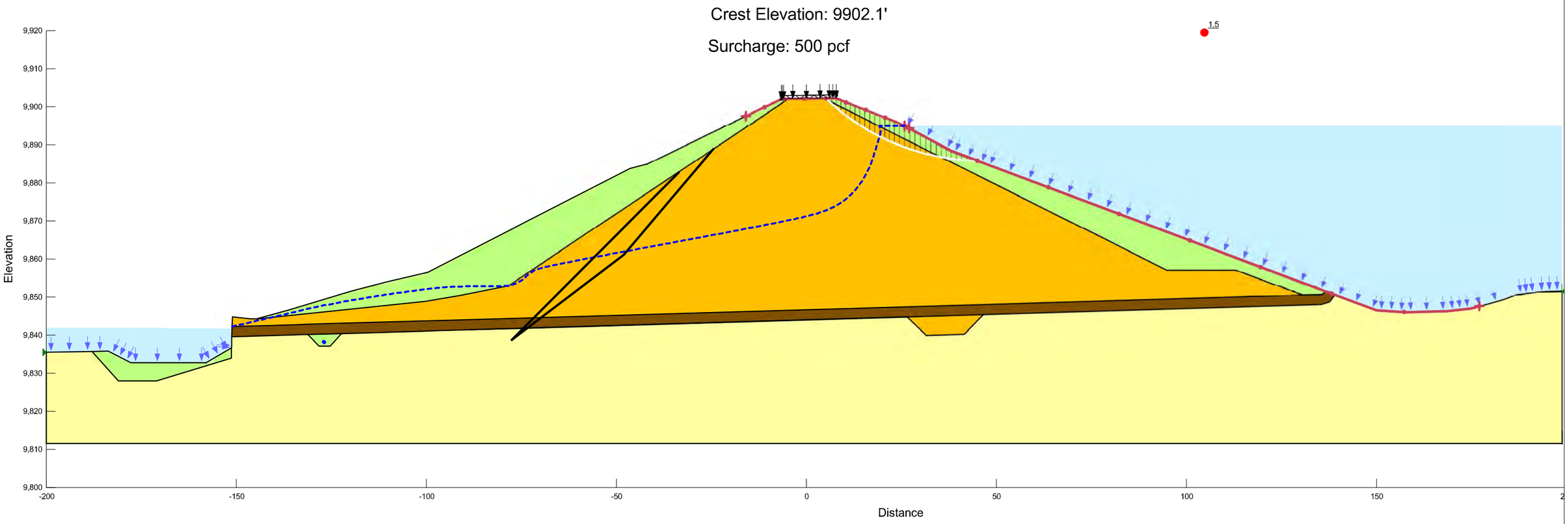
Project: DSB - Hogchute Dam, Station 21+50
Slope Stability
2/25/21 Existing Conditions
Upstream Slope, Normal Head Water Pool EL. 9895'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Outlet Conduit	High Strength	157.5				



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2020

Colorado Dam Safety Branch
Hogchute Dam Embankment

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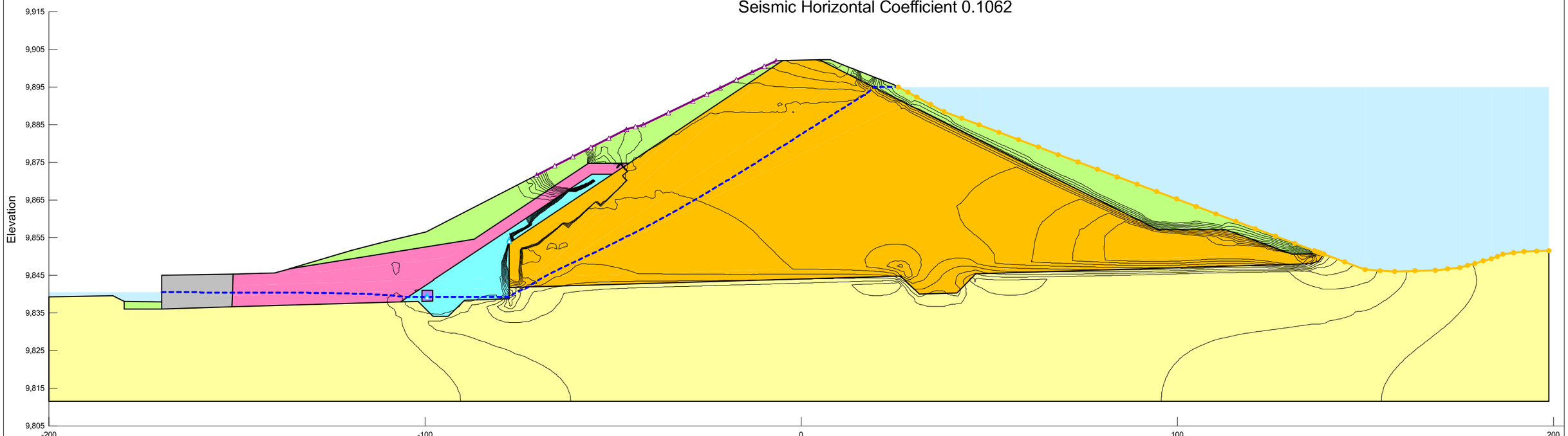
Project: DSB - Carson Dam, Station 21+50
Seepage Stability
2/25/21 Proposed Conditions
Seepage Material, Normal Pool Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	9895.0 Nominal Reservoir Level Pressure	Hydraulic	Water Total Head	9,895 ft
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Zero pressure Head	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Compacted Embankment Fill		Embankment	Embankment	0.3	
	Concrete		Concrete	Concrete	0.3	
	Embankment Fill		Embankment	Embankment	0.3	
	Type 1 Filter Material		Type 1 Filter	Type 1 Filter	0.3	
	Type 2 Filter Material		Type II Material	Type II Material	0.3	

Crest Elevation: 9902.1'
Surcharge: 500 pcf
Seismic Horizontal Coefficient 0.1062



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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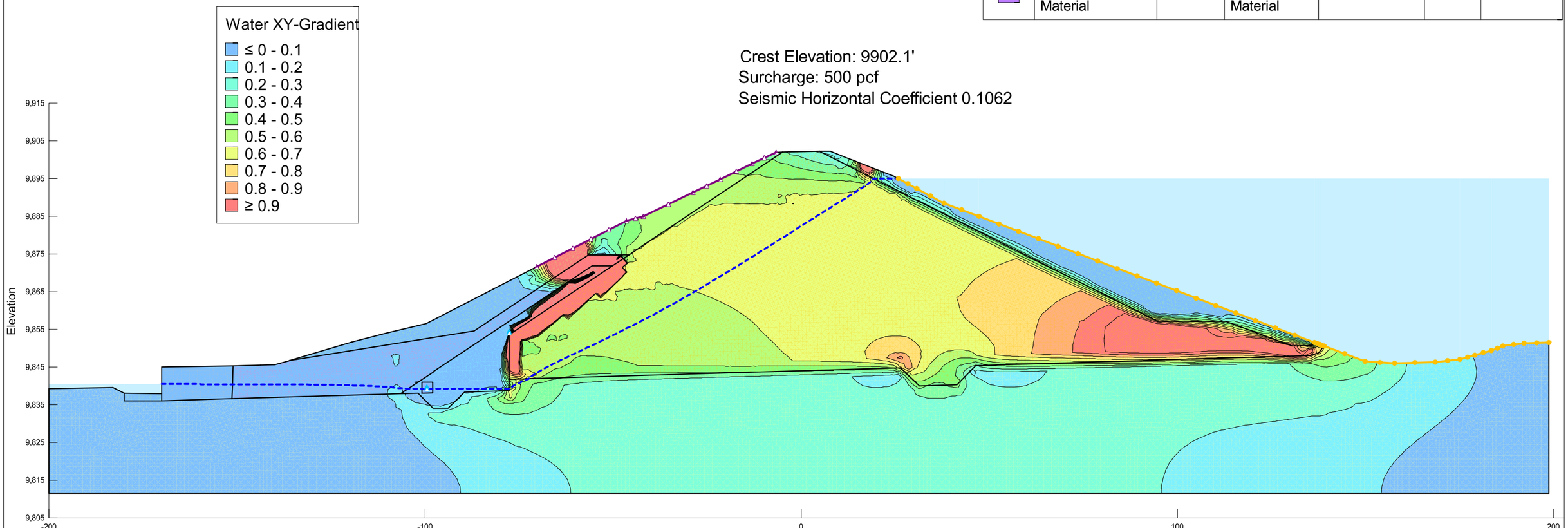
Project: DSB - Carson Dam, Station 21+50
Seepage Stability
2/25/21 Proposed Conditions
Seepage Gradient, Normal Pool Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	9895.0 Nominal Reservoir Level Pressure	Hydraulic	Water Total Head	9,895 ft
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Zero pressure Head	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Compacted Embankment Fill		Embankment	Embankment	0.3	
	Concrete		Concrete	Concrete	0.3	
	Embankment Fill		Embankment	Embankment	0.3	
	Type 1 Filter Material		Type 1 Filter	Type 1 Filter	0.3	
	Type 2 Filter Material		Type II Material	Type II Material	0.3	



Kind: Slope/W
Method: Morgenstern-Price

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Colorado Dam Safety Branch
Hogchute Dam Embankment

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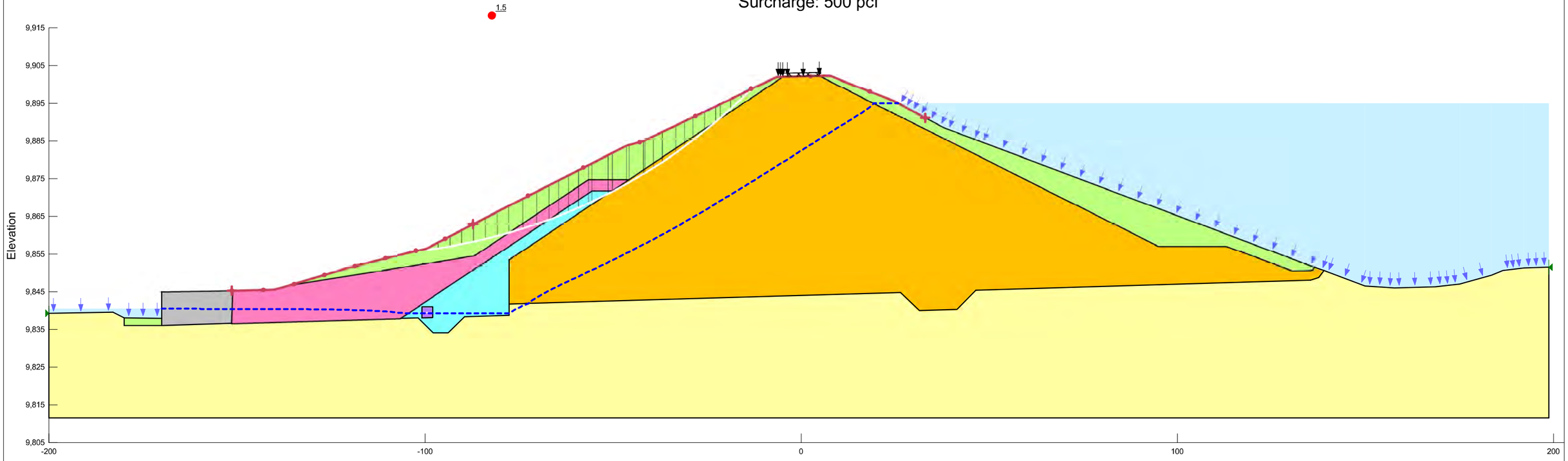
Project: DSB - Carson Dam, Station 21+50
Slope Stability
2/25/21 Proposed Conditions
Downstream Slope, Normal Pool Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment Fill	Mohr-Coulomb	130	0	34	0	
	Concrete	Bedrock (Impenetrable)					
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'
Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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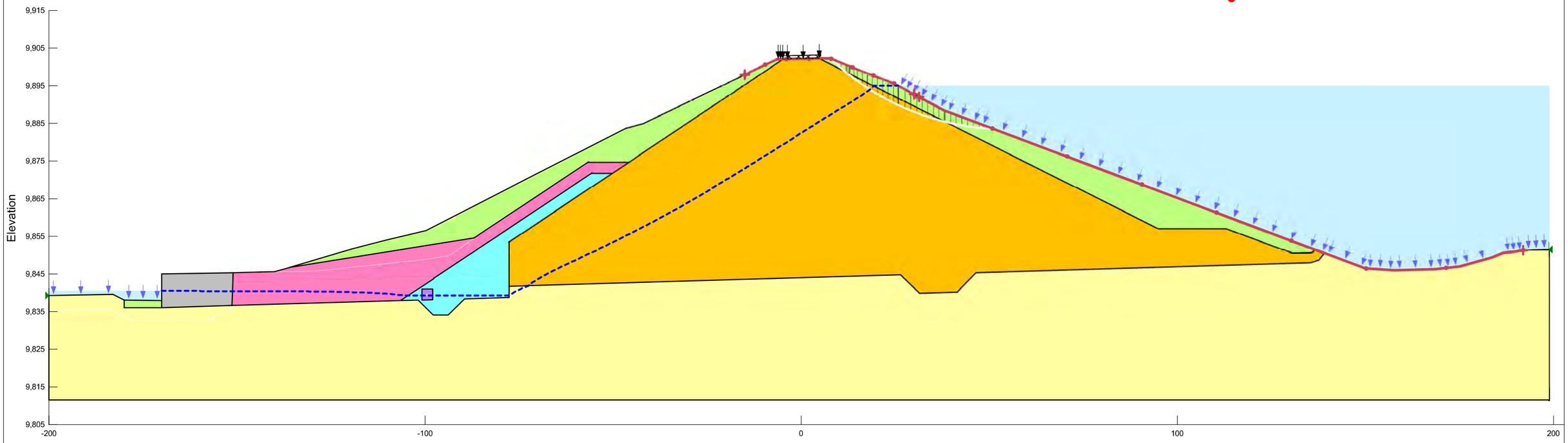
Project: DSB - Carson Dam, Station 21+50
Slope Stability
2/25/21 Proposed Conditions
Upstream Slope, Normal Pool Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment Fill	Mohr-Coulomb	130	0	34	0	
	Concrete	Bedrock (Impenetrable)					
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'
Surcharge: 500 pcf

1.6



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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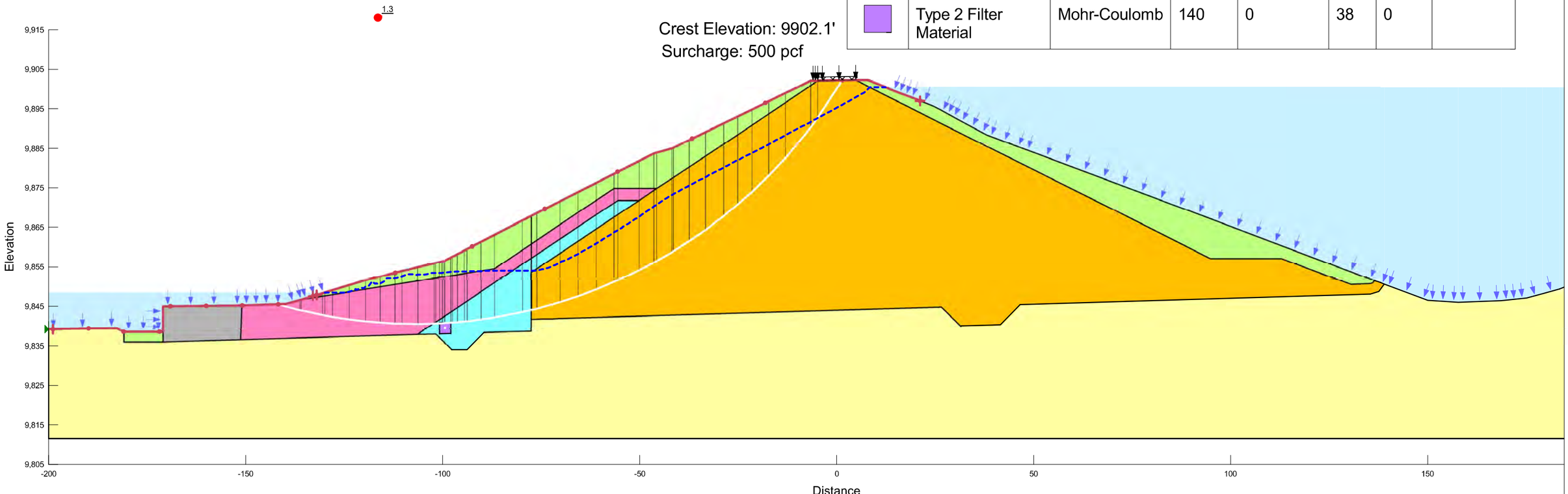
Project: DSB - Hogchute Dam, Station 21+50
Slope Stability
2/25/21 Proposed Conditions
Downstream Slope, IDF 9900.4' with Surcharge 9848.5'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	32	0	
	Concrete	Bedrock (Impenetrable)					
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'
Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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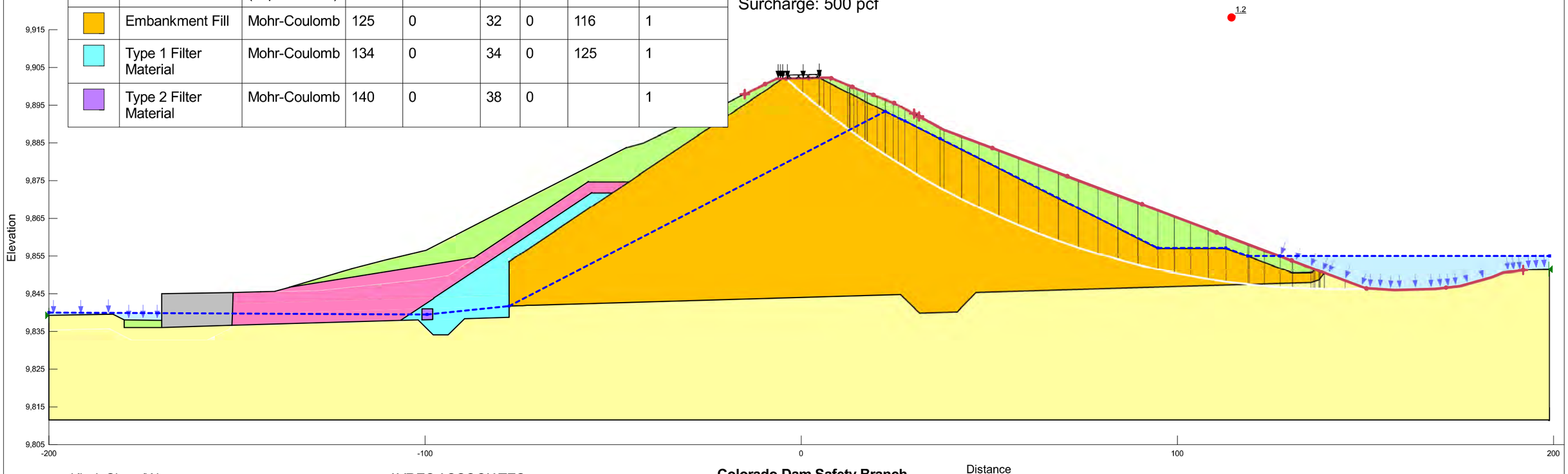
Project: DSB - Carson Dam, Station 21+50
Slope Stability
2/25/21 Proposed Conditions
Upstream Slope, Rapid Drawdown Head Water Pool EL. 9855.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)	Piezometric Line
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125	1
	Colluvium	Mohr-Coulomb	130	0	34	0	106	1
	Compacted Embankment Fill	Mohr-Coulomb	130	0	34	0		1
	Concrete	Bedrock (Impenetrable)						1
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116	1
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125	1
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0		1

Crest Elevation: 9902.1'
Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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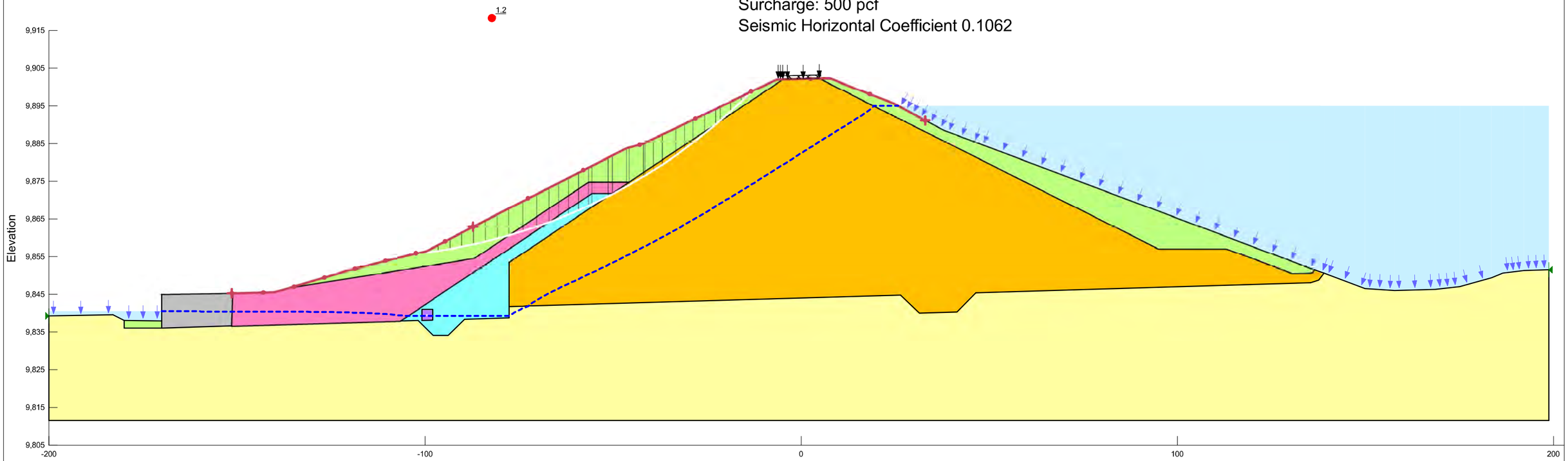
Project: DSB - Carson Dam, Station 21+50
Seismic Stability
2/25/21 Proposed Conditions
Downstream Slope, Normal Pool Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment Fill	Mohr-Coulomb	130	0	34	0	
	Concrete	Bedrock (Impenetrable)					
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'
Surcharge: 500 pcf
Seismic Horizontal Coefficient 0.1062



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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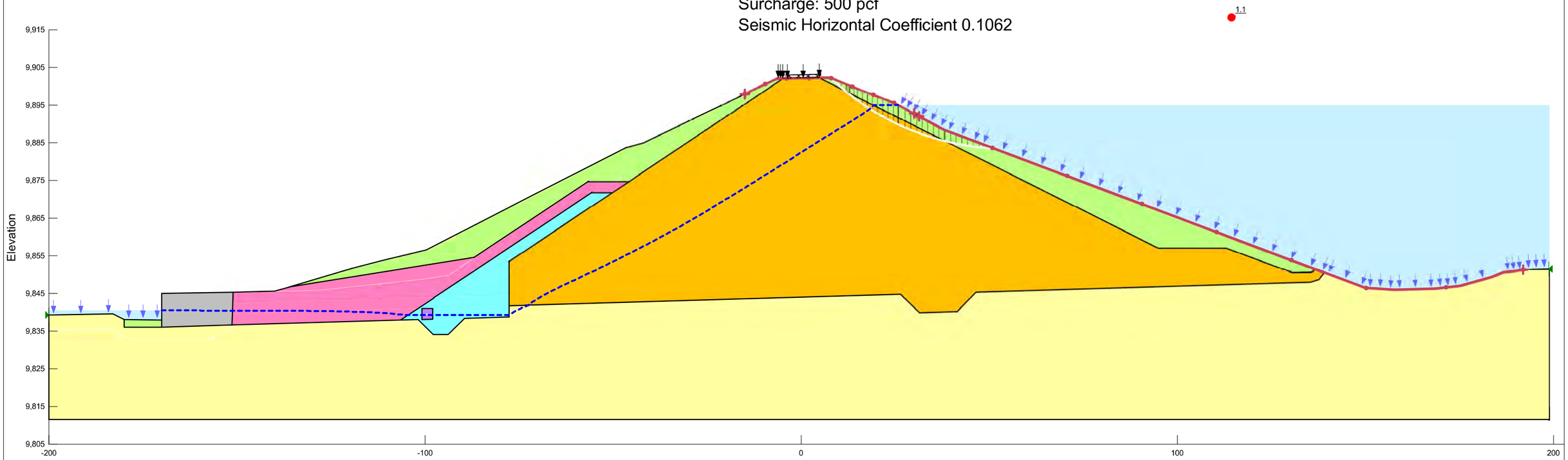
Project: DSB - Carson Dam, Station 21+50
Seismic Stability
2/25/21 Proposed Conditions
Upstream Slope, Normal Pool Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment Fill	Mohr-Coulomb	130	0	34	0	
	Concrete	Bedrock (Impenetrable)					
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter Material	Mohr-Coulomb	134	0	34	0	125
	Type 2 Filter Material	Mohr-Coulomb	140	0	38	0	

Crest Elevation: 9902.1'
Surcharge: 500 pcf
Seismic Horizontal Coefficient 0.1062



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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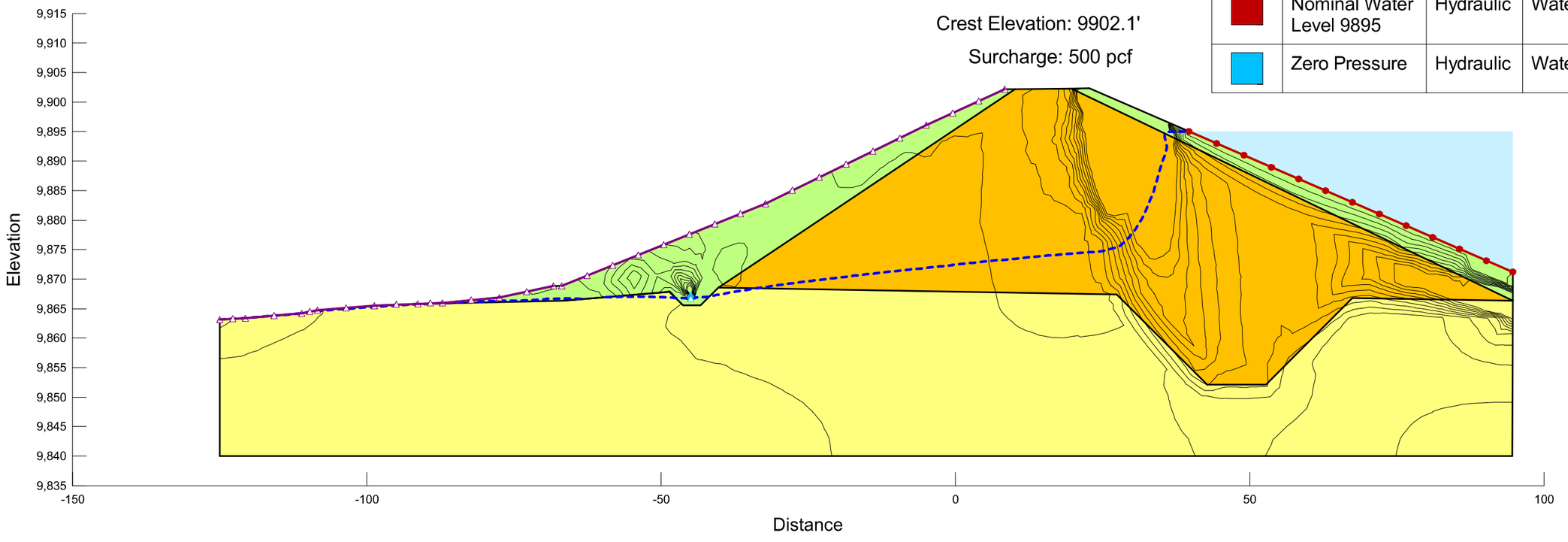
Project: DSB - Carson Dam, Station 22+50
Seepage Stability
2/25/21 Existing Conditions
Seepage Material, Normal Head Water Pool EL. 9895.0'

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Embankment Fill		Embankment	Embankment	15	

Color	Name	Category	Kind	Parameters
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Nominal Water Level 9895	Hydraulic	Water Total Head	9,895 ft
	Zero Pressure	Hydraulic	Water Rate	0 ft³/sec

Crest Elevation: 9902.1'

Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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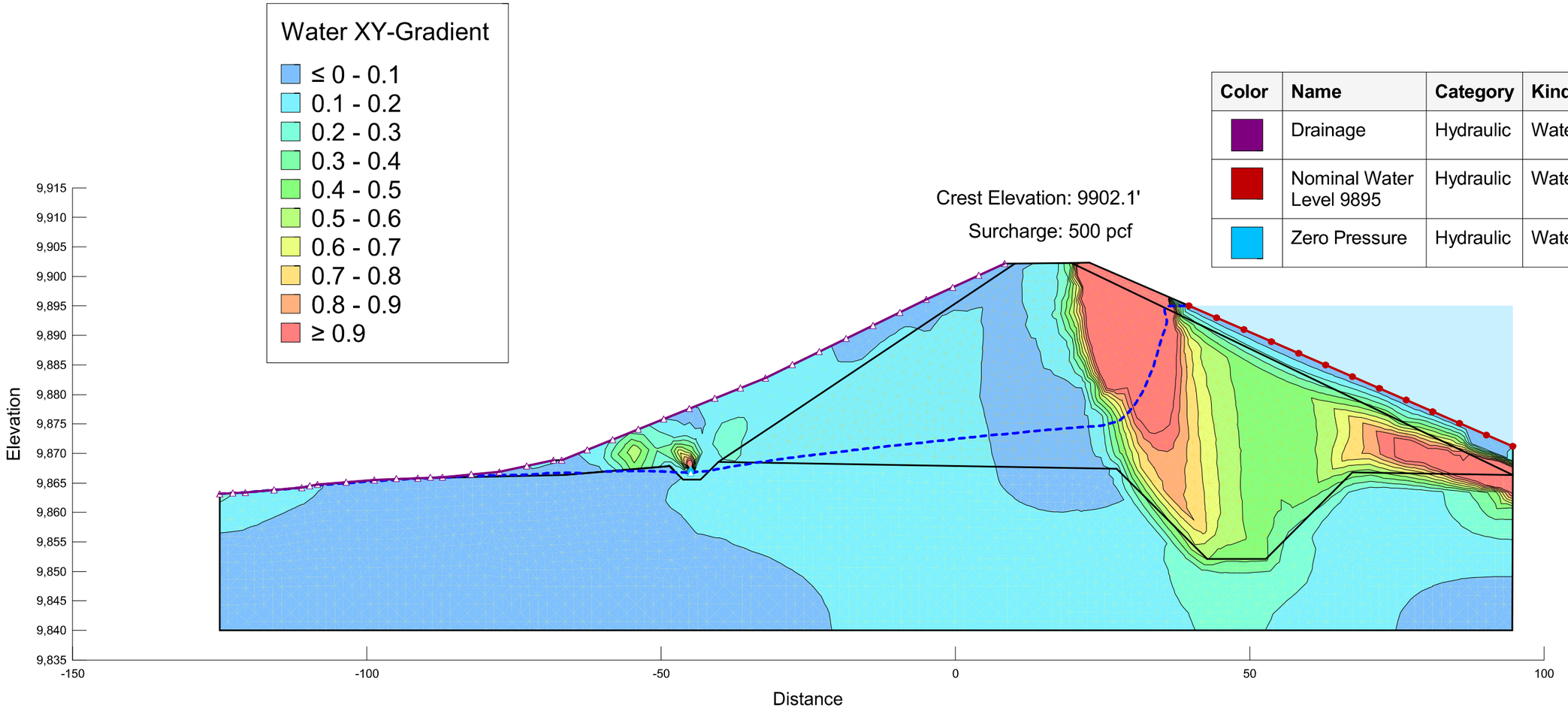
Project: DSB - Carson Dam, Station 22+50
Seepage Stability
2/25/21 Existing Conditions
Seepage Gradient, Normal Head Water Pool EL. 9895.0'

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Embankment Fill		Embankment	Embankment	15	



Color	Name	Category	Kind	Parameters
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Nominal Water Level 9895	Hydraulic	Water Total Head	9,895 ft
	Zero Pressure	Hydraulic	Water Rate	0 ft³/sec

Kind: Slope/W
Method: Morgenstern-Price

AYRES ASSOCIATES
February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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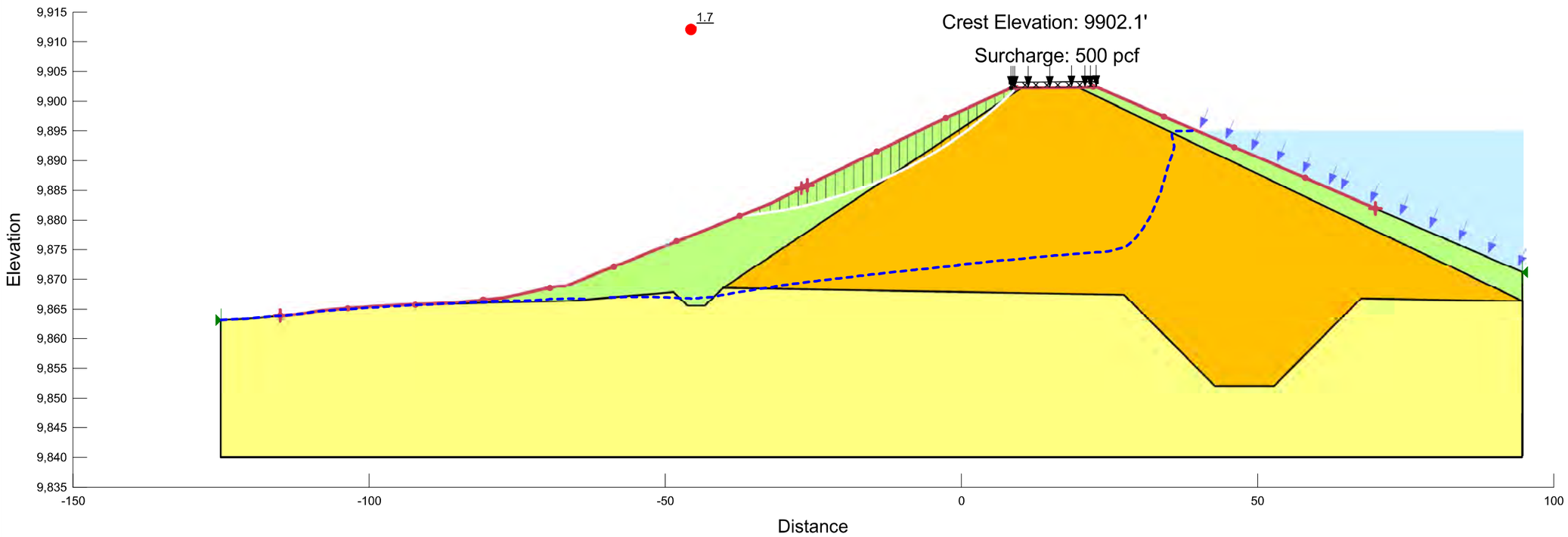
Project: DSB - Carson Dam, Station 22+50
Slope Stability
2/25/21 Existing Conditions
Downstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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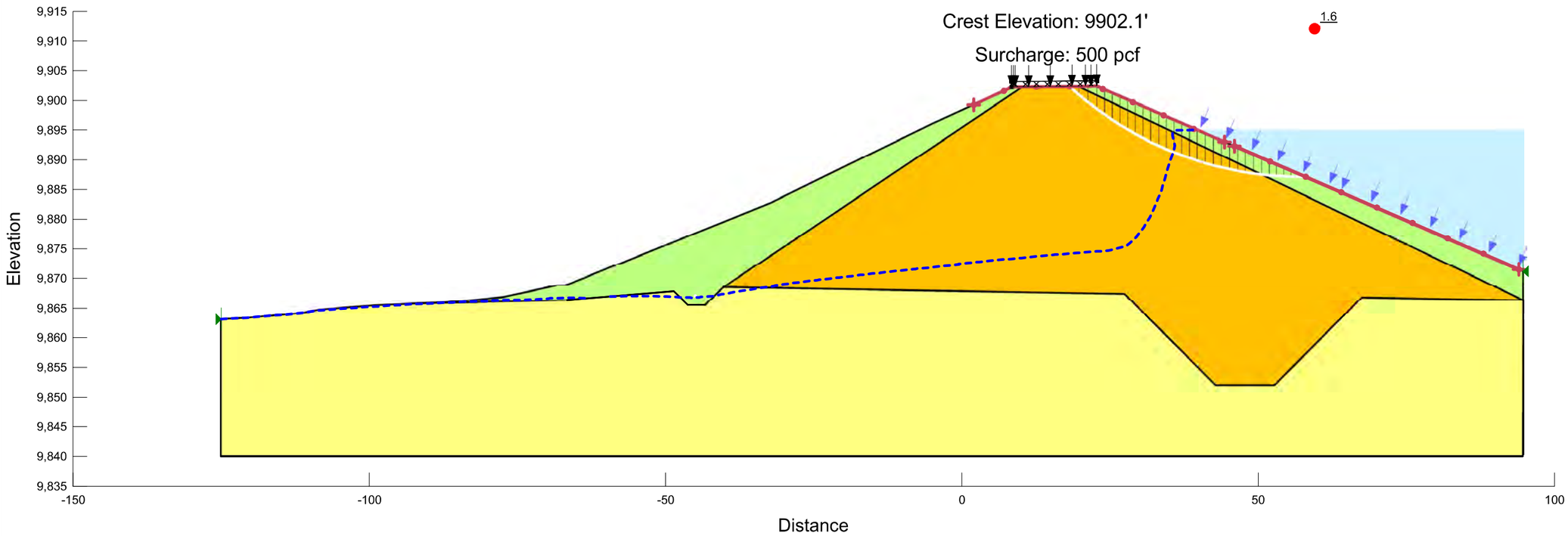
Project: DSB - Carson Dam, Station 22+50
Slope Stability
2/25/21 Existing Conditions
Upstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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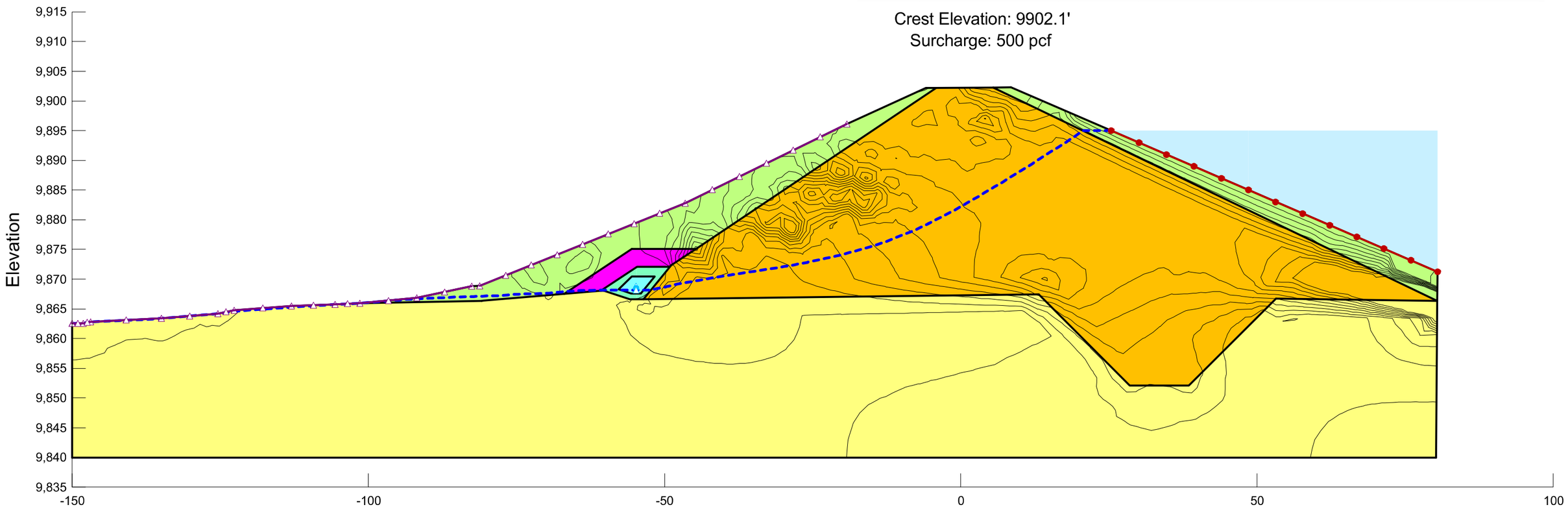
Project: DSB - Carson Dam, Station 22+50
Seepage Stability
2/25/21 Proposed Conditions
Seepage Material, Normal Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Nominal Water Level 9895	Hydraulic	Water Total Head	9,895 ft
	Zero Pressure	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Compacted Embankment		Embankment	Embankment	0.3	
	Embankment Fill		Embankment	Embankment	0.3	
	Type 1 Filter		Type 1 Filter	Type 1 Filter	0.3	
	Type II Filter		Type II Material	Type II Material	0.3	

Crest Elevation: 9902.1'
Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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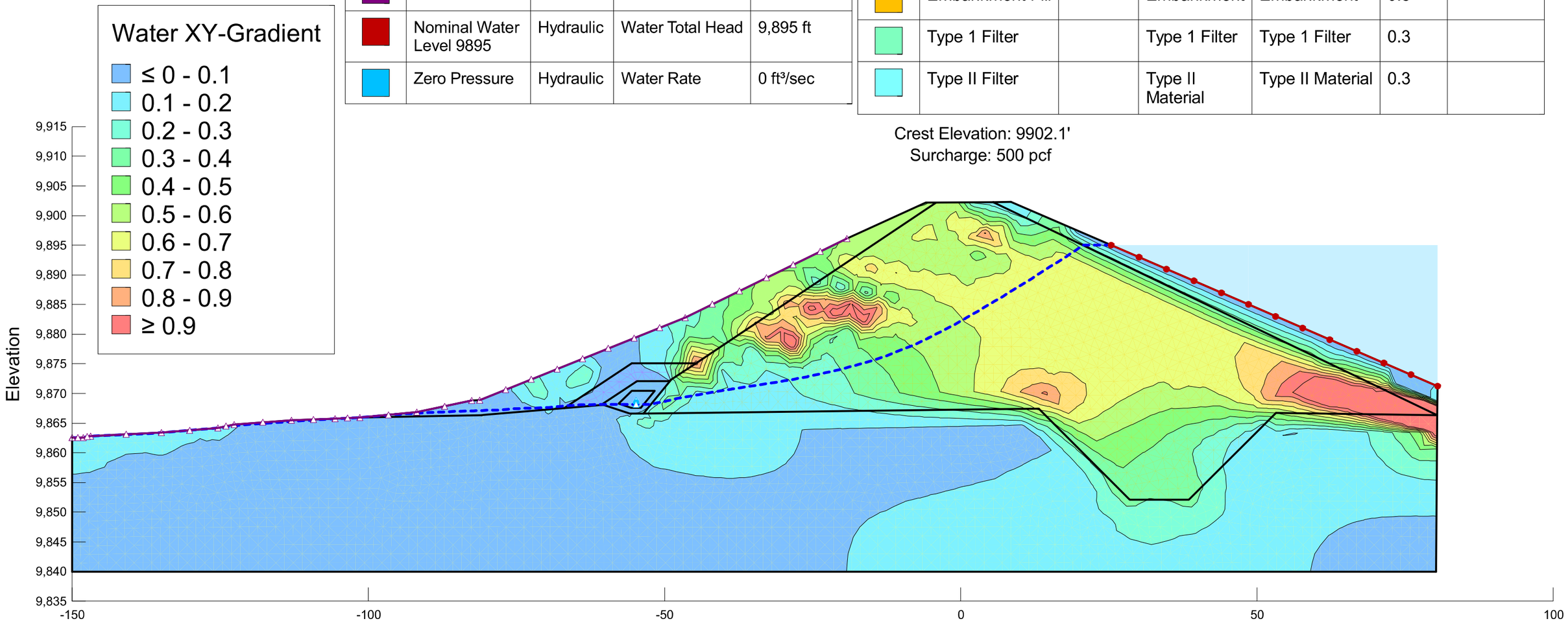
Project: DSB - Carson Dam, Station 22+50
Seepage Stability
2/25/21 Proposed Conditions
Seepage Gradient, Normal Head Water Pool EL. 9895.0'

Color	Name	Category	Kind	Parameters
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	Nominal Water Level 9895	Hydraulic	Water Total Head	9,895 ft
	Zero Pressure	Hydraulic	Water Rate	0 ft³/sec

Color	Name	Sat Kx (cm/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Volumetric Water Content
	Cobble and Rock Fill		Rock Fill	Rock fill	1	
	Colluvium	0.0031			0.52	25.2
	Compacted Embankment		Embankment	Embankment	0.3	
	Embankment Fill		Embankment	Embankment	0.3	
	Type 1 Filter		Type 1 Filter	Type 1 Filter	0.3	
	Type II Filter		Type II Material	Type II Material	0.3	

Crest Elevation: 9902.1'
Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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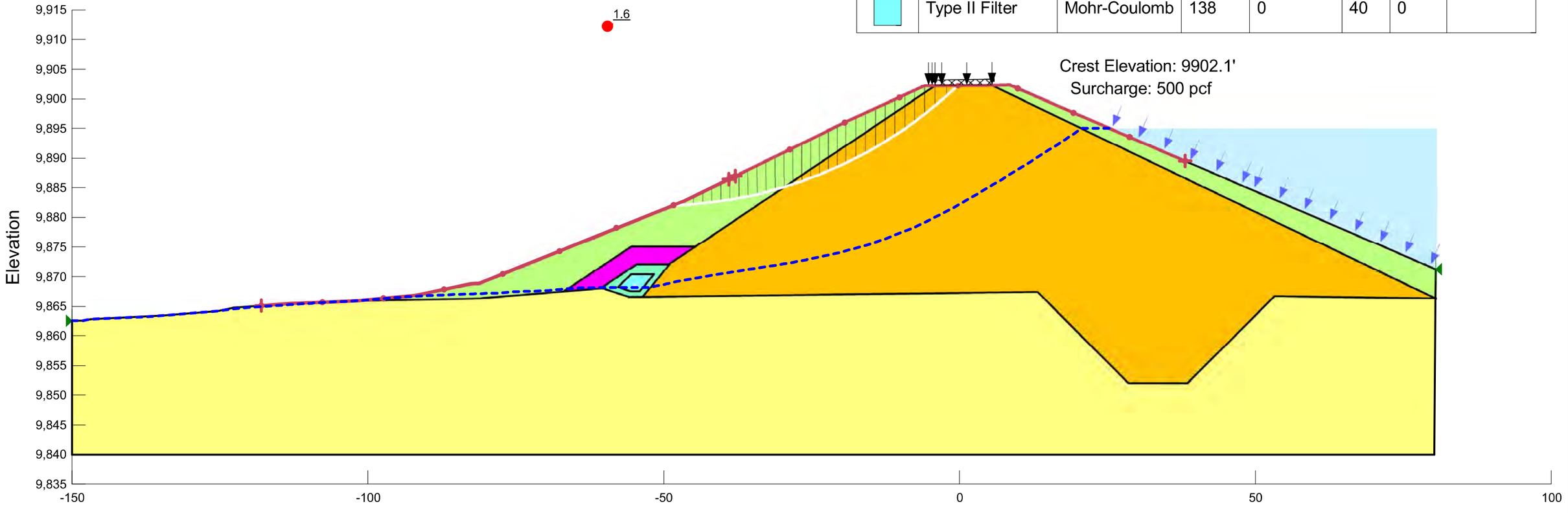
Project: DSB - Carson Dam, Station 22+50
Slope Stability
2/25/21 Proposed Conditions
Downstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116
	Type 1 Filter	Mohr-Coulomb	134	0	34	0	125
	Type II Filter	Mohr-Coulomb	138	0	40	0	



Kind: Slope/W
Method: Morgenstern-Price

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February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

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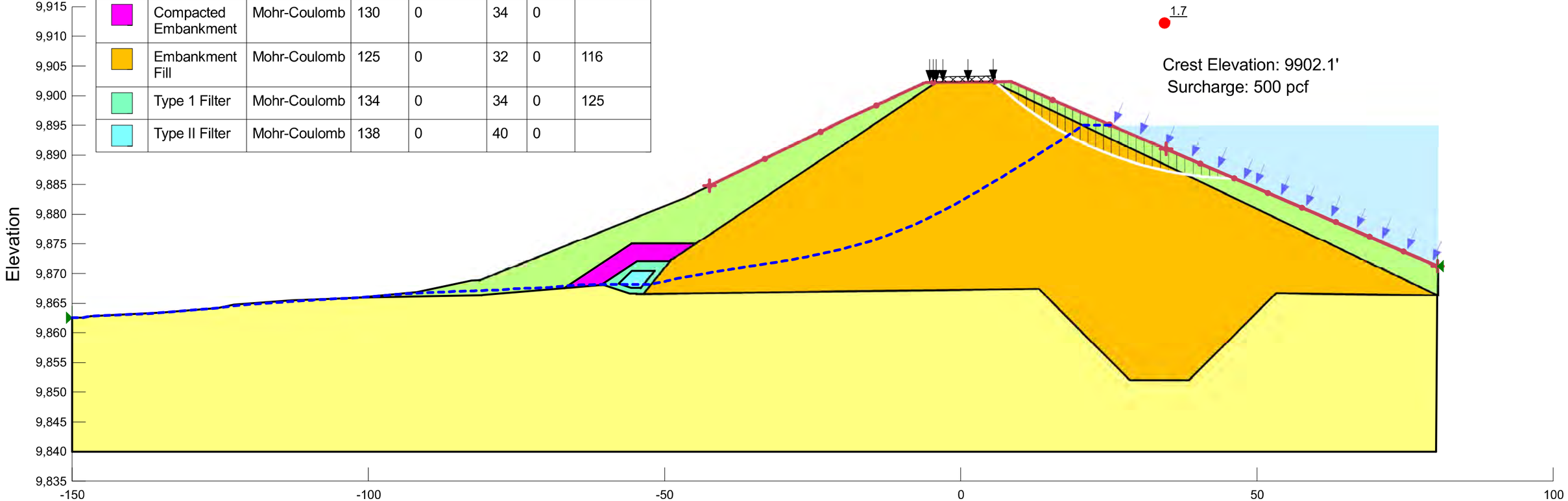
Project: DSB - Carson Dam, Station 22+50
Slope Stability
2/25/21 Proposed Conditions
Upstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter	Mohr-Coulomb	134	0	34	0	125
	Type II Filter	Mohr-Coulomb	138	0	40	0	



Kind: Slope/W
Method: Morgenstern-Price

AYRES ASSOCIATES
February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

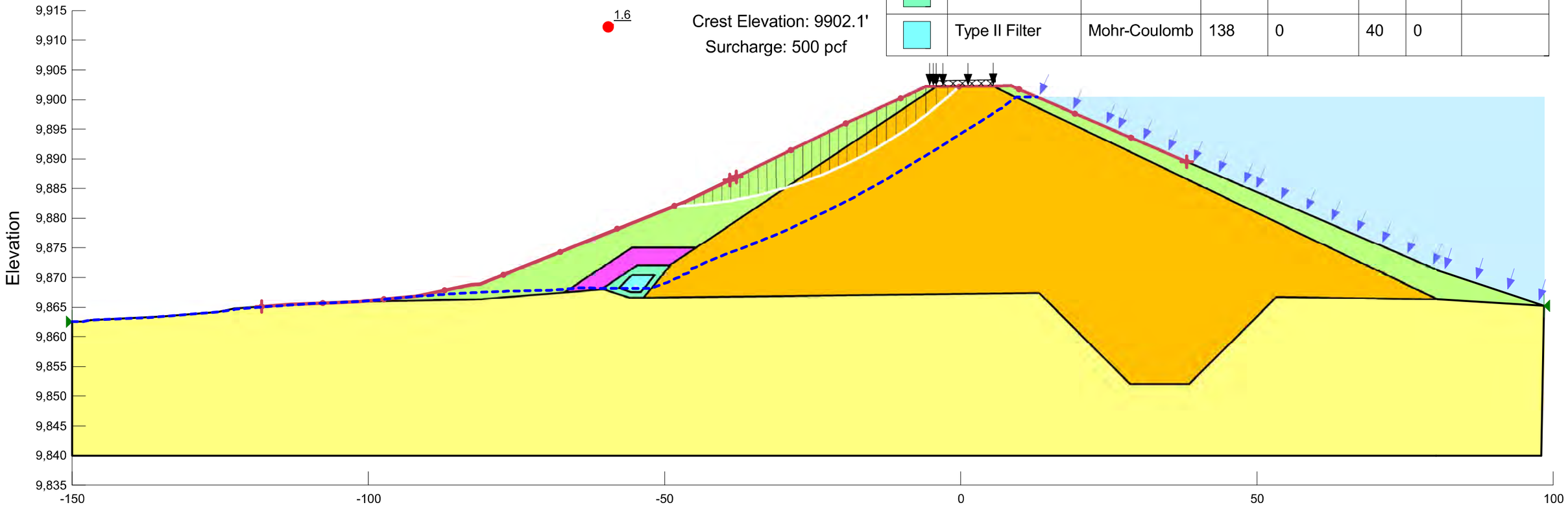
I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 22+50
Slope Stability
2/25/21 Proposed Conditions
Downstream Slope, IDF Head Water Pool EL. 9900.4'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	32	0	
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter	Mohr-Coulomb	134	0	34	0	125
	Type II Filter	Mohr-Coulomb	138	0	40	0	

1.6

Crest Elevation: 9902.1'
Surcharge: 500 pcf



Kind: Slope/W
Method: Morgenstern-Price

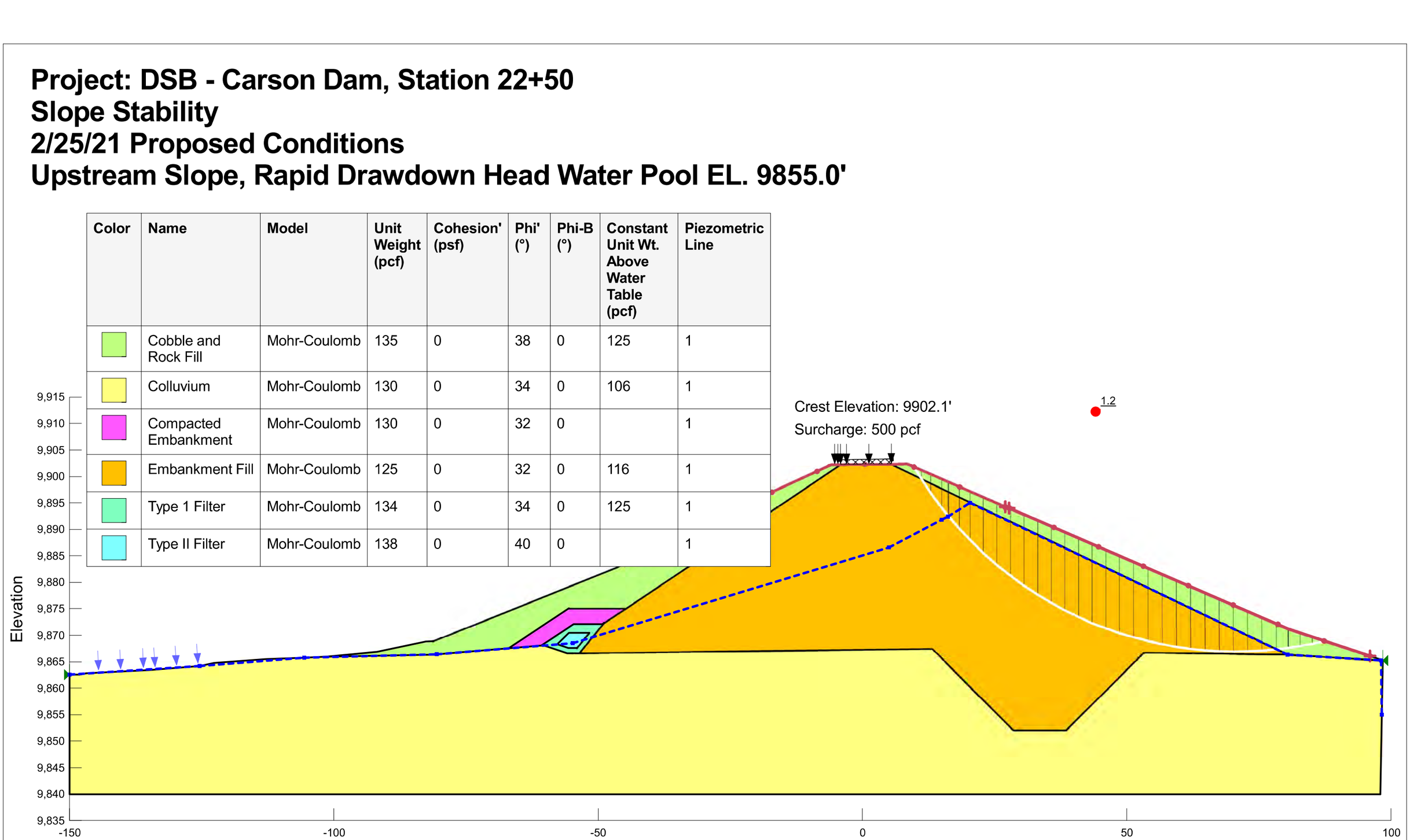
AYRES ASSOCIATES
February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 22+50
Slope Stability
2/25/21 Proposed Conditions
Upstream Slope, Rapid Drawdown Head Water Pool EL. 9855.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)	Piezometric Line
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125	1
	Colluvium	Mohr-Coulomb	130	0	34	0	106	1
	Compacted Embankment	Mohr-Coulomb	130	0	32	0		1
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116	1
	Type 1 Filter	Mohr-Coulomb	134	0	34	0	125	1
	Type II Filter	Mohr-Coulomb	138	0	40	0		1



Kind: Slope/W
Method: Morgenstern-Price

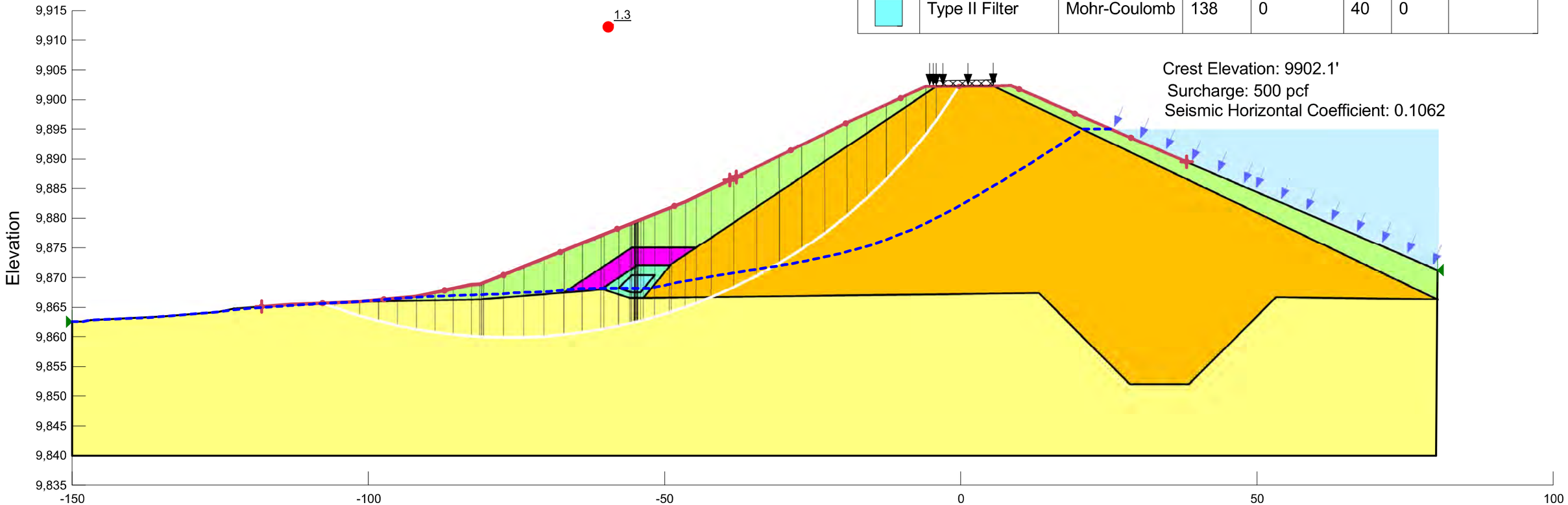
AYRES ASSOCIATES
February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 22+50
Seismic Stability
2/25/21 Proposed Conditions
Downstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	39	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	34	0	116
	Type 1 Filter	Mohr-Coulomb	134	0	34	0	125
	Type II Filter	Mohr-Coulomb	138	0	40	0	



Kind: Slope/W
Method: Morgenstern-Price

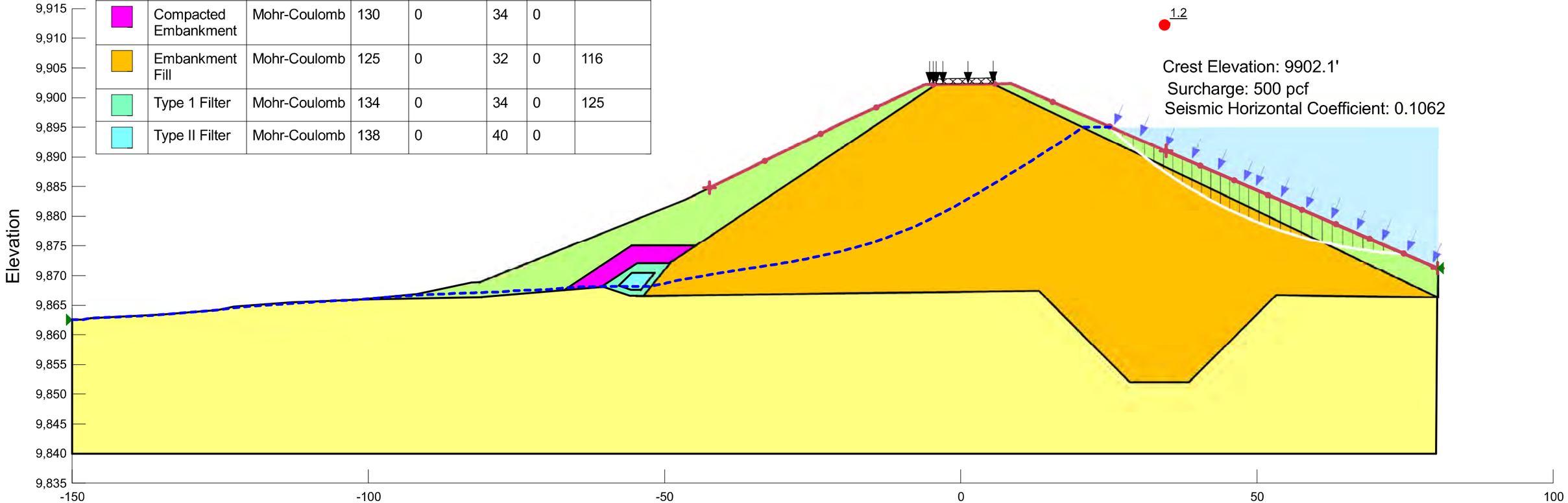
AYRES ASSOCIATES
February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 22+50
Seismic Stability
2/25/21 Proposed Conditions
Upstream Slope, Normal Head Water Pool EL. 9895.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Compacted Embankment	Mohr-Coulomb	130	0	34	0	
	Embankment Fill	Mohr-Coulomb	125	0	32	0	116
	Type 1 Filter	Mohr-Coulomb	134	0	34	0	125
	Type II Filter	Mohr-Coulomb	138	0	40	0	



Kind: Slope/W
Method: Morgenstern-Price

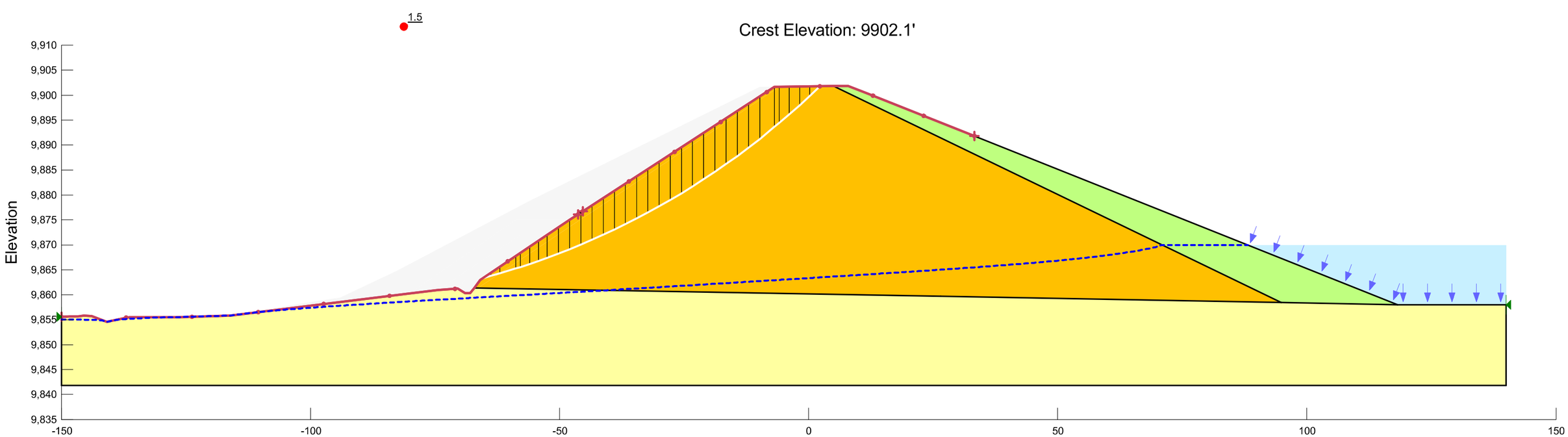
AYRES ASSOCIATES
February 25, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 20+60
Slope Stability
3/5/21 Construction Conditions
Downstream Slope, Head Water Pool EL. 9870'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	97	34	0	106
	Embankment Fill	Mohr-Coulomb	125	97	34	0	116



Kind: Slope/W
Method: Morgenstern-Price

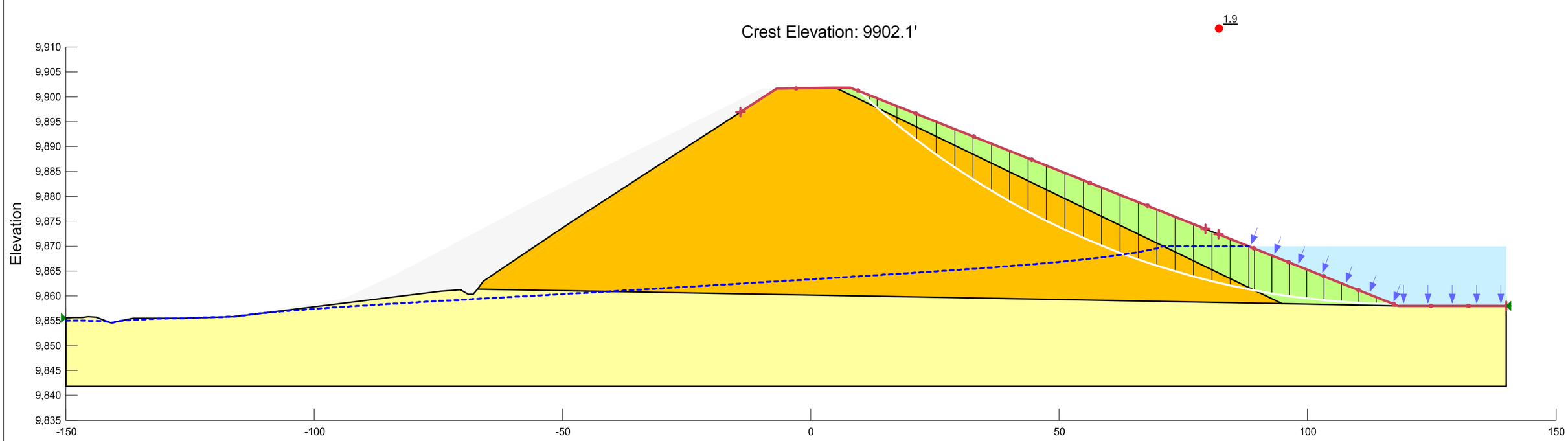
AYRES ASSOCIATES
March 5, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 20+60
Slope Stability
3/5/21 Construction Conditions
Upstream Slope, Head Water Pool EL. 9870'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	97	34	0	106
	Embankment Fill	Mohr-Coulomb	125	97	32	0	116



Kind: Slope/W
Method: Morgenstern-Price

AYRES ASSOCIATES
March 5, 2021

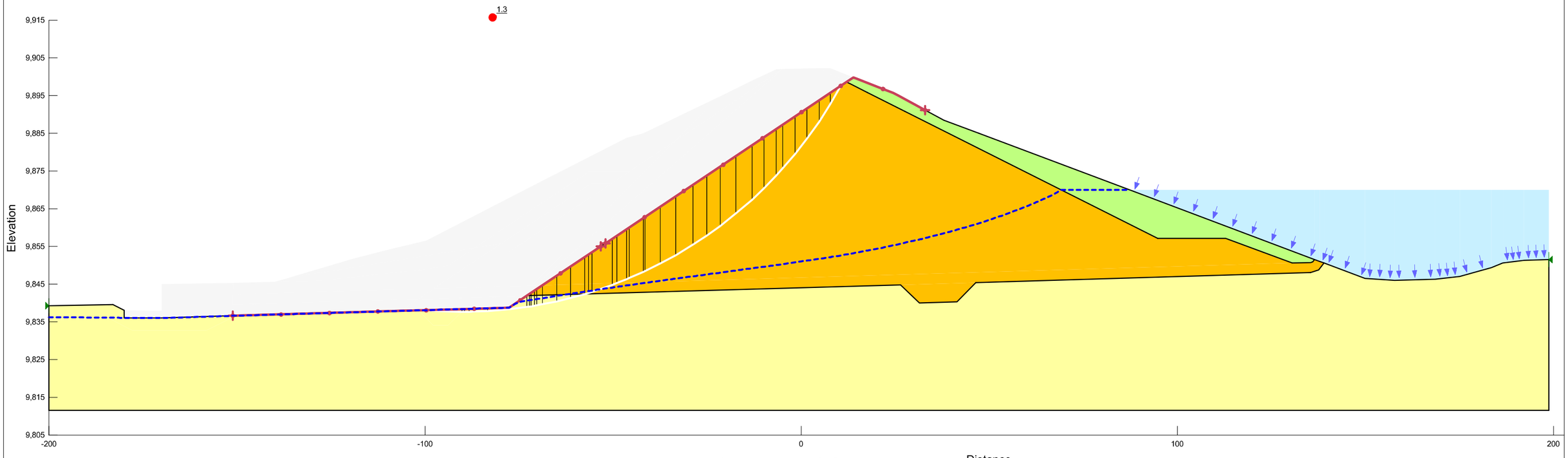
Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 21+50
Slope Stability
3/5/21 Construction Conditions
Downstream Slope, Head Water Pool EL. 9870.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	97	34	0	

Crest Elevation: 9902.1'



Kind: Slope/W
Method: Morgenstern-Price

AYRES ASSOCIATES
March 5, 2021

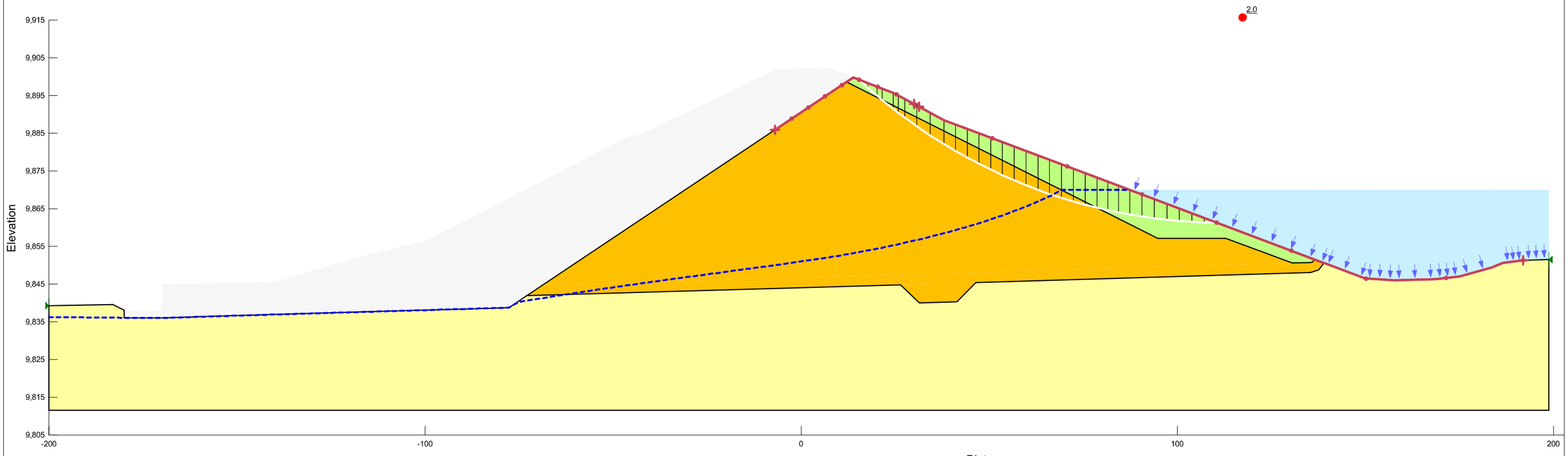
Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 21+50
Slope Stability
3/5/21 Construction Conditions
Upstream Slope, Head Water Pool EL. 9870.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	0	34	0	106
	Embankment Fill	Mohr-Coulomb	125	97	34	0	

Crest Elevation: 9902.1'



Kind: Slope/W
Method: Morgenstern-Price

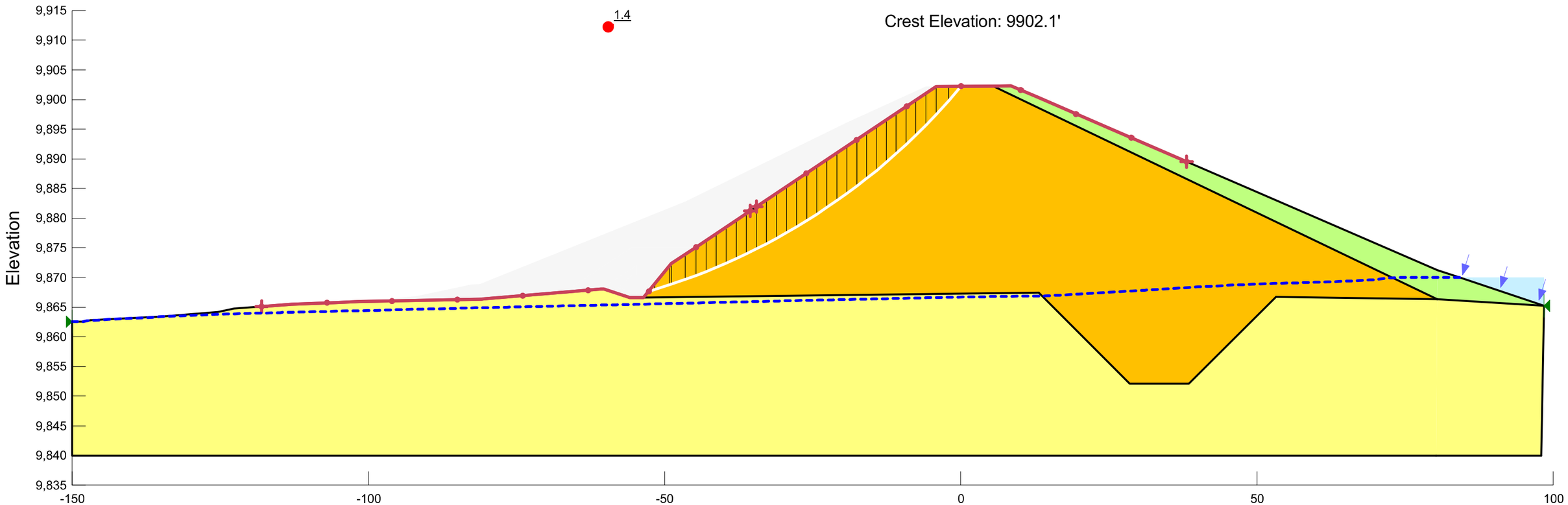
AYRES ASSOCIATES
March 5, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 22+50
Slope Stability
3/5/21 Construction Conditions
Downstream Slope, Head Water Pool EL. 9870.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	97	34	0	106
	Embankment Fill	Mohr-Coulomb	125	97	34	0	116



Kind: Slope/W
Method: Morgenstern-Price

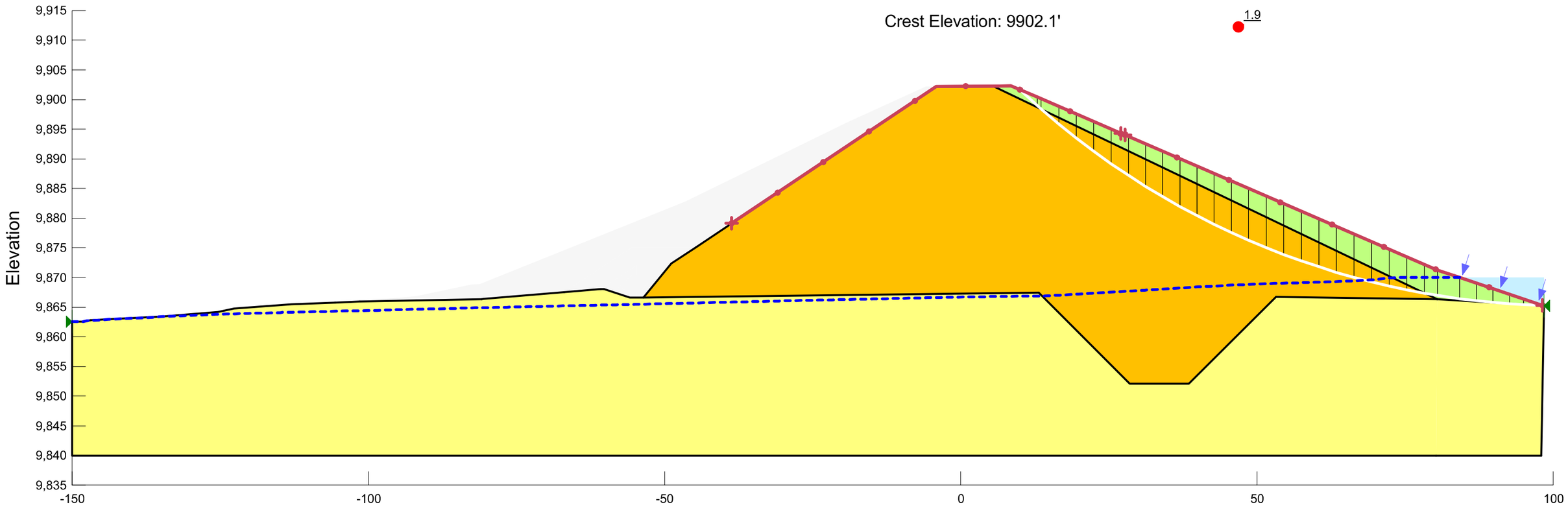
AYRES ASSOCIATES
March 5, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Project: DSB - Carson Dam, Station 22+50
Slope Stability
3/5/21 Construction Conditions
Upstream Slope, Head Water Pool EL. 9870.0'

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)
	Cobble and Rock Fill	Mohr-Coulomb	135	0	38	0	125
	Colluvium	Mohr-Coulomb	130	97	34	0	106
	Embankment Fill	Mohr-Coulomb	125	97	32	0	116



Kind: Slope/W
Method: Morgenstern-Price

AYRES ASSOCIATES
March 5, 2021

Colorado Dam Safety Branch
Hogchute Dam Embankment

I:\26\Grand Junction CO\Geostudio

Appendix F
Filter and Drain Design

LEGEND

- ◉ MONITORING WELL
- ◆ BORING BACKFILLED WITH CEMENT-BENTONITE GROUT

NOTES:

1. AERIAL IMAGERY WAS OBTAINED FROM MICROSOFT BING, IMAGERY DATE UNKNOWN.
2. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
3. LOCATIONS OF MONITORING WELLS AND BORINGS WERE OBTAINED FROM HANDHELD GPS UNIT AND HAVE NOT BEEN SURVEYED.

REPRODUCE IN COLOR

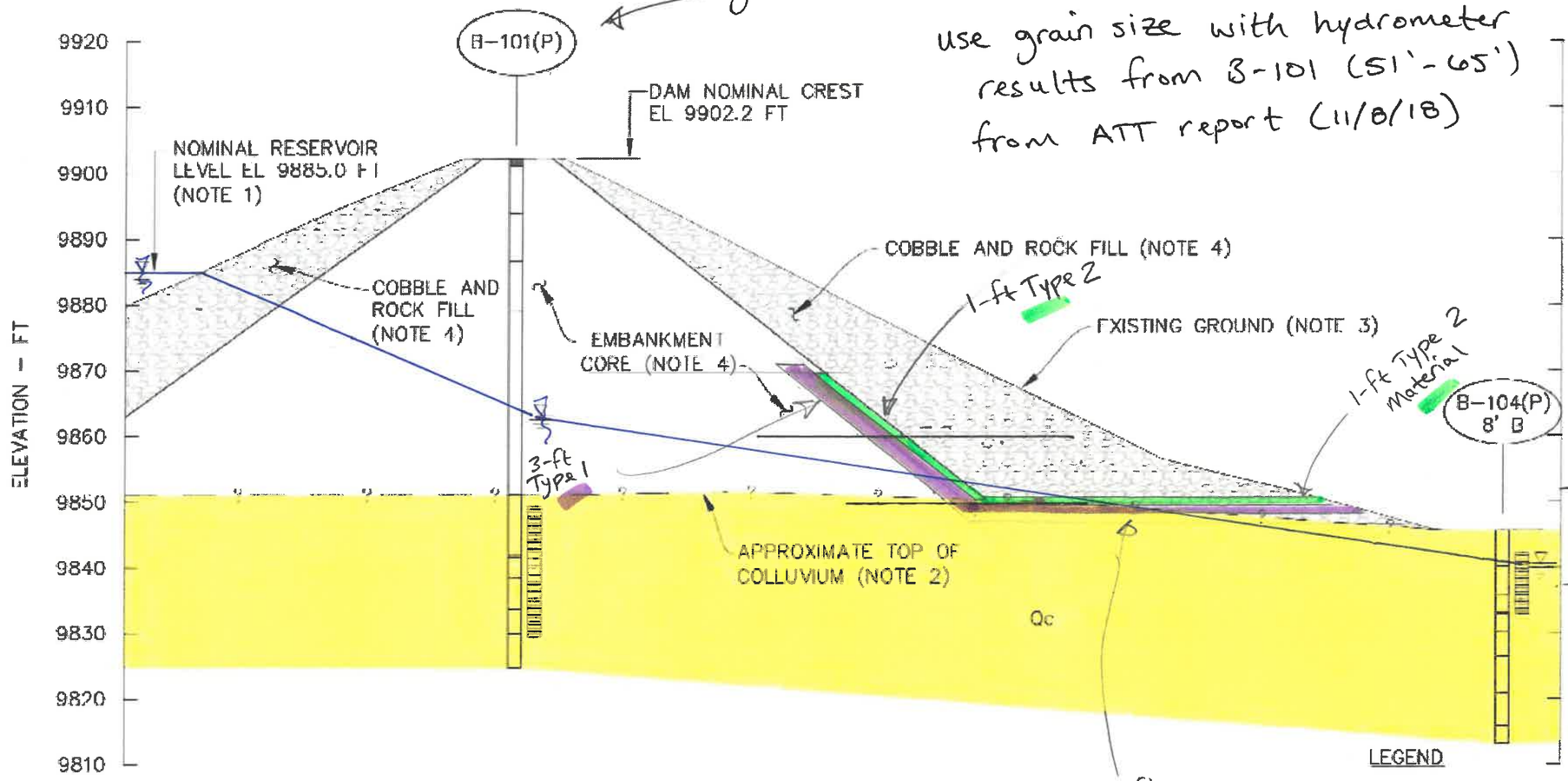
City of Grand Junction Colorado	RJH Consultants, Inc.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	GEOTECHNICAL INVESTIGATION BORING AND PIEZOMETER LOCATIONS
		PROJECT NO. 18115	January 2019 **Figure 4.1**

P:\18115 - HOGCHUTE DAM SAFETY EVALUATION\CAD\FIGURES\MONITORING WELLS\MONITORING WELLS.DWG 1/2/2019 4:03 PM

grain size 51'-65' below surface

Use grain size with hydrometer results from B-101 (51'-65') from ATT report (11/8/18)

ASTM C-33 No. 58



grain size 9837' - 9851'

SECTION A 1.2

0 10 20 10 60

SCALE IN FEET

LEGEND

- ROAD BASE
- EMBANKMENT FILL
- Qc COLLUVIUM
- MONITORING WELL SENSING ZONE
- GROUNDWATER LEVEL MEASURED FROM MONITORING WELL ON 9/22/2018
- B-101(P) 1' P — MONITORING WELL NAME; DISTANCE PROJECTED (P) OR BEYOND (B) SECTION LOCATION

NOTES:

1. NOMINAL RESERVOIR LEVEL DURING FINAL GROUNDWATER LEVEL MEASUREMENT ON 9/22/2018.
2. ALL GEOLOGICAL CONTACTS ARE APPROXIMATE AND NOT ALL INFORMATION IS SHOWN. BORING LOGS SHOWN GENERALIZE SUBSURFACE CONDITIONS AND ARE NOT INTENDED TO BE A COMPLETE OR ACCURATE DEPICTION OF THE SUBSURFACE CONDITIONS. COMPLETE BORING LOGS ARE PROVIDED IN APPENDIX B.
3. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
4. GEOMETRY OF EMBANKMENT ZONES WAS APPROXIMATED USING DATA FROM DESIGN DRAWINGS. OUTLET WORKS, TOE DRAIN, AND CUTOFF TRENCH NOT SHOWN.

City of Grand Junction Colorado	RJH Consultants, Inc.	HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	DAM SECTION A SOUTH
		PROJECT NO. 18115	January 2019 — Figure 4.3

P:\18115 HOGCHUTE DAM SAFETY EVALUATION\CAD\FIGURES\MONITORING WELLS MONITORING WELLS.DWG 1/2/2019 3:26 PM

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-101 ← use for filter design
JOB NO.	2679-130	DEPTH	51-65'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-20
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hygroscopic Moisture of Fines

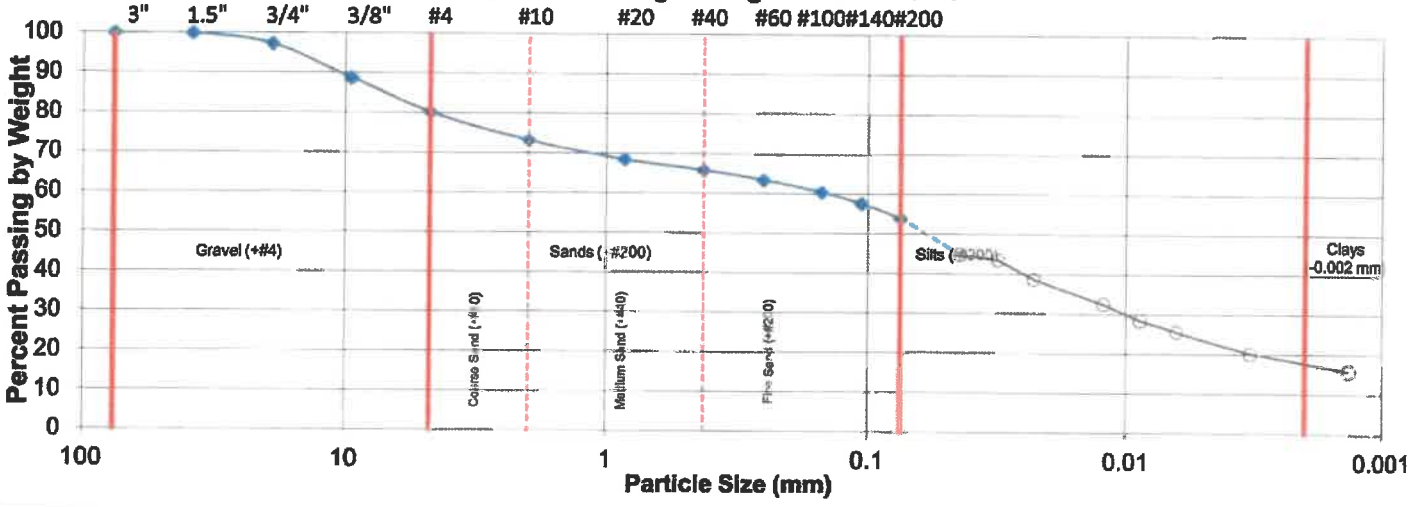
Mass Wet Pan and Soil (g): 116.73
Mass Dry Pan and Soil (g): 114.50
Mass of Pan (g): 3.16
Moisture (%): 2.0

Sample Data

Total Wet Mass of Sample (g): 19133.2
Total Dry Mass of Sample (g): 18857.9
Split Fraction: #10 3/8"
Mass of Sub-Sample Fraction (g): 60.91 1123.80

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	0.0	--	--	--	--
1.5"	38.1	0.0	--	--	--	--
3/4"	19.05	507.2	--	507.2	1.00	97.3
3/8"	9.53	1605.3	--	1605.3	1.00	88.8
#4	4.75	106.8	--	106.8	0.89	80.2
#10	2.00	87.4	--	87.39	0.888	73.2
#20	0.850	3.8	--	3.80	0.731	68.5
#40	0.425	2.2	--	2.15	0.731	65.9
#60	0.250	2.0	--	1.95	0.731	63.5
#100	0.150	2.5	--	2.50	0.731	60.5
#140	0.106	2.3	--	2.31	0.731	57.6
#200	0.075	3.0	--	2.98	0.731	54.0

Percent Passing vs Log of Particle Size



USCS Classification ASTM D 2487

Atterberg Classification: CL
Group Symbol: CL
USCS Classification: Sandy Lean Clay With Gravel

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: KMS Date: 11/8/2018
Checked by: KR Date: 11/8/18
File name: 2679130__Grain Size with Hydrometer ASTM D6913 D7928_0.xlsm

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-101
JOB NO.	2679-130	DEPTH	51-65'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-20
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hydrometer and Flask Parameters

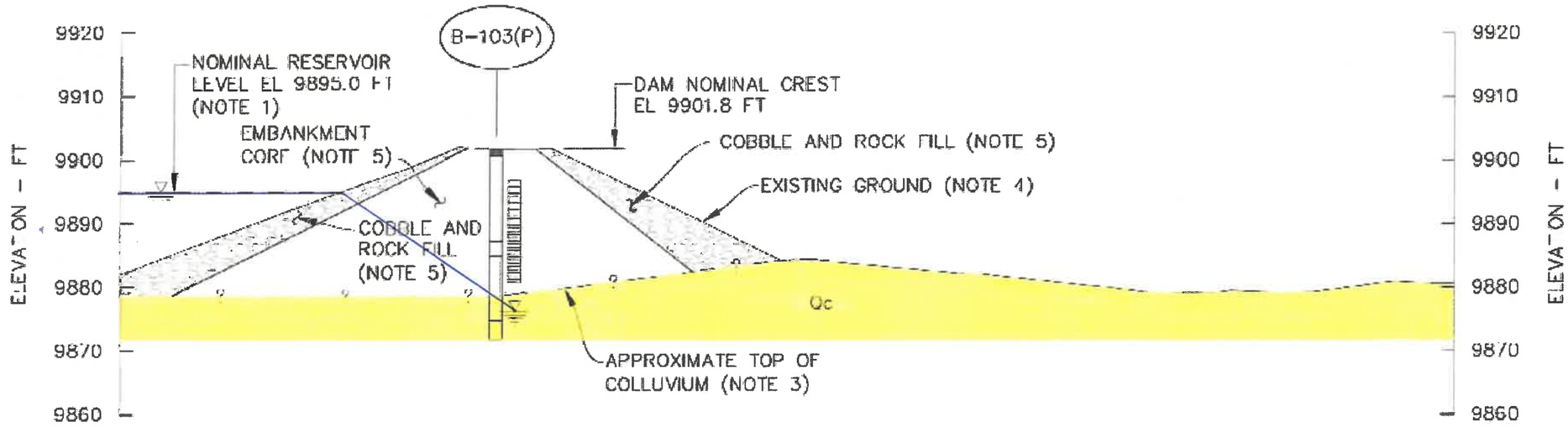
Hydrometer ID:	0805	Flask ID:	1186
Average Mass Offset (g/L):	9.87	Flask Volume (cm^3):	996.8
Hydrometer Bulb Volume (cm^3):	56.50	Flask Surface Area (cm^2):	28.60
Meniscus Correction (g/L):	1.00	Assumed Specific Gravity	2.65
H_b (cm):	24.5	Hydrometer Type:	152H
H_{cb} (cm):	6.8		
H_s (cm):	8.2	Percent Finer by Mass at 2 µm:	**17.3**

Hydrometer Data

Elapsed Time (minutes)	Hydrometer Reading (g/L)	Offset Reading (g/L)	Temperature (°C)	Effective Depth (cm)	Maximum Particle Diameter in Suspension (mm)	Percent Finer by Mass (%)
1	38.00	5.34	23.1	10.62	0.044	44.9
2	37.00	5.34	23.1	10.79	0.032	43.5
4	33.50	5.34	23.1	11.36	0.023	38.7
15	29.00	5.34	23.1	12.10	0.012	32.5
30	26.00	5.34	23.1	12.60	0.009	28.4
60	24.00	5.34	23.1	12.93	0.006	25.7
240	20.00	5.30	23.2	13.58	0.003	20.2
1440	17.00	5.53	22.6	14.08	0.001	15.8

NOTES:

$$2 \times 10^{-6}\ m \times \frac{1000\ mm}{m} = 0.002\ mm$$



SECTION C 1.2

0 10 20 40 60

SCALE IN FEET

NOTES:

1. NOMINAL RESERVOIR LEVEL DURING DRILLING ON 7/23/2018.
2. B–103(P) WELL WAS DRY DURING FINAL GROUNDWATER LEVEL MEASUREMENT FROM WELL ON 9/22/2018. GROUNDWATER LEVEL SHOWN WAS MEASURED DURING DRILLING ON 7/23/2018.
3. ALL GEOLOGICAL CONTACTS ARE APPROXIMATE AND NOT ALL INFORMATION IS SHOWN. BORING LOGS SHOWN GENERALIZE SUBSURFACE CONDITIONS AND ARE NOT INTENDED TO BE A COMPLETE OR ACCURATE DEPICTION OF THE SUBSURFACE CONDITIONS. COMPLETE BORING LOGS ARE PROVIDED IN APPENDIX B.
4. GROUND SURFACE TOPOGRAPHY DATA PROVIDED BY CITY OF GRAND JUNCTION, CO. HORIZONTAL DATUM IS NAD83 AND VERTICAL DATUM IS NAVD88.
5. GEOMETRY OF EMBANKMENT ZONES WAS APPROXIMATED USING DATA FROM DESIGN DRAWINGS. OUTLET WORKS, TOE DRAIN, AND CUTOFF TRENCH NOT SHOWN.

LEGEND

- ROAD BASE
- EMBANKMENT FILL
- Qc COLLUVIUM
- MONITORING WELL SENSING ZONE
- GROUNDWATER LEVEL MEASURED FROM MONITORING WELL ON 7/23/2018
- B–101(P) MONITORING WELL NAME

Grand Junction COLORADO

RJH CONSULTANTS, INC.

HOGCHUTE DAM SAFETY EVALUATION GEOTECHNICAL DATA REPORT	DAM SECTION C NORTH
PROJECT NO. 18115	January 2019 — Figure 4.5

P:\18115 HOGCHUTE DAM SAFETY EVALUATION\CAD\FIGURES\MONITORING WELLS\MONITORING WELLS.DWG 1/22/2019 3:02 PM

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	10-17.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-11
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hygroscopic Moisture of Fines

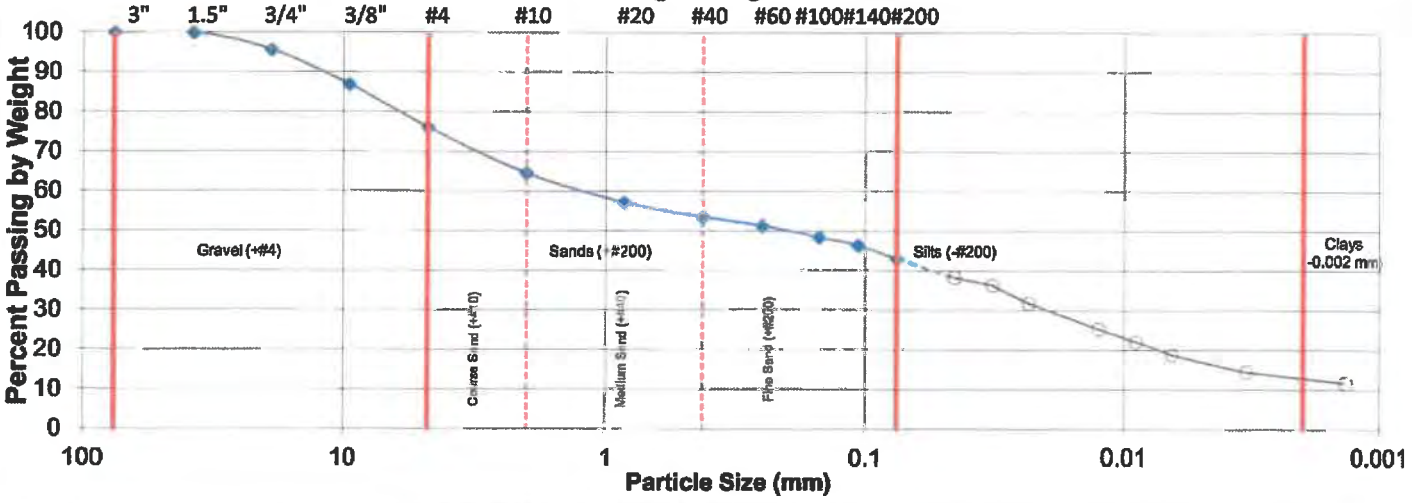
Mass Wet Pan and Soil (g): 121.30
Mass Dry Pan and Soil (g): 119.10
Mass of Pan (g): 3.15
Moisture (%): 1.9

Sample Data

Total Wet Mass of Sample (g): 26704.1
Total Dry Mass of Sample (g): 26381.7
Split Fraction: #10 3/8"
Mass of Sub-Sample Fraction (g): 57.98 1427.90

Sieve Number	Sieve Size (mm)	Mass of Pan and Soil (g)	Mass of Pan (g)	Mass of Individual Retained Soil (g)	Correction Factor	Percent Passing by Weight (%)
3"	76.2	0.0	--	--	--	--
1.5"	38.1	0.0	--	--	--	--
3/4"	19.05	1138.5	--	1138.5	1.00	95.7
3/8"	9.53	2287.5	--	2287.5	1.00	87.0
#4	4.75	175.9	--	175.9	0.87	76.1
#10	2.00	186.6	--	186.55	0.870	64.6
#20	0.850	6.5	--	6.52	0.646	57.2
#40	0.425	3.1	--	3.12	0.646	53.7
#60	0.250	2.0	--	2.00	0.646	51.4
#100	0.150	2.5	--	2.52	0.646	48.6
#140	0.106	1.8	--	1.83	0.646	46.5
#200	0.075	3.1	--	3.05	0.646	43.0

Percent Passing vs Log of Particle Size



USCS Classification ASTM D 2487

Atterberg Classification: CL
Group Symbol: SC
USCS Classification: Clayey Sand With Gravel

Coefficient of Curvature - C_c: --
Coefficient of Uniformity - C_u: --

Data entry by: KMS Date: 11/8/2018
Checked by: Date: 11/8/18
File name: 2679130__Grain Size with Hydrometer ASTM D6913 D7928_1.xlsm

ADVANCED TERRA TESTING

Grain Size Analysis with Hydrometer
ASTM D 6913 And D 7928

CLIENT	RJH Consultants	BORING NO.	B-103
JOB NO.	2679-130	DEPTH	10-17.5'
PROJECT	Hogchute Dam Safety Evaluation	SAMPLE NO.	Bu-11
PROJECT NO.	18115	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	11/07/18		
TECHNICIAN	BDF		

Hydrometer and Flask Parameters

Hydrometer ID:	0805	Flask ID:	1185
Average Mass Offset (g/L):	9.87	Flask Volume (cm^3):	995.9
Hydrometer Bulb Volume (cm^3):	56.50	Flask Surface Area (cm^2):	28.45
Meniscus Correction (g/L):	1.00	Assumed Specific Gravity	2.65
H_b (cm):	24.5	Hydrometer Type:	152H
H_{cb} (cm):	6.8		
H_s (cm):	8.2	Percent Finer by Mass at 2 µm:	**12.6**

Hydrometer Data

Elapsed Time (minutes)	Hydrometer Reading (g/L)	Offset Reading (g/L)	Temperature (°C)	Effective Depth (cm)	Maximum Particle Diameter in Suspension (mm)	Percent Finer by Mass (%)
1	35.00	5.49	22.7	11.11	0.045	**38.3**
2	33.50	5.49	22.7	11.36	0.032	**36.4**
4	30.00	5.49	22.7	11.93	0.024	**31.8**
15	25.00	5.49	22.7	12.76	0.013	**25.3**
30	22.50	5.45	22.8	13.17	0.009	**22.1**
60	20.00	5.45	22.8	13.58	0.006	**18.9**
240	16.50	5.34	23.1	14.16	0.003	**14.5**
1440	14.50	5.53	22.6	14.48	0.001	**11.7**

NOTES:

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North Drain

↳ min slope = 4.78% = 0.0478 ft/ft

↳ estimated discharge = 12 gpm = 0.02676 ft^3/s

↳ perf hole size = 5/32" = 3.9688 mm

Capacity of One Perforation

$$Q = C_d A \sqrt{2gh}$$

$C_d = 0.6$

$$A = \pi \left(\frac{5/32 \text{ in} / 12}{4} \right)^2 = 0.00013316 \text{ ft}^2$$

$g = 32.2 \text{ ft/s}^2$

$h = 1 \text{ in} = 0.08333 \text{ ft}$ (assume 1 ft in head over pipe)

$$Q = 0.6 \times 0.00013316 \times \sqrt{2 \times 32.2 \times 0.08333}$$

$$Q = 1.85 \times 10^{-4} \frac{ft^3}{s} \text{ per hole}$$

$$\text{Total Holes} = \frac{0.02676}{1.85 \times 10^{-4}} = 145 \text{ holes}$$

Effective open area of holes for flow is assumed to be 30% of total perforated area (FEMA 2007)

$$\text{Total Holes (after accounting for blockage)} = \frac{145}{0.3} = 483$$

Project No. 26-1144.00	Remarks	Prepared by ajs	Date 10/29/2020
Project Name Hogchute		Checked by	Date
Title/Item Toe Drain Perforation Design			Sheet 1 of 2

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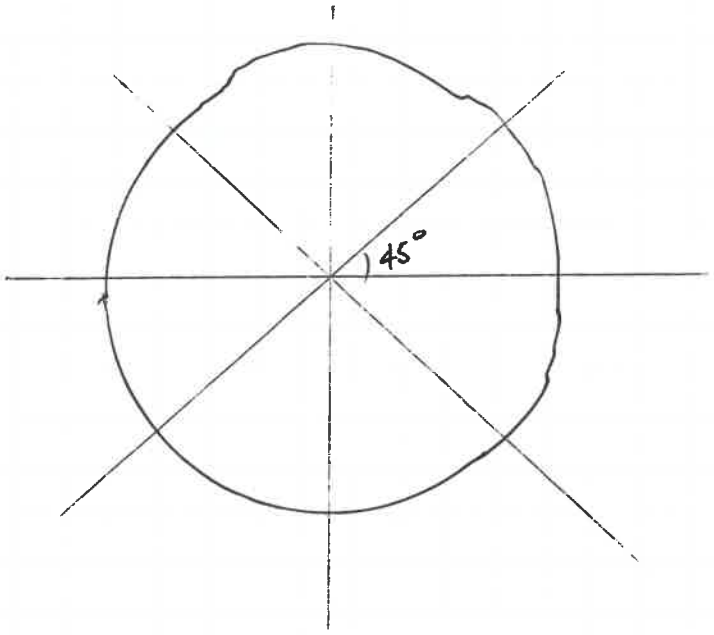
Perforated North Drain Length = 150 ft of pipe

$\frac{500 \text{ holes needed}}{150 \text{ ft pipe}}$ = 3.33 hole per lf



8 holes around pipe circumference every 3 inches

↳ $\left(\frac{12}{3}\right) \times 8$

= 32 holes per lf

FS = $\frac{32}{3.33}$ = 9.6

8 holes every 2 inches

$\left(\frac{12}{2}\right) \times 8$ = 48 holes per lf

FS = $\frac{48}{3.33}$ = 14.4

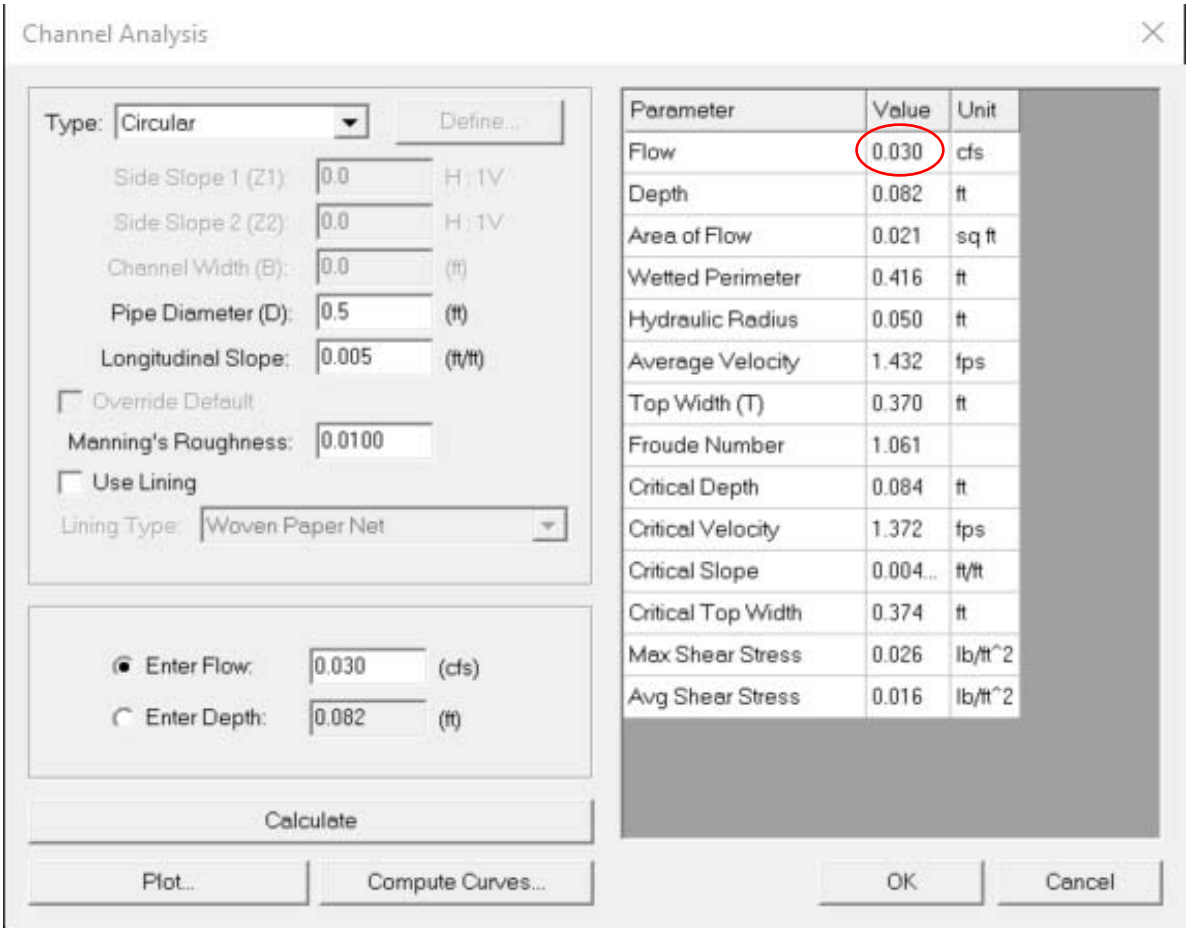
Perforation Requirement:

Eight 5/32-inch holes evenly spaced around pipe diameter every 2 inches wherever perforation is required.

Project No. 26-1144.00	Remarks	Prepared by ajp	Date 10/29/2020
Project Name Hogchute		Checked by	Date
Title/Item Toe Drain Perforation Des.			Sheet 2 of 2

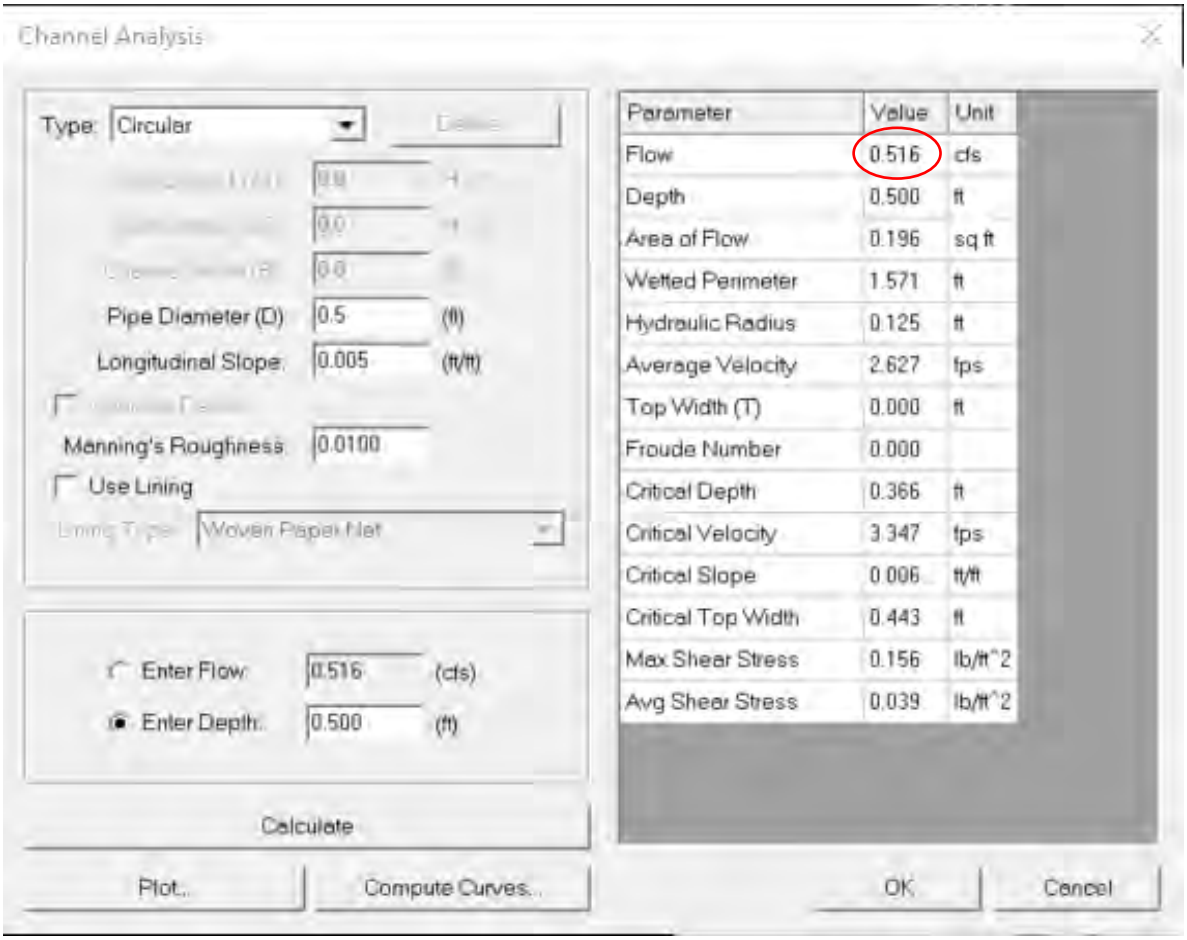
Portion of toe drain pipe with flattest slope (0.50%) with constant flow rate of 12 gpm = 0.03 cfs
Normal depth calculations for 6-inch HDPE pipe (*n* = 0.010)

Computed using Hydraulic Toolbox 4.4



Portion of toe drain pipe with flattest slope (0.50%) flowing full
Normal depth calculations for 6-inch HDPE pipe (*n* = 0.010)

Computed using Hydraulic Toolbox 4.4



Factor of Safety = 0.516 cfs (capacity) / 0.030 cfs (actual) = **17.2**

6" HDPE pipe is sufficient for toe drain pipe

Type I Filter Gradation

Hogchute Dam, Mesa County, CO

References

1) Design Standards No. 13 - Embankment Dams, U.S. Department of the Interior, Bureau of Reclamation, Chapter 5: Protective Filters, November 2011.
2) Grain Size Analysis with Hydrometer (ASTM D6913 and ASTM D7928), ATT, 11/8/2018

Analysis for Type I Filter Material

Embankment gradation: Location B-101, Depth 51'-65'

Design Gradation			
Sieve Size	Opening (mm)	Percent Passing	Regraded Percent Passing
1 in.	25.00	100.0	
3/4 in.	19.00	97.3	
3/8 in.	9.510	88.8	
#4	4.750	80.2	100.0
#10	2.000	73.2	91.3
#40	0.425	65.9	82.2
#100	0.150	60.5	75.4
#200	0.075	54.0	67.3
hydrometer	0.044	44.9	56.0
hydrometer	0.032	43.5	54.2
hydrometer	0.023	38.7	48.3
hydrometer	0.012	32.5	40.5
hydrometer	0.0090	28.4	35.4
hydrometer	0.0060	25.7	32.0
hydrometer	0.0030	20.2	25.2
hydrometer	0.0010	15.8	19.7

Regrade to omit gravels. Only consider particles passing the #4 sieve.

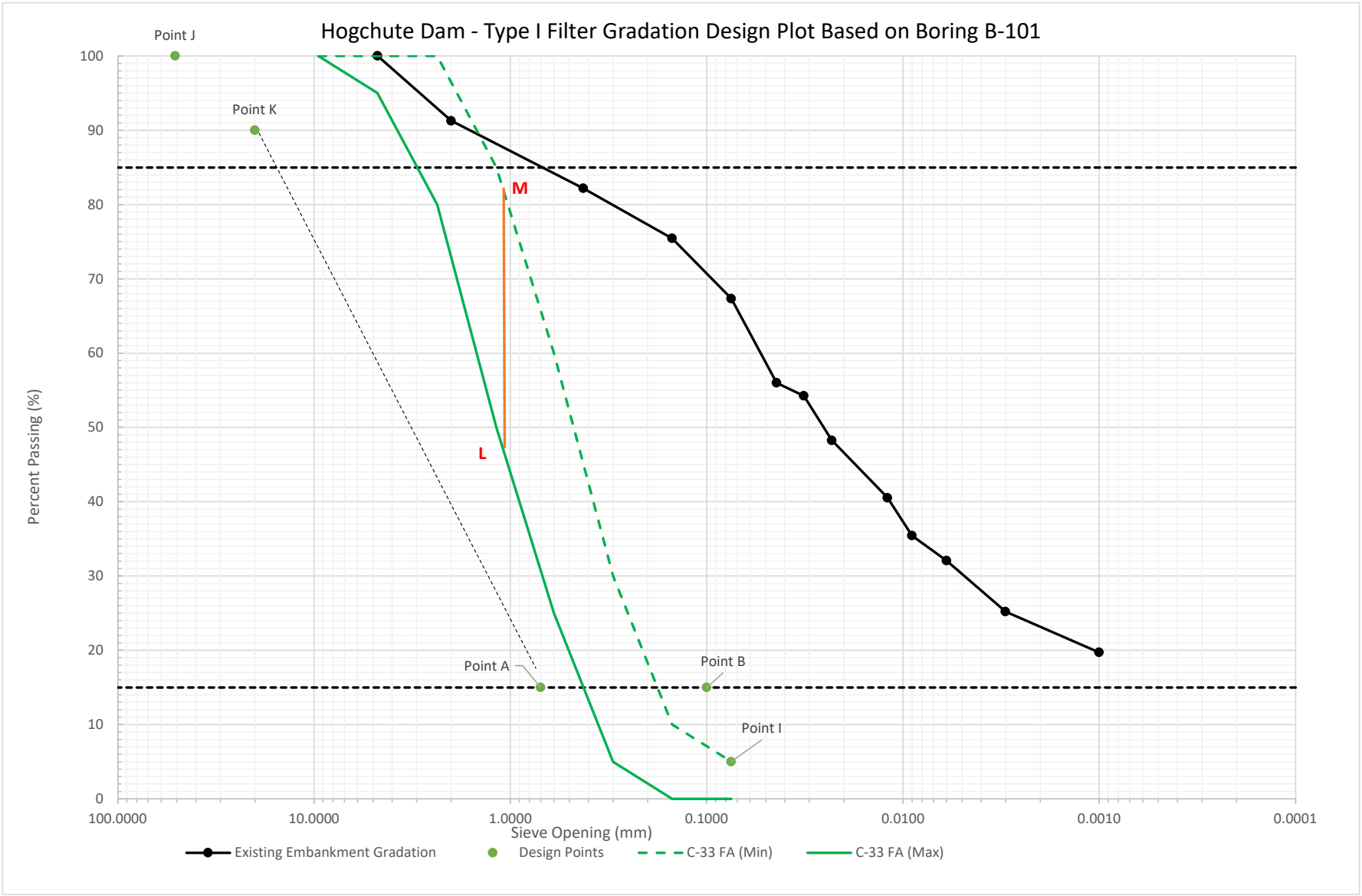
Base Soil Category	Percent Finer than #200 Sieve	(Ref. 1, Table 5.4.3-1)
1	>85	
2	40-85	
3	15-39	
4	<15	

--> Complete following based on above information:			
Finest Sample Percent Passing #200:	67.3	%	
Finest Sample D_{15}:	0.0004	mm	approximated.
Finest Sample D_{85}:	0.7	mm	
Soil Category:	2		

	Opening (mm)	% Passing			
Point A	0.7	15		(Ref. 1, Table 5.4.4-1)	MAX D15F
Point B	0.1	15		(Ref. 1, Section 5.4.5)	MIN D15F
Point J	51	100		(Ref. 1, Table 5.4.6-1)	same for all
Point I	0.075	5		(Ref. 1, Table 5.4.6-1)	same for all
Point K	20	90	<<-Manually input	(Ref. 1, Table 5.4.6-2)	

Type 1 Filter Gradation			
Sieve Size	Opening (mm)	Modified C-33 FA Percent Passing	
1/2 in.	12.5		
3/8 in.	9.51	100	100
#4	4.75	95	100
#8	2.36	80	100
#16	1.18	50	85
#30	0.60	25	60
#50	0.30	5	30
#100	0.15	0	10
#200	0.075	0	5

C-33 Fine Aggregate is acceptable.


Hogchute Dam - Type I Filter Gradation Design Plot Based on Boring B-101
Point J
Point K
M
L
Point A
Point B
Point I
100
90
80
70
60
50
40
30
20
10
0
Percent Passing (%)
100.0000
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Sieve Opening (mm)
Existing Embankment Gradation
Design Points
C-33 FA (Min)
C-33 FA (Max)

Type I Filter Gradation

Hogchute Dam, Mesa County, CO

References

1) Design Standards No. 13 - Embankment Dams, U.S. Department of the Interior, Bureau of Reclamation, Chapter 5: Protective Filters, November 2011.
2) Grain Size Analysis with Hydrometer (ASTM D6913 and ASTM D7928), ATT, 11/8/2018

Analysis for Type I Filter Material

Embankment gradation: Location B-103, Depth 10'-17.5'

Design Gradation			
Sieve Size	Opening (mm)	Percent Passing	Regraded Percent Passing
1 in.	25.00	100.0	
3/4 in.	19.00	95.7	
3/8 in.	9.510	87.0	
#4	4.750	76.1	100.0
#10	2.000	64.6	84.9
#40	0.425	53.7	70.6
#100	0.150	48.6	63.9
#200	0.075	43.0	56.5
hydrometer	0.045	38.3	50.3
hydrometer	0.032	36.4	47.8
hydrometer	0.024	31.8	41.8
hydrometer	0.013	25.3	33.2
hydrometer	0.0090	22.1	29.0
hydrometer	0.0060	18.9	24.8
hydrometer	0.0030	14.5	19.1
hydrometer	0.0010	11.7	15.4

Regrade to omit gravels. Only consider particles passing the #4 sieve.

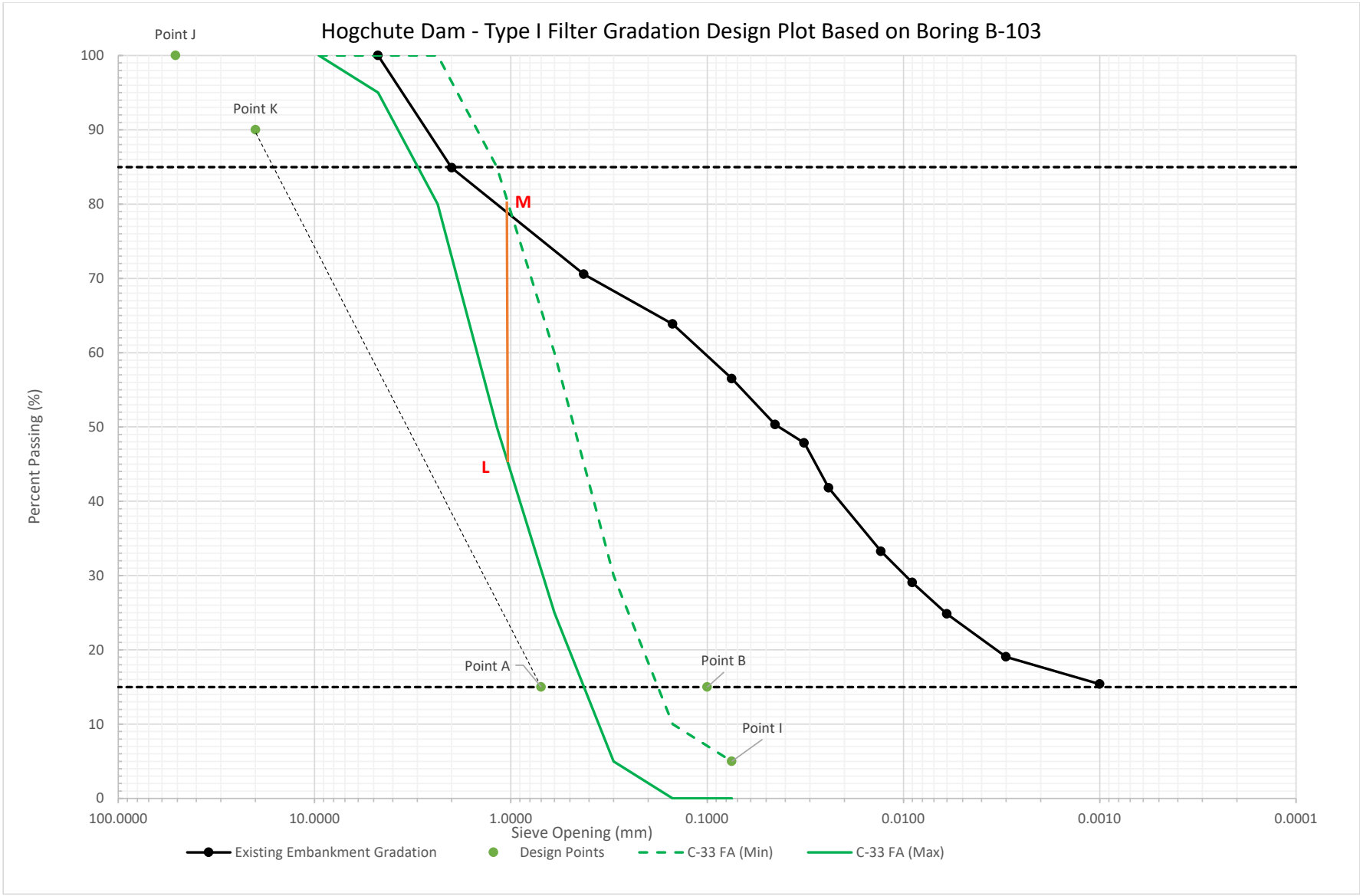
Base Soil Category	Percent Finer than #200 Sieve	(Ref. 1, Table 5.4.3-1)
1	>85	
2	40-85	
3	15-39	
4	<15	

--> Complete following based on above information:			
Finest Sample Percent Passing #200:	56.5	%	
Finest Sample D_{15}:	0.001	mm	approximated.
Finest Sample D_{85}:	2	mm	
Soil Category:	2		

	Opening (mm)	% Passing			
Point A	0.7	15		(Ref. 1, Table 5.4.4-1)	MAX D15F
Point B	0.1	15		(Ref. 1, Section 5.4.5)	MIN D15F
Point J	51	100		(Ref. 1, Table 5.4.6-1)	same for all
Point I	0.075	5		(Ref. 1, Table 5.4.6-1)	same for all
Point K	20	90	<<-Manually input	(Ref. 1, Table 5.4.6-2)	

Type 1 Filter Gradation			
Sieve Size	Opening (mm)	Modified C-33 FA Percent Passing	
1/2 in.	12.5		
3/8 in.	9.51	100	100
#4	4.75	95	100
#8	2.36	80	100
#16	1.18	50	85
#30	0.60	25	60
#50	0.30	5	30
#100	0.15	0	10
#200	0.075	0	5

C-33 Fine Aggregate is acceptable.


Hogchute Dam - Type I Filter Gradation Design Plot Based on Boring B-103
Point J
Point K
M
L
Point A
Point B
Point I
Percent Passing (%)
100
90
80
70
60
50
40
30
20
10
0
100.0000
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Sieve Opening (mm)
Existing Embankment Gradation
Design Points
C-33 FA (Min)
C-33 FA (Max)

Type I Filter Gradation

Hogchute Dam, Mesa County, CO

References

1) Design Standards No. 13 - Embankment Dams, U.S. Department of the Interior, Bureau of Reclamation, Chapter 5: Protective Filters, November 2011.
2) Grain Size Analysis with Hydrometer (ASTM D6913 and ASTM D7928), ATT, 11/8/2018

Analysis for Type II Filter Material

Type I Filter is ASTM C-33 FA

Design Gradation			
Sieve Size	Opening (mm)	Percent Passing	Regraded Percent Passing
3/8 in.	9.51	100.0	
#4	4.75	100.0	
#8	2.36	100.0	100.0
#16	1.18	85.0	85.0
#30	0.60	60.0	60.0
#50	0.30	30.0	30.0
#100	0.15	10.0	10.0
#200	0.075	5.0	5.0

No Regrade.

Base Soil Category — Percent Finer than #200 Sieve — (Ref. 1, Table 5.4.3-1)

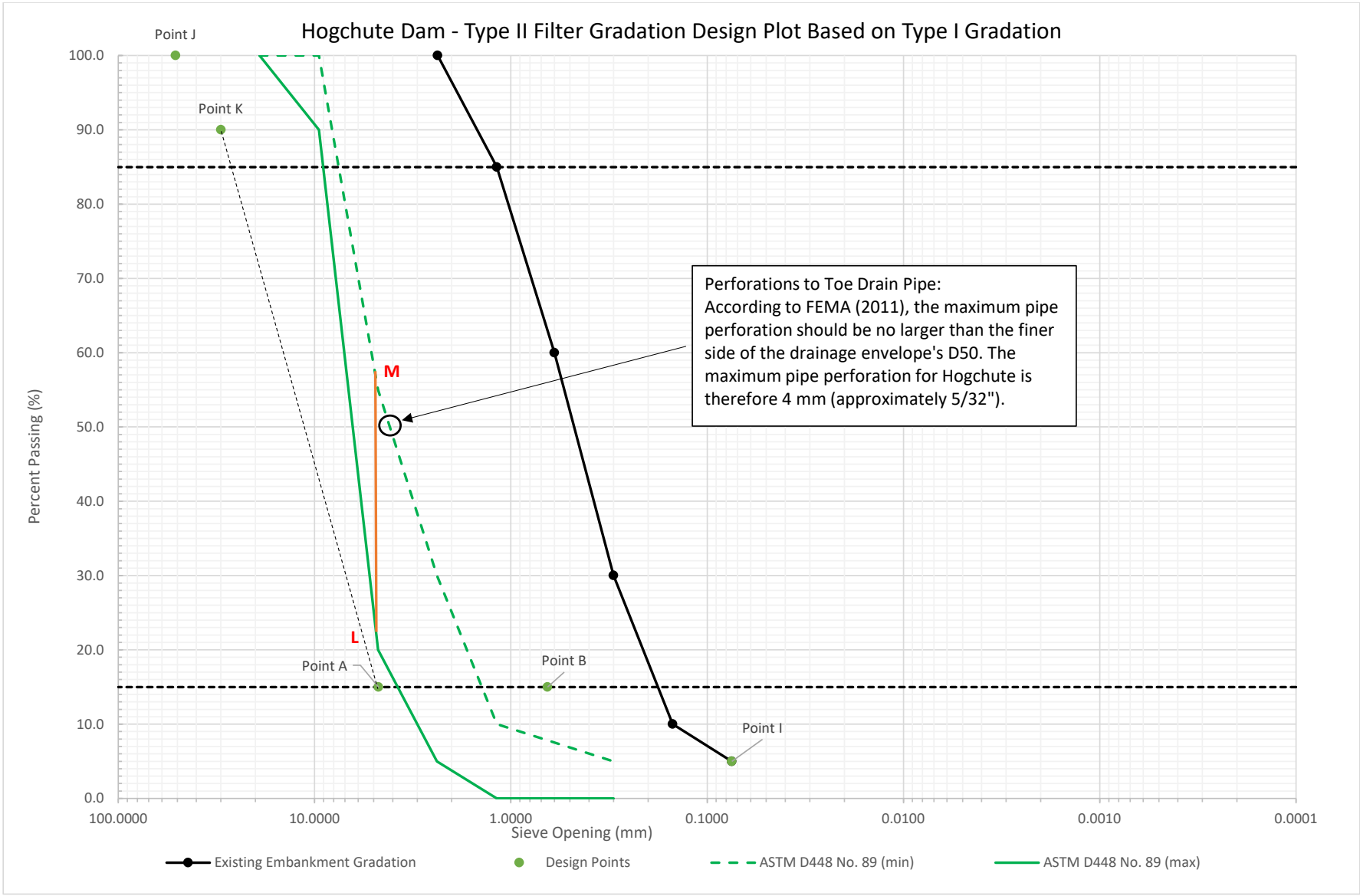
1 >85
2 40-85
3 15-39
4 <15

--> Complete following based on above information:			
Finest Sample Percent Passing #200:	5.0	%	
Finest Sample D_{15}:	0.13	mm	approximated.
Finest Sample D_{85}:	1.18	mm	
Soil Category:	4		

	Opening (mm)	% Passing			
Point A	4.72	15		(Ref. 1, Table 5.4.4-1)	MAX D15F
Point B	0.65	15		(Ref. 1, Section 5.4.5)	MIN D15F
Point J	51	100		(Ref. 1, Table 5.4.6-1)	same for all
Point I	0.075	5		(Ref. 1, Table 5.4.6-1)	same for all
Point K	30	90	<<-Manually input	(Ref. 1, Table 5.4.6-2)	

Type 1 Filter Gradation			
Sieve Size	Opening (mm)	ASTM D448 No. 89	
3 in.	76.2		
2 in.	50.80		
1 in.	25.40		
3/4 in.	19.10	100	100
3/8 in.	9.51	90	100
#4	4.75	20	55
#8	2.38	5	30
#16	1.18	0	10
#50	0.300	0	5

ASTM D448 No. 89 is acceptable.


Hogchute Dam - Type II Filter Gradation Design Plot Based on Type I Gradation
Point J
Point K
Point A
Point B
Point I
M
L
Perforations to Toe Drain Pipe:
According to FEMA (2011), the maximum pipe perforation should be no larger than the finer side of the drainage envelope's D50. The maximum pipe perforation for Hogchute is therefore 4 mm (approximately 5/32").
Percent Passing (%)
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
100.0000
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Sieve Opening (mm)
Existing Embankment Gradation
Design Points
ASTM D448 No. 89 (min)
ASTM D448 No. 89 (max)

Appendix G
Freeboard Analysis

AYRES

Computation of Design Wind Speed for Hogchute Dam Freeboard Analysis

References

1. Historical wind data from Grand Junction Airport. Normals, Means, and Extremes reported in Local Climatological Data annual bulletins for 1979, 1996, and 2019. Printouts included in this appendix.
2. US Department of Agriculture (USDA) (1983). Riprap for Slope Protection Against Wave Action. Technical Release No. 69 (TR-69). February.

From Page 6 of TR-69:

1. Overland Wind Velocity (U_L)

Method 1 - Use of Climatological Data

The local climatological data bulletin lists the wind speed for the fastest mile by months, under the section "Normals, Means, and Extremes." List the 12 speeds given and obtain an average. The average represents the overland wind velocity.

Method 2 - Use of Generalized Data

A value for wind velocity can be obtained from Figure 4, which is a map of isolines showing the maximum wind velocity 30 feet above ground, for a 50-year period of recurrence. The contour lines were developed in part by H.C.S. Thom (13) and in part from U.S. Weather Service records at 141 stations throughout the United States. Judgment must be used with Figure 4, since wind velocity can change sharply in short distances, particularly in mountainous areas. These velocity changes are due to differences in altitude, slope of land, sheltered areas, and other topographical features.

Method 3 - Use of Maximum Speed of Record

The local climatological data bulletin lists the fastest mile of record under the section "Normals, Means, and Extremes." This value can be adjusted for duration and effective fetch in three steps:

a. Obtain the maximum wind velocity (fastest mile of record) for the area being studied.

b. Plot wind velocity (U_L) versus wind duration (T_d) from Figure 5 for the maximum wind velocity obtained in step a. above.

c. As in Figure 6, on the same graph plot wind velocity versus wind duration (T_d) from Figure 2 for the effective fetch (F_e) determined for the site.

The graph plotted should be similar to Figure 6. The value of wind velocity where the two curves intersect is the overland wind velocity for the site.

a) Obtain the maximum wind velocity (fastest mile of record) for the area being studied.

Reviewed Local Climatological Bulletins for the Grand Junction Airport ASOS gauge (period of record 1946-2019).

Identified a period-of-record "fastest mile" (1-min) wind speed of 66 mph, which was recorded in June 1951.

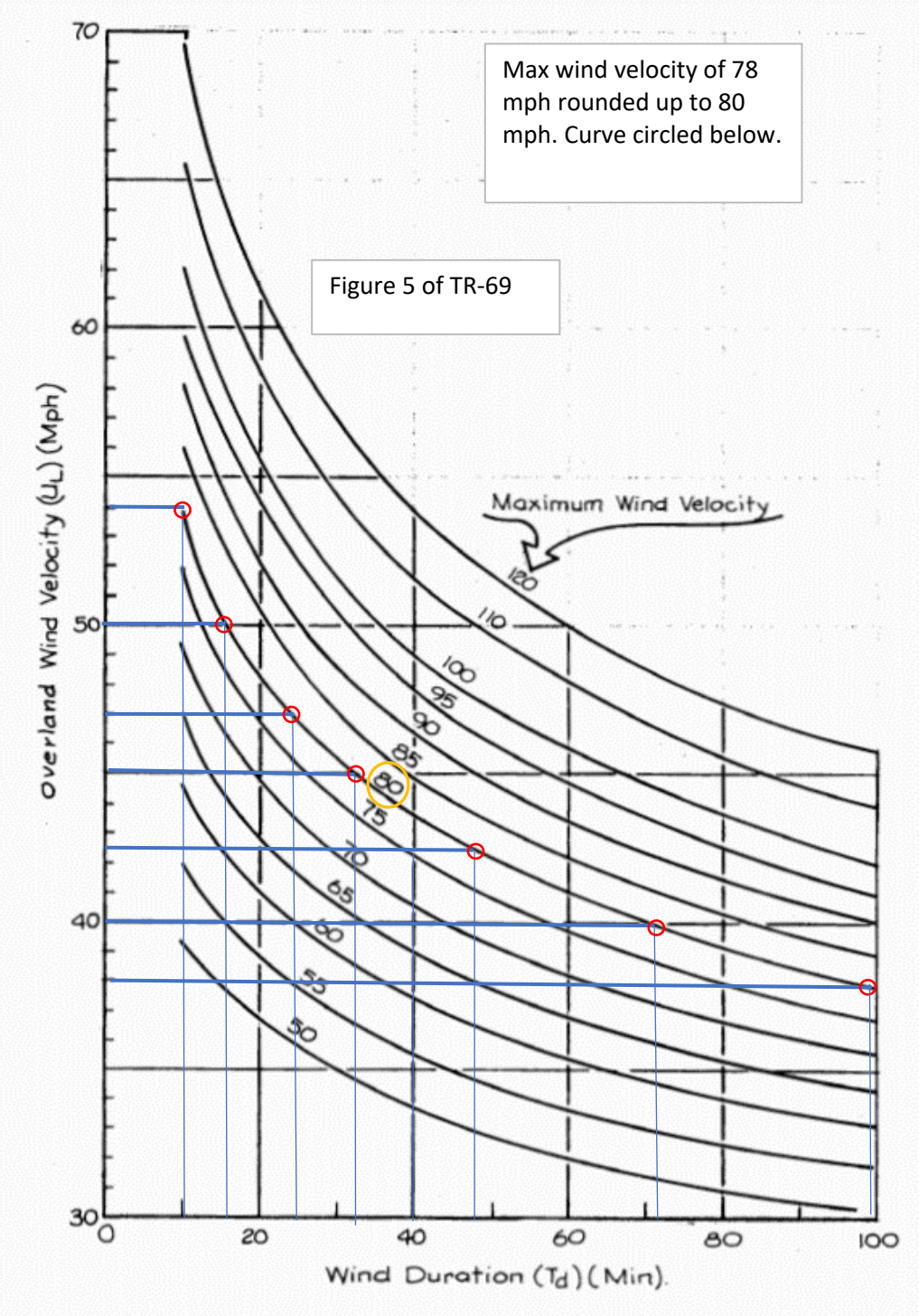
Note that fastest mile wind speeds were only reported by NWS through 1979. Afterward, wind was reported using the following metrics (extremes also shown):

Maximum 2-minute wind speed: 57 mph in June 1996.
Maximum 5-second wind speed: 78 mph in April 1985.
Maximum 3-second wind speed: 73 mph in April 2019.

For conservatism, choose the maximum 5-second wind speed of 78 mph, which was recorded in April 1985. Round up to 80 mph.

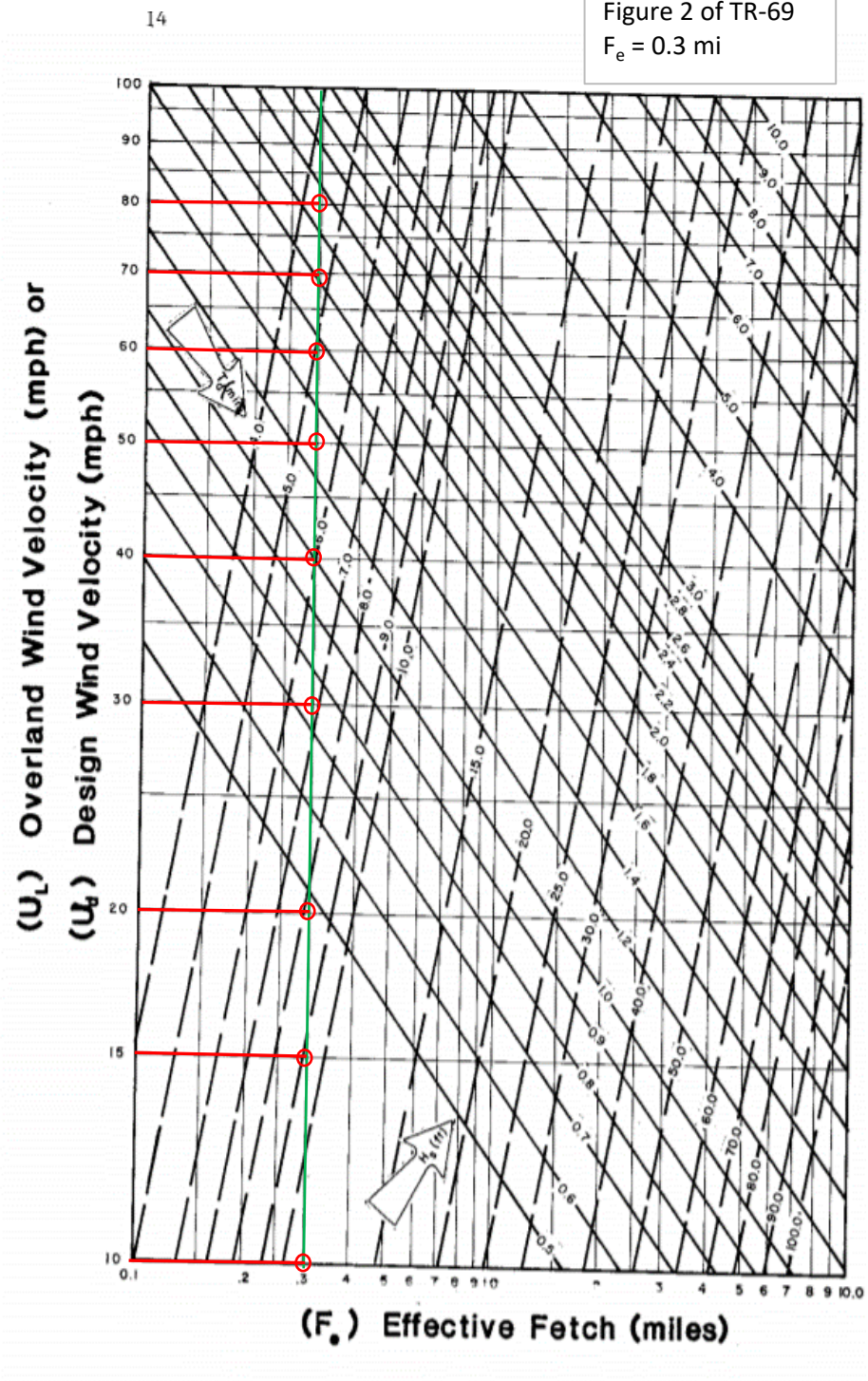
Assume "Fastest-Mile" (1-min) Maximum Wind Velocity = 80 mph

b) Plot wind velocity (U_L) versus wind duration (Td) from Figure 5 for the maximum wind velocity obtained in step a. above.



c) Plot wind velocity vs. wind duration (Td) from Figure 2 for the effective fetch determined for the site

Figure 2 of TR-69
F_e = 0.3 mi



d) On the same graph plot wind velocity versus wind duration (Td) from from Figure 2 and Figure 5 (see previous two pages).

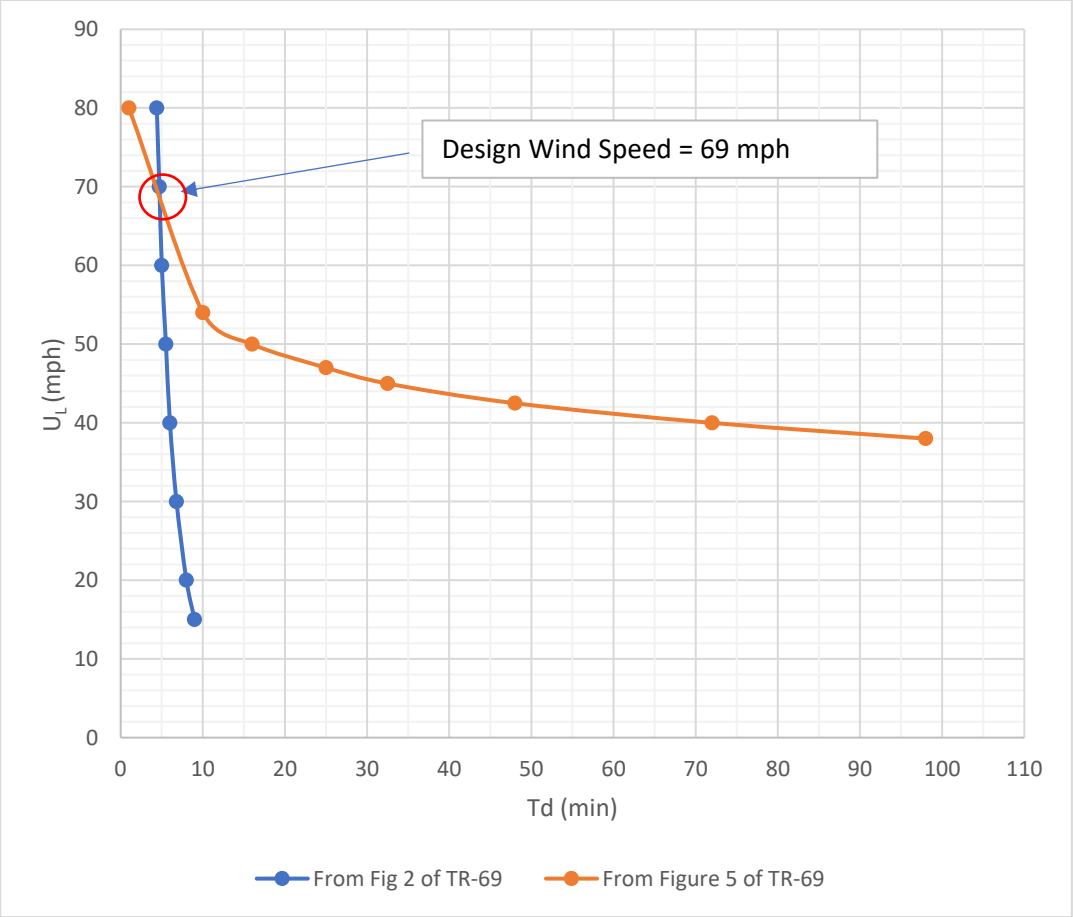
F_e estimated for Hogchute as 0.339 mi. Round down to 0.3 mi. See vertical green line in Figure 2 plot on previous page.

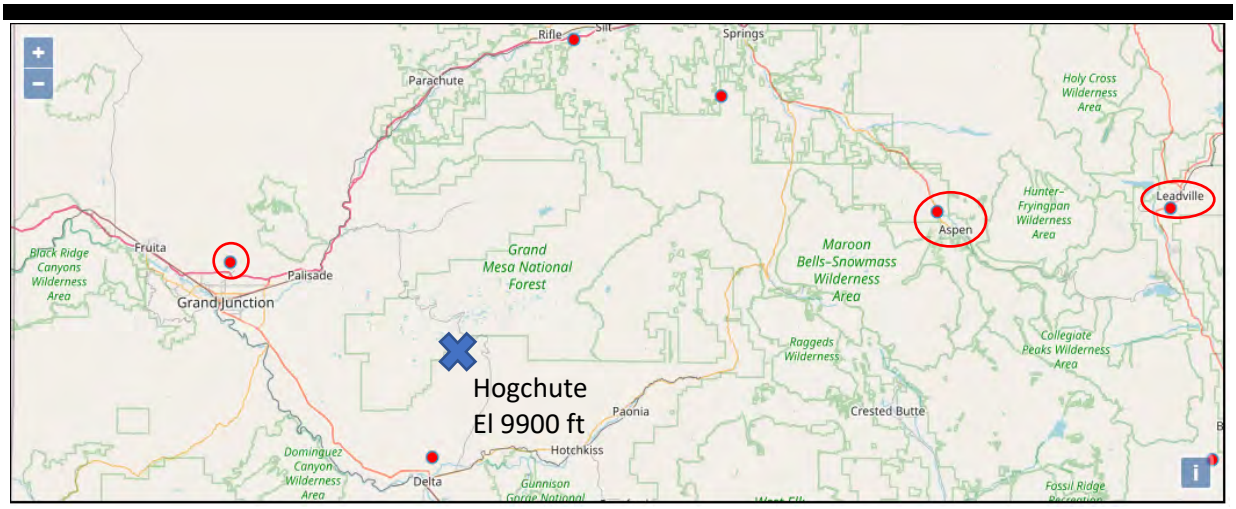
U_L (mph)	Td (min)	
80	1	Obs fastest 1-mile (1-min) wind speed
54	10	
50	16	
47	25	
45	32.5	
42.5	48	
40	72	
38	98	

From Figure 5 of TR-69 for a maximum wind speed of 80 mph

U_L (mph)	Td (min)
15	9
20	8
30	6.8
40	6
50	5.5
60	5
70	4.7
80	4.4

From Figure 2 of TR-69 for a fetch length of 0.3 mi


Design Wind Speed = 69 mph
U_L (mph)
Td (min)
0
10
20
30
40
50
60
70
80
90
100
110
From Fig 2 of TR-69
From Figure 5 of TR-69



	Elev (ft)	Max 5-Second Wind Speed (mph)	Dir (deg)	POR
Aspen Pitkin Co Airport, Sardy Field, CO	7820	62	44.1	98-19
Leadville Lake Co Airport, CO	9934	69.1	36	98-20
Grand Junction Walker Field, CO	4858	72.9	290	96-20

The maximum 5-second wind speeds and directions above were obtained directly from NCEI daily data summaries.

The 80-mph fastest-mile (1-min) wind speed used as the basis for wind-wave calculations at Hogchute is larger than any of the maximum 5-second wind speeds noted above, despite the significantly larger duration.

Grand Junction Airport, EL 4858, reports the largest 5-second wind speed of all three sites above despite being at the lowest elevation.

From these observations, we can conclude that the 80-mph fastest-mile (1-min) wind speed used for Hogchute is appropriate and conservative.

Hogchute Dam
Freeboard Analysis

Ref.: *Design Standards No. 13, Embankment Dams* , Chapter 6: Freeboard, Phase 4 (Final), USBR, September 2012.

Fetch Calculation

$$\text{Fetch} = \frac{\text{sum of radial lengths}}{\text{number of radials}}$$

(Appendix B, B.1) Nine radials at 3-degree intervals were extended from the dam to the opposite shoreline.

Located in Mesa County, CO.
Measurements taken from recent Google Earth Pro Imagery.

	Hogchute Dam	
Central Radial (deg)	0	-23.5
Radial length 1 (mi)	0.251	0.365
Radial length 2 (mi)	0.271	0.400
Radial length 3 (mi)	0.318	0.389
Radial length 4 (mi)	0.342	0.390
Radial length 5 (mi)	0.363	0.392
Radial length 6 (mi)	0.391	0.358
Radial length 7 (mi)	0.373	0.351
Radial length 8 (mi)	0.371	0.348
Radial length 9 (mi)	0.370	0.350
Fetch (mi)	**0.339**	**0.371**

Radial Measurements (feet)	
1327	1927
1432	2114
1681	2055
1805	2061
1914	2071
2062	1889
1969	1852
1959	1837
1952	1847

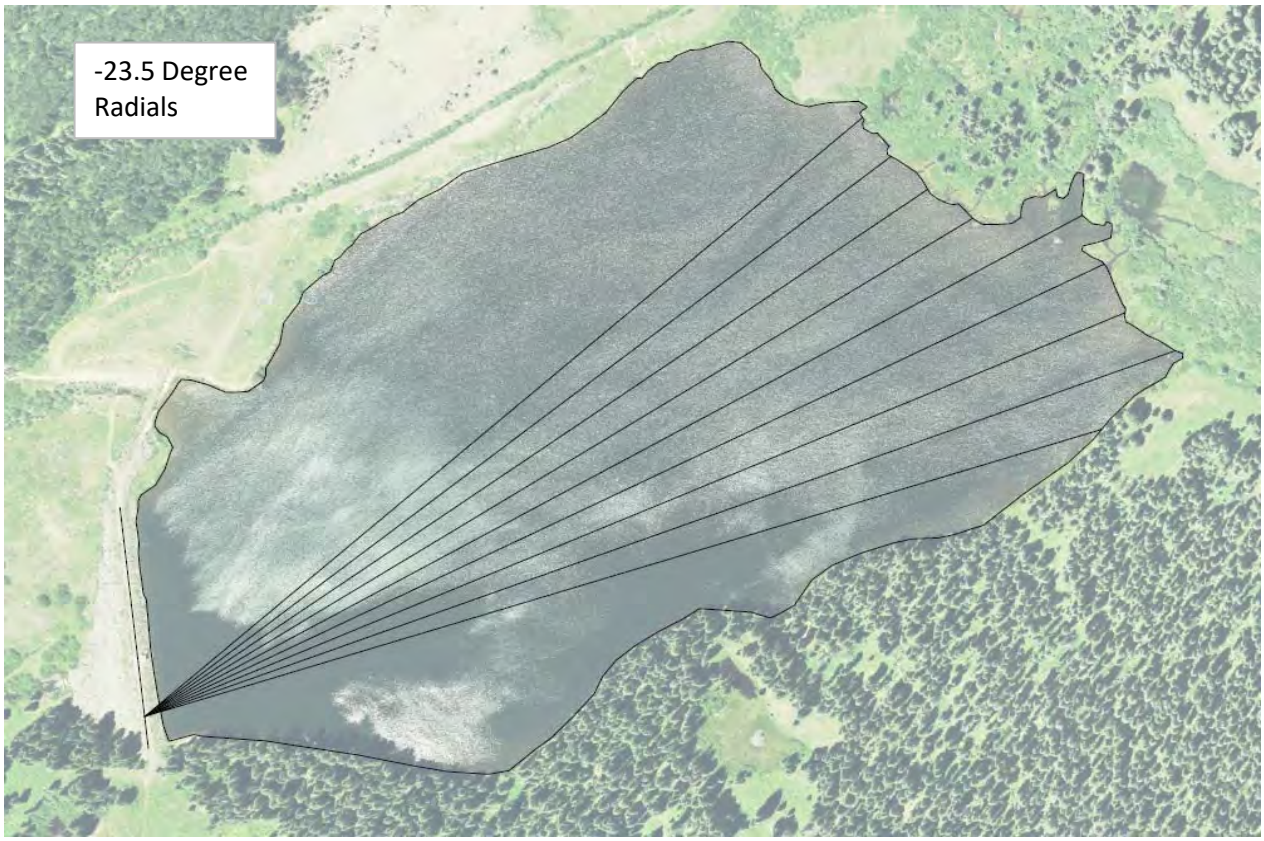
Axis is normal to dam. Positive angle is right of normal axis looking upstream.

Average Bed Elevation

From Hogchute Dam Repairs drawings: 9883.1'

Note: Elevations with respect to the plan datum.

0 Degree
Radials


-23.5 Degree
Radials

AYRES

Hogchute Dam
Freeboard Analysis

Ref.: *Design Standards No. 13, Embankment Dams* , Chapter 6: Freeboard, Phase 4 (Final), USBR, September 2012.

Central radial angle with respect to line normal to dam axis, β:	0	degrees	
Fetch, F:	0.339	miles	
Angle of dam face from horizontal, $\cot\theta_{emb}$:	2.8	horizontal : 1 vertical	
Top of embankment elev.:	9902.00	feet	
Water elevation:	9900.40	feet	[Flood]
Average bed elevation :	9883.1	feet	9883.05
Average water depth, D:	17.35	feet	
Wind velocity (land), V_{land}:	69	mph	

Overwater Wind Correction

R_L = 0.90 (Figure B-2)

$V = V_{land} \times R_L$ = 62.1 mph

Axis is normal to dam. Positive angle is right of normal axis looking upstream.

Reduction Factor for Central Radial Angle Normal to Dam Axis

R_H = 1.00 (Figure B-3)

Minimum Wind Duration (Appendix B.3.6)

- duration needed to reach maximum wave heights (Equ. 1)

$$t_{min} = 1.87 \times \frac{F^{0.67}}{V^{0.34}} = \quad 0.22 \text{ hours}$$

Wave Information (Appendix B.4.1 & B.4.2)

Significant wave height (H_s) is defined as the average of the highest one-third of the waves in a wave field.

Significant Wave (Equ. 2): $H_s = 0.0245 \times F^{1/2} \times (V \times (1.1 + 0.0156V)^{1/2})$ = 1.27 feet

Wave Period (Equ. 4): $T = 0.464 \times F^{1/3} \times [(V)^{1/3} \times (1.1 + 0.0156V)^{1/6}]$ = 1.45 sec

Hogchute Dam
Freeboard Analysis

Wave Length (Equ. 3): $L = 5.12 \times T^2 =$ 10.7 feet

- Equations (4, 6, and 1) are valid when the water is deeper than 1/2 of the wave length.

DEEP WATER WAVES, OK

Reduced significant wave: $H_{s_r} = R_H \times H_s =$ 1.27 feet

Wave Height Statistic (Table B-2)

Maximum wave height ratio: [1.27] for [4] % wave exceedance

Specific wave height H = ratio x H_{s_r} = 1.62 feet

Wave Runup (Appendix B.4.3)

Surf Similarity Factor: $\xi_p = \frac{2.26 \times T \times \cot\theta_{emb}^{-1}}{H^{1/2}} =$ 0.92 feet

OK - valid range

Coefficient (Table B-3): $\gamma_r =$ [0.55] (0.55 for riprap & 0.90 for grass)

Coefficients: $\gamma_b =$ [1.0] and $\gamma_h =$ [1.0]

Coefficient (Figure B-4): $\gamma_\beta = 1.0 - [(1.0 - 0.8)/90 \text{ deg}] \times |\beta| =$ 1.00

Coefficients (Figure B-4): $A = if(\xi_p > 2.5, -0.2, 1.6) =$ 1.6

$C = if(\xi_p > 2.5, 4.5, 0) =$ 0.0

Runup: $R = H(A \times \xi_p + C) \times \gamma_r \times \gamma_b \times \gamma_h \times \gamma_b =$ 1.31 feet

Wind Setup (Appendix B.4.4)

$S = \frac{V^2 F}{1400 D} =$ 0.05 feet

Summary

Total height on embankment = Runup + S = 1.36 feet

Water elev. + Total, ft = 9901.76 < 9902.00 (EL. embankment, ft)

FREEBOARD - OK

AYRES

Hogchute Dam
Freeboard Analysis

Ref.: *Design Standards No. 13, Embankment Dams* , Chapter 6: Freeboard, Phase 4 (Final), USBR, September 2012.

Central radial angle with respect to line normal to dam axis, β:	-23.5	degrees	
Fetch, F:	0.371	miles	
Angle of dam face from horizontal, $\cot\theta_{emb}$:	2.8	horizontal : 1 vertical	
Top of embankment elev.:	9902.00	feet	
Water elevation:	9900.40	feet	[Flood]
Average bed elevation :	9883.1	feet	
Average water depth, D:	17.35	feet	
Wind velocity (land), V_{land}:	69	mph	

Overwater Wind Correction

R_L = 0.90 (Figure B-2)

$V = V_{land} \times R_L$ = 62.1 mph

Axis is normal to dam. Positive angle is right of normal axis looking upstream.

Reduction Factor for Central Radial Angle Normal to Dam Axis

R_H = 0.96 (Figure B-3)

Minimum Wind Duration (Appendix B.3.6)

- duration needed to reach maximum wave heights (Equ. 1)

$$t_{min} = 1.87 \times \frac{F^{0.67}}{V^{0.34}} = 0.24 \text{ hours}$$

Wave Information (Appendix B.4.1 & B.4.2)

Significant wave height (H_s) is defined as the average of the highest one-third of the waves in a wave field.

Significant Wave (Equ. 2): $H_s = 0.0245 \times F^{1/2} \times (V \times (1.1 + 0.0156V)^{1/2})$ = 1.33 feet

Wave Period (Equ. 4): $T = 0.464 \times F^{1/3} \times [(V)^{1/3} \times (1.1 + 0.0156V)^{1/6}]$ = 1.49 sec

AYRES

Hogchute Dam
Freeboard Analysis

Wave Length (Equ. 3): $L = 5.12 \times T^2 =$ 11.4 feet

- Equations (4, 6, and 1) are valid when the water is deeper than 1/2 of the wave length.

DEEP WATER WAVES, OK

Reduced significant wave: $H_{s_r} = R_H \times H_s =$ 1.28 feet

Wave Height Statistic (Table B-2)

Maximum wave height ratio: [1.27] for [4] % wave exceedance

Specific wave height H = ratio x H_{s_r} = 1.63 feet

Wave Runup (Appendix B.4.3)

Surf Similarity Factor: $\xi_p = \frac{2.26 \times T \times \cot\theta_{emb}^{-1}}{H^{1/2}} =$ 0.94 feet

OK - valid range

Coefficient (Table B-3): $\gamma_r =$ [0.55] (0.55 for riprap & 0.90 for grass)

Coefficients: $\gamma_b =$ [1.0] and $\gamma_h =$ [1.0]

Coefficient (Figure B-4): $\gamma_\beta = 1.0 - [(1.0 - 0.8)/90 \text{ deg}] \times |\beta| =$ 0.95

Coefficients (Figure B-4): $A = \text{if}(\xi_p > 2.5, -0.2, 1.6) =$ 1.6

$C = \text{if}(\xi_p > 2.5, 4.5, 0) =$ 0.0

Runup: $R = H(A \times \xi_p + C) \times \gamma_r \times \gamma_b \times \gamma_h \times \gamma_b =$ 1.28 feet

Wind Setup (Appendix B.4.4)

$S = \frac{V^2 F}{1400 D} =$ 0.06 feet

Summary

Total height on embankment = Runup + S = 1.34 feet

Water elev. + Total, ft = 9901.74 < 9902.00 (EL. embankment, ft)

FREEBOARD - OK

Local Climatological Data

Annual Summary With Comparative Data

1979

GRAND JUNCTION, COLORADO

Narrative Climatological Summary

Located in a large mountain valley, the junction of the Colorado and Gunnison rivers, on the west slope of the Rockies, Grand Junction has a climate marked by the wide seasonal range usual to interior localities at this latitude. Thanks, however, to the protective topography of the vicinity, sudden and severe weather changes are very infrequent. Elevation of the valley floor ranges from 4,400 to 4,800 feet above sea level, with mountains on all sides at distances of from 10 to 60 miles, reaching heights of 9,000 to over 12,000 feet.

This mountain valley location, with attendant "valley breezes" provides protection from spring and fall frosts, resulting in a growing season averaging 191 days in the city of Grand Junction. This value varies considerably in the outlying districts, is about the same in the upper valley around Palisade, and 3 to 4 weeks shorter near the river west of Grand Junction, where the "valley breeze" is less effective. Farming areas located on mesas also enjoy longer frost-free seasons than adjacent lower lying ground where cool air tends to collect at night; this effect is more noticeable in the west, or lower portion of the valley. The growing season is sufficiently long to permit growth commercially of almost all fruits except citrus varieties. Summer grazing of cattle and sheep on nearby mountain ranges is extensive; foundation herds are wintered in the valley and there is some winter feeding of fat cattle and sheep.

The interior, continental location, ringed by mountains on all sides, results in quite low precipitation in all seasons. Consequently, agriculture is dependent on irrigation, for which an adequate supply of water has been available from mountain snows and rains. Summer rains occur chiefly as scattered light showers from thunderstorms which develop over nearby mountains. Winter snows are fairly frequent, but mostly light and quickly melt off. Even the infrequent snows of from 4 to 8 inches, which are heavy for this locality, seldom remain on the ground for prolonged periods. Blizzard conditions in the valley are extremely rare.

Temperatures at Grand Junction have ranged from 105° to -23°, but readings of 100° or higher are infrequent, and about one-third of the winters have no readings below zero. Summer days with maximum temperatures in the middle and low 90's and minima in the low 60's are common. Relative humidity is very low during the summer, with values close to such other dry localities as the southern parts of New Mexico and Arizona. Spells of cold winter weather are sometimes prolonged due to cold air becoming trapped in the valley. Winds are usually very light during the coldest weather. Changes in winter are generally gradual, and abrupt changes are much less frequent than in eastern Colorado. "Cold Waves" are rare. Sunny days predominate in all seasons.

Flying weather conditions are generally favorable for operation of light airplanes, with visibilities of 20 miles or more and ceilings of 5,000 feet or higher prevailing approximately 95 percent of the time. Gusty surface winds are rather frequent in the spring and early summer. The prevailing wind is from the east-southeast due to the "valley breeze" effects, but the strongest winds are usually from the south and southwest, and are associated with thunderstorms or with pre-frontal weather.

noaa NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION / ENVIRONMENTAL DATA AND INFORMATION SERVICE / NATIONAL CLIMATIC CENTER ASHEVILLE, N.C.

Meteorological Data For The Current Year

Station: GRAND JUNCTION, COLORADO WALKER FIELD # 23066 — Standard time used: MOUNTAIN — Latitude: 39° 07′ N — Longitude: 108° 32′ W — Elevation (ground): 4843 feet — Year: 1979

Month	Temperature °F: Averages: Daily maximum	Daily minimum	Monthly	Extremes: Highest	Date	Lowest	Date	Degree days Base 65 °F: Heating	Cooling	Precipitation in inches: Water equivalent: Total	Greatest in 24 hrs.	Date	Snow, Ice pellets: Total	Greatest in 24 hrs.	Date	Relative humidity, pct.: Hour 05 (Local time)	Hour 11 (Local time)	Hour 17 (Local time)	Hour 23 (Local time)	Wind: Resultant: Direction	Speed m.p.h.	Average speed m.p.h.	Fastest mile: Speed m.p.h.	Direction	Date	Percent of possible sunshine	Average sky cover, tenths, sunrise to sunset	Number of days: Sunrise to sunset: Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog, visibility ¼ mile or less	Temperature °F: Maximum: (b) 90° and above	32° and below	Minimum: 32° and below	0° and below	Average station pressure mb: Elev. 4839 feet m.s.l.
JAN	25.2	7.9	16.6	40	18	-10	31	1493	0	1.36	0.29	12-13	18.7	4.4	12	80	72	75	83	03	0.7	2.6	24	N	29	35	7.8	6	3	22	15	9	0	3	0	27	31	9	852.7
FEB	33.1	13.8	23.5	43	15	-7	5	1154	0	0.63	0.28	23-24	9.6	4.7	23-24	87	71	75	84	08	0.5	2.5	39	W	14	36	7.0	5	7	16	9	3	0	3	0	11	28	1	853.7
MAR	50.5	31.7	41.1	64	28	18	4	732	0	2.02	0.56	30-31	3.4	3.3	31	80	58	55	76	03	1.8	7.8	26	SW	29	47	6.2	9	4	18	10	1	0	0	0	0	17	0	851.3
APR	64.7	40.2	52.5	81	16	27	3	377	6	0.42	0.31	9-10	1.1	0.9	1	60	35	31	51	14	1.0	8.7	38	SW	17	64	5.6	10	8	12	4	0	0	1	0	0	7	0	849.0
MAY†	72.3	48.2	60.3	87	23	32	11	192	52	1.45	0.65	8	5.0	5.0	8	68	42	37	56	11	1.4	9.8	41	W	6	61	†5.7	10	8	13	7	1	2	0	0	0	1	0	849.6
JUN	86.1	55.9	71.0	100	29	46	19	37	225	0.78	0.46	7-8	0.0	0.0		42	26	20	33	15	3.5	9.8	40	W	30	78	3.4	17	8	5	3	0	2	0	17	0	0	0	852.7
JUL	94.2	63.1	78.7	101	16	55	4	0	428	0.08	0.04	6	0.0	0.0		45	26	19	30	14	3.8	9.7	38	SE	1	76	4.3	15	13	3	3	0	6	0	25	0	0	0	853.0
AUG	88.3	60.8	74.6	102	5	52	20	3	310	0.61	0.27	19-20	0.0	0.0		50	34	27	40	14	3.5	9.3	30	SE	30	66	4.7	15	5	11	7	0	4	0	17	0	0	0	852.4
SEP	86.7	57.3	72.0	95	7	51	16	0	215	0.01	0.01	10	0.0	0.0		34	23	16	25	13	2.7	9.2	30	NE	13	88	2.4	23	4	3	1	0	2	0	11	0	0	0	853.7
OCT	72.3	45.5	58.9	85	2	20	31	209	27	0.25	0.21	20-21	0.0	0.0		46	29	25	37	12	1.9	8.6	34	SW	20	74	4.4	14	9	8	2	0	1	0	0	0	2	0	852.7
NOV	43.4	23.0	33.2	58	4	4	30	945	0	1.02	0.47	19-20	8.2	4.7	19	70	55	53	66	10	1.7	6.9	19	N	9	54	5.0	12	5	13	6	2	0	4	0	7	26	0	855.4
DEC	36.5	17.2	26.9	45	9	6	2	1175	0	0.27	0.13	27-28	3.5	1.7	22-23	80	65	70	81	08	2.5	6.2	17	NE	11	59	6.0	9	8	14	5	2	0	3	0	5	31	0	858.5
YEAR	62.8	38.7	50.8	102	AUG 5	-10	JAN 31	6317	1263	8.90	0.65	MAY 8	49.5	5.0	MAY 8	62	45	42	55	12	1.7	7.6	41	W	MAY 6	63	5.2	145	82	138	72	18	17	14	70	50	143	10	852.9

† DATA CORRECTED AFTER PUBLICATION OF THE MONTHLY ISSUE.

Normals, Means, And Extremes

Month	Temperatures °F: Normal: Daily maximum	Daily minimum	Monthly	Extremes: Record highest	Year	Record lowest	Year	Normal Degree days Base 65 °F: Heating	Cooling	Precipitation in inches: Water equivalent: Normal	Maximum monthly	Year	Minimum monthly	Year	Maximum in 24 hrs.	Year	Snow, Ice pellets: Maximum monthly	Year	Maximum in 24 hrs.	Year	Relative humidity pct.: Hour 05 (Local time)	Hour 11 (Local time)	Hour 17 (Local time)	Hour 23 (Local time)	Wind: Mean speed m.p.h.	Prevailing direction	Fastest mile: Speed m.p.h.	Direction	Year	Pct. of possible sunshine	Mean sky cover, tenths, sunrise to sunset	Mean number of days: Sunrise to sunset: Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog, visibility ¼ mile or less	Temperatures °F: Max.: (b) 90° and above	32° and below	Min.: 32° and below	0° and below	Average station pressure mb: Elev. 4839 feet m.s.l.
(a)				33		33					33		33		33		33		33		16	16	16	16	33	1S	33	33		33	33	33	33	33	33	33	33	16	16	16	16	7
J	36.7	16.5	26.6	60	1971	-23	1963	1190	0	0.64	2.46	1957	T	1961	0.64	1956	33.7	1957	9.1	1957	78	67	64	77	5.4	ESE	54	S	1950	59	6.2	9	7	15	7	3	*	3	0	14	31	5	854.6
F	44.0	23.2	33.6	66	1976	-14	1951	879	0	0.61	1.56	1948	T	1972	0.53	1960	18.4	1948	8.6	1948	70	52	46	65	6.6	ESE	56	W	1967	64	6.2	8	6	14	6	1	*	2	0	2	26	1	853.8
M	52.8	29.6	41.2	81	1971	5	1948	738	0	0.75	2.02	1979	0.02	1972	0.78	1949	14.9	1948	6.1	1948	60	40	34	53	8.3	ESE	65	S	1954	64	6.0	9	8	14	7	1	1	1	0	*	19	0	848.9
A	64.6	38.8	51.7	85	1969	11	1975	404	0	0.79	1.95	1965	0.06	1958	1.33	1965	14.3	1975	8.9	1975	54	34	27	45	9.6	ESE	59	W	1955	67	5.9	8	10	12	6	*	2	*	0	0	7	0	849.0
M	75.8	48.5	62.2	95	1956	26	1970	133	47	0.63	1.79	1957	T	1970	0.74	1973	5.0	1979	5.0	1979	50	29	23	39	9.6	ESE	65	NW	1966	71	5.4	10	11	10	6	*	*	*	2	0	1	0	849.3
J	85.9	56.6	71.3	103	1971	34	1976	20	209	0.55	2.07	1969	T	1961	1.57	1969	0.0		0.0		44	25	19	33	9.8	ESE	66	S	1951	79	3.9	15	10	5	4	0	5	0	13	0	0	0	851.2
J	93.1	64.2	78.7	105	1976	48	1968	0	425	0.46	1.53	1974	0.03	1972	1.42	1974	0.0		0.0		47	28	21	35	9.3	ESE	56	SE	1955	78	4.2	14	12	5	5	0	8	0	26	0	0	0	853.0
A	89.1	61.6	75.4	103	1969	43	1968	0	322	1.05	3.48	1957	0.04	1956	1.21	1953	0.0		0.0		49	31	22	38	9.0	ESE	56	W	1957	75	4.3	14	11	6	7	0	7	0	20	0	0	0	853.0
S	81.3	53.0	67.2	98	1977	29	1978	60	126	0.84	2.52	1965	T	1953	1.35	1965	3.1	1965	3.1	1965	50	32	25	40	9.0	ESE	61	S	1947	79	3.5	17	8	5	5	*	5	0	5	0	*	0	853.4
O	67.9	41.9	54.9	88	1963	18	1975	324	11	0.93	3.45	1972	0.00	1952	1.24	1957	6.1	1975	6.1	1975	55	37	32	48	7.9	ESE	61	NW	1954	74	4.1	15	8	8	5	*	1	*	0	0	4	0	854.2
N	50.9	28.6	39.8	75	1977	-2	1976	756	0	0.61	1.69	1954	0.04	1976	0.82	1974	12.1	1964	8.4	1954	68	51	46	64	6.6	ESE	54	NW	1954	63	5.3	11	8	11	5	1	*	1	0	1	21	*	854.1
D	39.4	19.6	29.5	64	1973	-12	1978	1101	0	0.55	1.89	1951	0.01	1976	1.16	1951	16.7	1967	6.0	1967	76	62	61	74	5.8	ESE	48	NW	1951	60	5.8	10	7	14	6	2	*	2	0	9	30	2	855.2
YR	65.1	40.2	52.7	105	JUL 1976	-23	JAN 1963	5605	1140	8.41	3.48	AUG 1957	0.00	OCT 1952	1.57	JUN 1969	33.7	JAN 1957	9.1	JAN 1957	59	41	35	51	8.1	ESE	66	S	JUN 1951	70	5.1	140	106	119	70	9	33	8	66	26	138	7	852.5

Means and extremes above are from existing and comparable exposures. Annual extremes have been exceeded at other sites in the locality as follows: Maximum monthly precipitation 3.78 in September 1896; maximum precipitation in 24 hours 2.50 in October 1908; maximum snowfall in 24 hours 17.0 in November 1919.

POR
1946-1979

(a) Length of record, years, through the current year unless otherwise noted, based on January data.
(b) 70° and above at Alaskan stations.
* Less than one half.
T Trace.

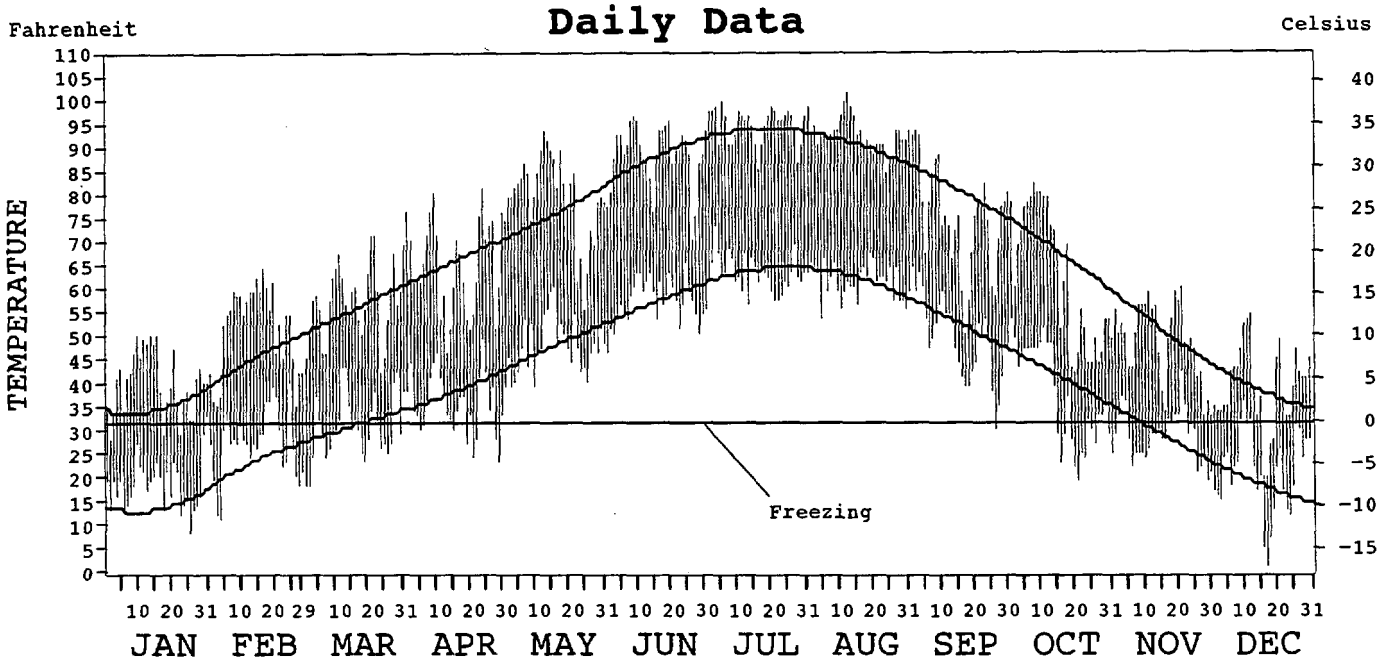
NORMALS - Based on record for the 1941-1970 period.
DATE OF AN EXTREME - The most recent in cases of multiple occurrence.
PREVAILING WIND DIRECTION - Record through 1963.
WIND DIRECTION - Numerals indicate tens of degrees clockwise from true north. 00 indicates calm.
FASTEST MILE WIND - Speed is fastest observed 1-minute value when the direction is in tens of degrees.

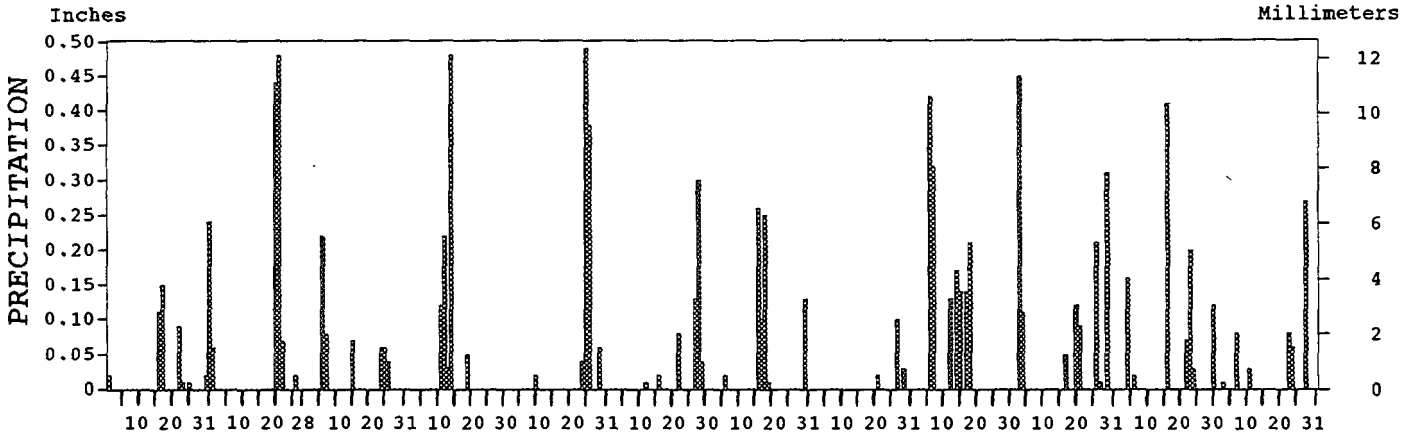
1996 LOCAL CLIMATOLOGICAL DATA
ANNUAL SUMMARY WITH COMPARATIVE DATA

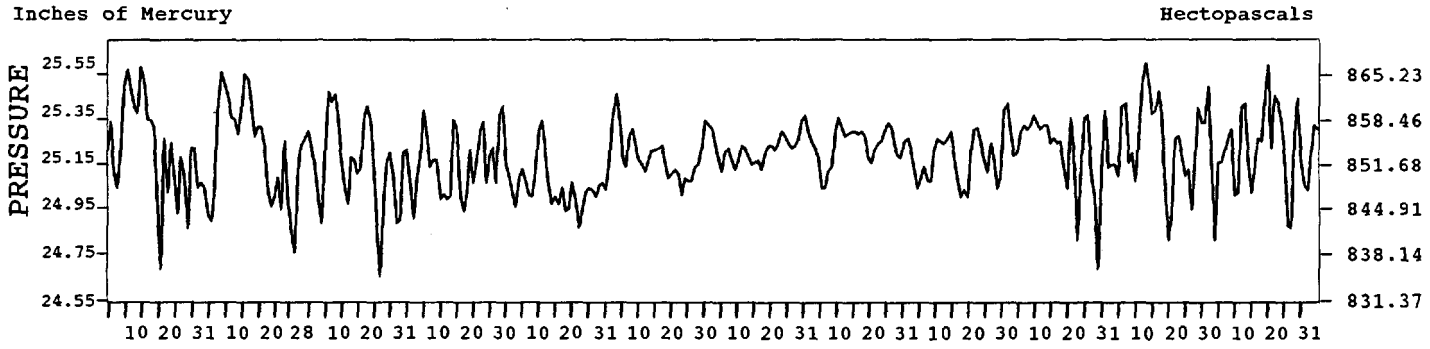
ISSN 0198-766

GRAND JUNCTION, COLORADO (GJT)

Daily Data







I CERTIFY THAT THIS IS AN OFFICIAL PUBLICATION OF THE NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION, AND IS COMPILED FROM RECORDS ON FILE AT THE NATIONAL CLIMATIC DATA CENTER.

Kenneth D. Hinkelman

ACTING DIRECTOR
NATIONAL CLIMATIC DATA CENTER

NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NATIONAL ENVIRONMENTAL SATELLITE, DATA, AND INFORMATION SERVICE
NATIONAL CLIMATIC DATA CENTER
ASHEVILLE, NORTH CAROLINA

NORMALS, MEANS, AND EXTREMES FOR 1996

GRAND JUNCTION, CO (GJT)

LATITUDE: 39° 08′ 03″ N LONGITUDE: 108° 32′ 15″ W ELEVATION (FT): GRND: 4824 BARO: 4828 TIME ZONE: MOUNTAIN (UTC+ 7) WBAN: 23066

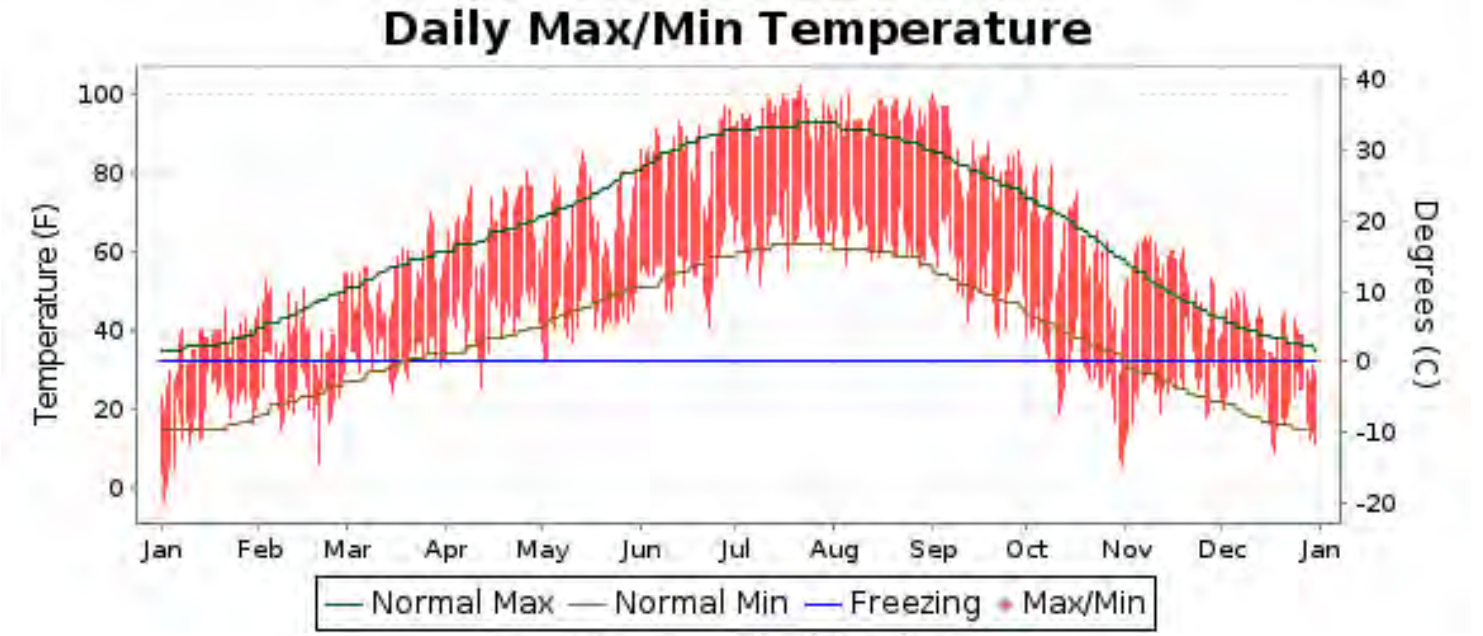
	ELEMENT	POR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
TEMPERATURE °F	NORMAL DAILY MAXIMUM	30	35.5	45.4	55.6	65.8	76.0	87.7	93.6	90.5	81.1	67.7	51.4	38.7	65.8
	MEAN DAILY MAXIMUM	49	36.1	44.6	54.5	65.2	75.7	87.3	93.0	89.9	81.1	67.7	50.6	39.0	65.4
	HIGHEST DAILY MAXIMUM	50	60	68	81	89	95	105	105	103	100	88	75	64	105
	YEAR OF OCCURRENCE		1971	1986	1971	1992	1956	1990	1976	1995	1995	1963	1977	1980	JUN 1990
	MEAN OF EXTREME MAXS.	49	49.0	58.3	70.5	80.5	89.2	98.2	100.6	98.2	92.8	81.4	65.0	52.4	78.0
	NORMAL DAILY MINIMUM	30	14.5	23.5	31.3	38.5	47.9	57.1	63.9	62.2	52.8	41.6	29.4	18.7	40.1
	MEAN DAILY MINIMUM	49	15.9	23.1	30.9	38.7	48.1	57.2	63.9	61.8	52.9	41.3	28.5	19.1	40.1
	LOWEST DAILY MINIMUM	50	-23	-18	5	11	26	34	44	43	29	18	-2	-17	-23
	YEAR OF OCCURRENCE		1963	1989	1948	1975	1970	1976	1993	1968	1978	1993	1976	1990	JAN 1963
	MEAN OF EXTREME MINS.	49	.5	7.3	17.7	25.7	35.0	44.5	55.6	53.0	40.3	28.1	15.2	5.4	27.4
	NORMAL DRY BULB	30	25.0	34.5	43.4	52.2	62.0	72.4	78.8	76.4	67.0	54.7	40.4	28.7	53.0
	MEAN DRY BULB	49	26.0	33.8	42.7	52.0	61.8	72.3	78.4	75.8	67.0	54.5	39.6	29.0	52.7
	MEAN WET BULB	13	22.6	29.6	36.3	41.6	48.1	53.5	58.3	58.1	51.2	42.7	32.8	25.3	41.7
	MEAN DEW POINT	13	16.7	22.3	25.0	28.0	33.1	35.7	43.2	44.7	37.4	30.4	24.3	18.8	30.0
	NORMAL NO. DAYS WITH:														
	MAXIMUM ≥ 90°	30	0.0	0.0	0.0	0.0	1.2	14.3	24.8	19.4	4.2	0.0	0.0	0.0	63.9
	MAXIMUM ≤ 32°	30	11.7	2.3	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	7.2	22.0
	MINIMUM ≤ 32°	30	30.3	24.7	17.1	6.6	0.4	0.0	0.0	0.0	0.1	2.9	20.2	29.5	131.8
	MINIMUM ≤ 0°	30	4.0	0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	*	1.4	6.0
H/C	NORMAL HEATING DEG. DAYS	30	1240	854	670	389	132	13	0	0	55	332	738	1125	5548
	NORMAL COOLING DEG. DAYS	30	0	0	0	0	39	235	428	353	115	13	0	0	1183
RH	NORMAL (PERCENT)	30	70	60	50	40	36	29	34	37	39	46	58	68	47
	HOUR 05 LST	30	77	71	63	55	51	44	48	51	52	58	70	76	60
	HOUR 11 LST	30	64	53	42	33	30	24	28	32	34	38	50	61	41
	HOUR 17 LST	30	61	47	36	27	24	19	22	24	26	33	46	59	35
	HOUR 23 LST	30	75	66	56	45	40	32	36	39	42	51	64	73	52
S	PERCENT POSSIBLE SUNSHINE	50	61	65	65	70	73	81	79	77	79	74	63	61	71
W/O	MEAN NO. DAYS WITH:														
	HEAVY FOG(VISBY ≤ 1/4 MI)	51	2.7	1.8	0.6	0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.8	2.0	8.1
	THUNDERSTORMS	51	0.1	0.3	0.8	2.1	4.8	4.8	7.5	7.9	5.2	1.5	0.4	0.1	35.5
CLOUDINESS	MEAN:														
	SUNRISE-SUNSET (OKTAS)	50	4.9	5.0	5.0	4.8	4.4	3.2	3.3	3.4	2.9	3.4	4.2	4.5	4.1
	MIDNIGHT-MIDNIGHT (OKTAS)	32	4.6	4.4	4.6	4.3	4.0	3.0	3.2	3.3	2.7	3.2	4.0	4.3	3.8
	MEAN NO. DAYS WITH:														
	CLEAR	50	9.1	7.6	8.0	7.9	9.6	14.9	13.7	13.1	16.0	14.7	10.4	9.6	134.6
		0	7.0	7.4	8.6	9.4	10.7	9.4	11.2	11.2	8.1	7.6	7.4	7.6	105.6
		0	14.8	13.3	14.4	12.7	10.7	5.7	5.5	6.1	5.3	8.0	11.6	13.1	121.2
PR	ME	4	25.20	25.20	25.10	25.10	25.10	25.10	25.20	25.20	25.20	25.20	25.20	25.30	25.17
	MEAN SEA-LEVEL PRES. (IN.)	13	30.26	30.12	29.96	29.89	29.83	29.82	29.87	29.91	29.95	30.03	30.11	30.22	30.00
WINDS	MEAN SPEED (MPH)	50	5.8	6.7	8.4	9.4	9.6	9.8	9.6	9.3	9.2	8.1	6.9	6.1	8.2
	PREVAIL.DIR(TENS OF DEGS.)	0	00	00	00	00	00	00	00	00	00	00	00	00	00
	MAXIMUM 2-MINUTE:														
	SPEED (MPH)	17	35	38	37	46	46	57	40	37	49	41	39	35	57
	DIR. (TENS OF DEGS.)		18	24	24	26	20	30	08	25	28	22	31	26	30
	YEAR OF OCCURRENCE		1982	1995	1994	1995	1989	1996	1988	1988	1996	1996	1982	1994	JUN 1996
	MAXIMUM 5-SECOND:														
	SPEED (MPH)	13	41	59	53	78	62	66	64	74	64	57	49	45	78
	DIR. (TENS OF DEGS.)		00	00	00	00	00	18	00	00	00	210	00	00	00
	YEAR OF OCCURRENCE		1993	1995	1994	1985	1989	1991	1993	1984	1992	1996	1984	1994	APR 1985
PRECIPITATION	NORMAL TOTAL (IN.)	30	0.56	0.48	0.90	0.75	0.87	0.50	0.65	0.81	0.82	0.98	0.71	0.61	8.64
	MAXIMUM TOTAL (IN.)	50	2.46	1.56	2.02	1.95	2.04	2.07	1.92	3.48	2.81	3.45	2.00	1.89	3.48
	YEAR OF OCCURRENCE		1957	1948	1979	1965	1995	1969	1983	1957	1982	1972	1983	1951	AUG 1957
	MINIMUM TOTAL (IN.)	50	T	T	0.02	0.06	T	T	0.01	0.04		0.00		0.01	0.00
	YEAR OF OCCURRENCE		1961	1972	1972	1958	1970	1980	1994	1956		1952		1976	NOV
	MAXIMUM IN 24 HOURS (IN.)	50	0.71	0.81	1.15	1.33	1.13	1.57	1.42	1.21	1.35	1.24	0.83	1.16	1.57
	YEAR OF OCCURRENCE		1989	1996	1993	1965	1983	1969	1974	1953	1965	1957	1983	1951	JUN 1969
	NORMAL NO. DAYS WITH:														
	PRECIPITATION ≥ 0.01	30	6.4	5.2	7.7	6.6	6.3	4.2	5.8	6.1	6.3	5.6	5.6	6.4	72.2
	PRECIPITATION ≥ 1.00	30	0.0	0.0	0.0	0.0	0.0	*	*	0.0	*	0.0	0.0	0.0	0.0
SNOWFALL	NORMAL TOTAL (IN.)	30	6.6	3.3	3.7	1.2	0.2	0.0	0.0	0.0	0.1	0.7	2.5	5.7	24.0
	MAXIMUM TOTAL (IN.)	50	33.7	18.4	14.9	14.3	5.0	0.0	0.0	T	3.1	6.1	12.1	19.0	33.7
	YEAR OF OCCURRENCE		1957	1948	1948	1975	1979			1993	1965	1975	1964	1983	JAN 1957
	MAXIMUM IN 24 HOURS (IN.)	50	9.1	9.0	6.1	8.9	5.0	0.0	0.0	T	3.1	6.1	8.4	6.0	9.1
	YEAR OF OCCURRENCE		1957	1989	1948	1975	1979			1993	1965	1975	1954	1967	JAN 1957
	MAXIMUM SNOW DEPTH (IN.)	48	16	12	8	7	1	0	0	0	2	5	8	11	16
	YEAR OF OCCURRENCE		1957	1957	1960	1975	1979				1965	1975	1954	1983	JAN 1957
	NORMAL NO. DAYS WITH:														
	SNOWFALL ≥ 1.0	30	2.5	1.0	1.2	0.3	0.1	0.0	0.0	0.0	0.*	0.2	0.8	2.1	8.2

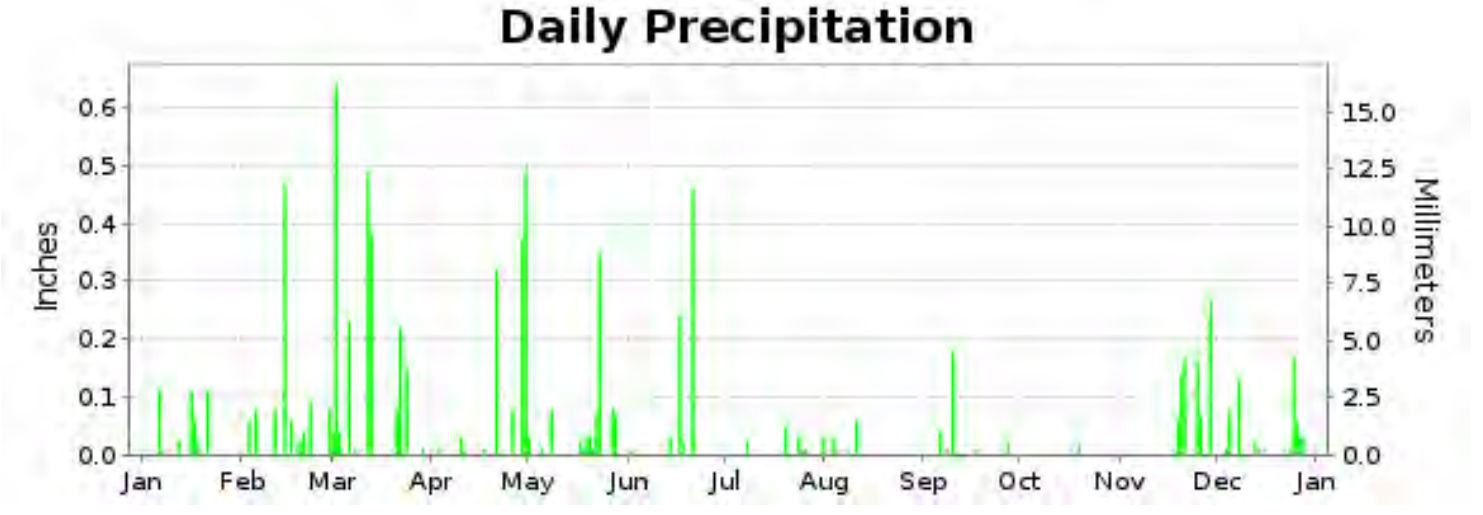
POR 1980-1996

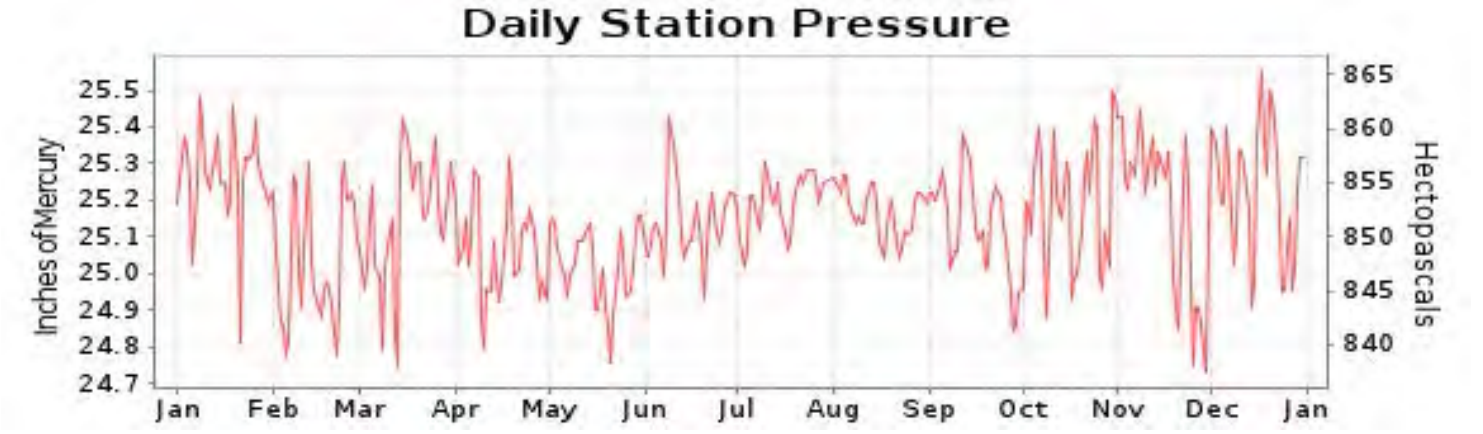
2019
LOCAL CLIMATOLOGICAL DATA
ANNUAL SUMMARY WITH COMPARATIVE DATA

ISSN 0198-7666

GRAND JUNCTION, COLORADO (KGJT)







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NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION	NATIONAL ENVIRONMENTAL SATELLITE, DATA AND INFORMATION SERVICE	NATIONAL CENTERS for ENVIRONMENTAL INFORMATION (NCEI) ASHEVILLE, NORTH CAROLINA	DIRECTOR NCEI

NORMALS, MEANS, AND EXTREMES
GRAND JUNCTION (KGJT)

LATITUDE: 39° 8'N **LONGITUDE:** 108° 32'W **ELEVATION (FT):** GRND: 4858 BARO: 4826 **TIME ZONE:** MOUNTAIN (UTC -7) **WBAN:** 23066

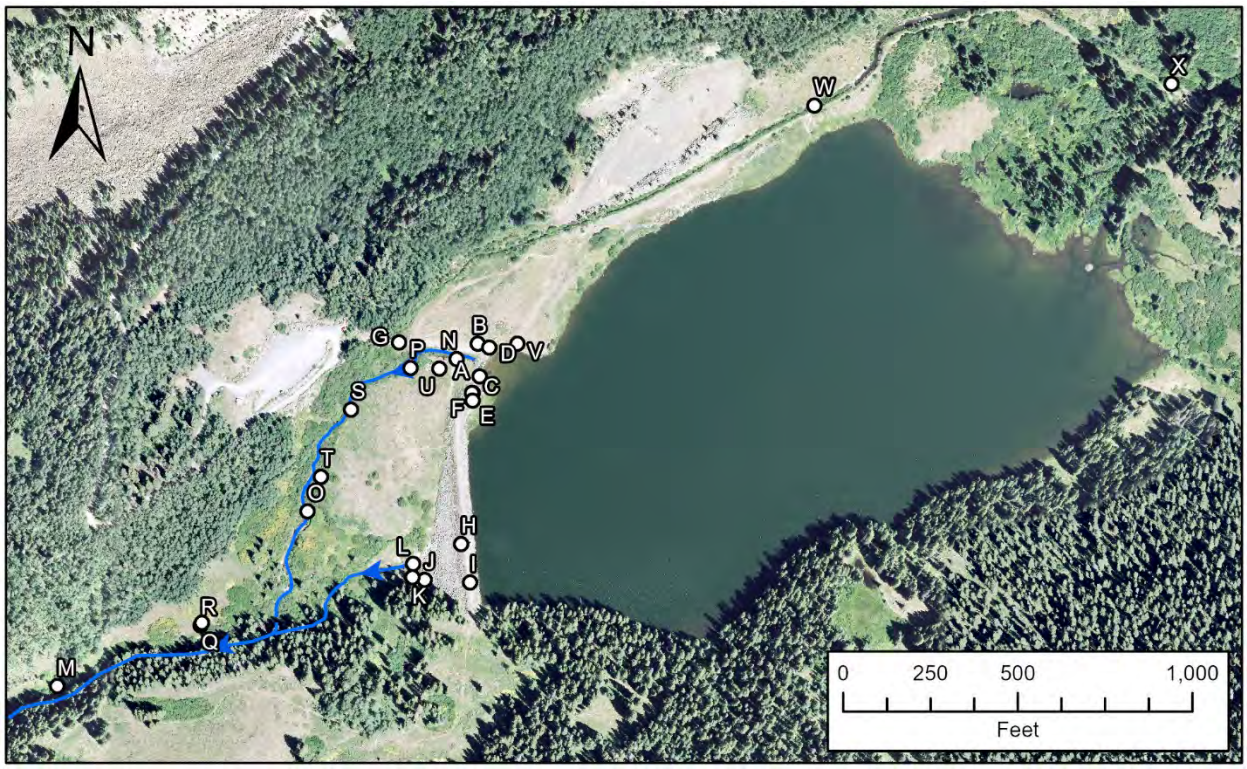
	ELEMENT	POR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
TEMPERATURE °F	NORMAL DAILY MAXIMUM	30	37.7	45.4	56.5	65.3	76.1	87.6	93.2	89.7	80.1	66.2	50.4	38.5	65.6
	MEAN DAILY MAXIMUM	120	36.6	43.1	55.4	64.3	75.5	86.0	92.9	89.5	79.7	67.1	50.6	38.8	65.0
	HIGHEST DAILY MAXIMUM	73	60	69	81	89	101	105	106	103	100	88	76	64	106
	YEAR OF OCCURRENCE		1971	2017	2004	2012	2000	1990	2005	2000	2019	2015	2008	1980	JUL 2005
	MEAN OF EXTREME MAXS.	120	49.4	58.8	71.4	80.7	89.8	98.6	101.1	98.2	92.8	81.7	65.3	52.2	78.3
	NORMAL DAILY MINIMUM	30	17.0	23.7	31.4	38.1	47.0	56.3	63.2	61.3	52.0	39.9	27.6	18.5	39.7
	MEAN DAILY MINIMUM	120	15.9	22.5	31.3	38.6	48.1	56.4	64.1	61.9	52.4	40.8	27.8	18.6	39.9
	LOWEST DAILY MINIMUM	73	-23	-18	5	11	24	34	44	43	29	6	-4	-17	-23
	YEAR OF OCCURRENCE		1963	1989	1948	1975	2011	1976	1993	1968	1978	2019	2004	1990	JAN 1963
	MEAN OF EXTREME MINS.	120	1.2	8.2	17.9	25.4	34.2	44.5	55.5	53.1	40.0	27.7	14.6	4.4	27.2
	NORMAL DRY BULB	30	27.4	34.5	43.9	51.7	61.6	72.0	78.2	75.5	66.1	53.0	39.0	28.5	52.6
	MEAN DRY BULB	120	26.2	32.8	43.3	51.5	61.8	71.3	78.5	75.7	66.1	53.9	39.2	28.7	52.4
	MEAN WET BULB	35	21.9	26.7	30.8	34.9	41.1	44.5	53.3	52.5	46.5	38.7	29.1	22.7	36.9
	MEAN DEW POINT	35	20.3	24.9	28.8	32.5	37.9	41.8	49.2	50.1	43.4	35.2	27.9	20.8	34.4
	NORMAL NO. DAYS WITH:														
	MAXIMUM >= 90	30	0.0	0.0	0.0	0.0	1.3	13.4	23.3	16.7	2.7	0.0	0.0	0.0	57.4
	MAXIMUM <= 32	30	7.9	1.6	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	6.6	16.7
	MINIMUM <= 32	30	30.2	24.3	17.1	6.2	0.4	0.0	0.0	0.0	0.2	3.9	22.8	30.1	135.2
	MINIMUM <= 0	30	1.6	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.8	2.7
H/C	NORMAL HEATING DEG. DAYS	30	1167	853	653	402	155	22	1	2	71	374	780	1131	5611
	NORMAL COOLING DEG. DAYS	30	0	0	0	3	48	231	410	327	103	3	0	0	1125
RH	NORMAL (PERCENT)		72	64	52	44	40	30	34	38	40	47	60	69	49
	HOUR 05 LST	30	79	74	65	59	56	44	49	52	54	60	71	78	62
	HOUR 11 LST	30	65	55	43	36	32	24	29	32	34	39	51	61	42
	HOUR 17 LST	30	63	49	36	29	26	19	22	25	27	33	48	59	36
	HOUR 23 LST	30	77	69	58	49	45	32	37	41	43	52	66	74	54
S	PERCENT POSSIBLE SUNSHINE	56	61	65	65	70	73	81	79	77	79	74	63	61	71
W/O	MEAN NO. DAYS WITH:														
	HEAVY FOG(VISBY <= 1/4 MI)	56	2.8	1.6	0.6	0.2	0.0	0.0	0.0	0.0	0.1	0.2	0.8	1.9	8.2
	THUNDERSTORMS	72	0.1	0.2	0.7	1.8	4.3	4.3	7.3	7.3	5.1	1.6	0.3	0.1	33.1
CLOUDINESS	MEAN:														
	SUNRISE-SUNSET (OKTAS)	50	4.9	5.0	5.0	4.8	4.4	3.2	3.3	3.4	2.9	3.4	4.2	4.5	4.1
	MIDNIGHT-MIDNIGHT (OKTAS)	32	4.6	4.4	4.6	4.3	4.0	3.0	3.2	3.3	2.7	3.2	4.0	4.3	3.8
	MEAN NO. DAYS WITH:														
	CLEAR	50	9.1	7.6	8.0	7.9	9.6	14.9	13.7	13.1	16.0	14.7	10.4	9.6	134.6
	PARTLY CLOUDY	50	7.0	7.4	8.6	9.4	10.7	9.4	11.2	11.2	8.1	7.6	7.4	7.6	105.6
	CLOU		14.8	13.3	14.4	12.7	10.7	5.7	5.5	6.1	5.3	8.0	11.6	13.1	121.2
PR	MEAN		25.26	25.17	25.13	25.08	25.09	25.11	25.19	25.21	25.20	25.20	25.23	25.24	25.18
	MEAN		30.23	30.11	29.96	29.87	29.82	29.80	29.86	29.90	29.93	30.01	30.12	30.21	29.99
WINDS	MEAN SPEED (MPH)	36	5.3	6.4	7.9	8.9	9.1	9.4	9.2	8.7	8.5	7.4	6.1	5.3	7.7
	PREVAIL.DIR(TENS OF DEGS)	14	08	11	11	12	12	12	12	12	12	12	11	08	12
	MAXIMUM 2-MINUTE:														
	SPEED (MPH)	23	40	41	53	49	46	48	54	45	46	51	45	44	54
	DIR. (TENS OF DEGS)		18	32	34	19	21	30	31	31	22	24	33	22	31
	YEAR OF OCCURRENCE		2005	2012	2001	2007	2011	1999	2018	2002	2016	2006	2005	2007	JUL 2018
	MAXIMUM 3-SECOND														
	SPEED (MPH)	23	49	55	66	73	68	71	70	64	60	62	55	54	73
	DIR. (TENS OF DEGS)		18	19	26	29	28	25	31	26	36	25	33	22	29
	YEAR OF OCCURRENCE		2005	1998	2009	2019	2004	2017	2018	1999	2017	2006	2005	2016	APR 2019
PRECIPITATION	NORMAL (IN)	30	0.58	0.54	0.92	0.91	0.88	0.46	0.61	0.95	1.19	1.06	0.73	0.59	9.42
	MAXIMUM MONTHLY (IN)	73	2.46	1.56	2.29	2.30	2.04	2.07	1.92	3.48	3.09	3.45	2.00	2.05	3.48
	YEAR OF OCCURRENCE		1957	1948	2019	2004	1995	1969	1983	1957	2013	1972	1983	2007	AUG 1957
	MINIMUM MONTHLY (IN)	73	T	T	0.02	0.06	T	T	0.01	0.04	T	0.00	T	0.01	0.00
	YEAR OF OCCURRENCE		1961	1972	1972	1958	1970	1980	1994	1956	2018	1952	1989	1976	OCT 1952
	MAXIMUM IN 24 HOURS (IN)	73	.74	0.81	1.15	1.33	1.13	1.57	1.42	1.68	1.35	1.36	0.83	1.16	1.68
	YEAR OF OCCURRENCE		2005	1996	1993	1965	1983	1969	1974	1997	1965	2018	1983	1951	AUG 1997
	NORMAL NO. DAYS WITH:														
	PRECIPITATION >= 0.01	30	5.9	6.2	7.3	7.3	6.4	4.1	5.3	6.7	7.0	6.5	5.9	5.7	74.3
	PRECIPITATION >= 1.00	30	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.2
SNOWFALL	NORMAL (IN)	30	4.9	3.2	2.6	0.9	0.1	0.0	0.0	0.0	0.0	0.4	2.1	4.9	19.1
	MAXIMUM MONTHLY (IN)	73	33.7	18.4	14.9	14.3	5.0	T	T	T	3.1	6.1	12.1	19.0	33.7
	YEAR OF OCCURRENCE		1957	1948	1948	1975	1979	2009	2015	2018	1965	1975	1964	1983	JAN 1957
	MAXIMUM IN 24 HOURS (IN)	73	9.1	9.0	6.1	8.9	5.0	T	T	T	3.1	6.1	8.4	7.2	9.1
	YEAR OF OCCURRENCE'		1957	1989	1948	1975	1979	2009	2015	2018	1965	1975	1954	2009	JAN 1957
	MAXIMUM SNOW DEPTH (IN)	71	16	12	8	7	1	0	0	0	2	5	8	11	16
	YEAR OF OCCURRENCE		1957	1957	1960	1975	1979				1965	1975	1954	1983	JAN 1957
	NORMAL NO. DAYS WITH:														
	SNOWFALL >= 1.0	30	1.5	1.1	0.9	0.3	0.0	0.0	0.0	0.0	0.0	0.1	0.7	1.8	6.4

POR 1997-2019

Appendix H
Site Photos

Appendix

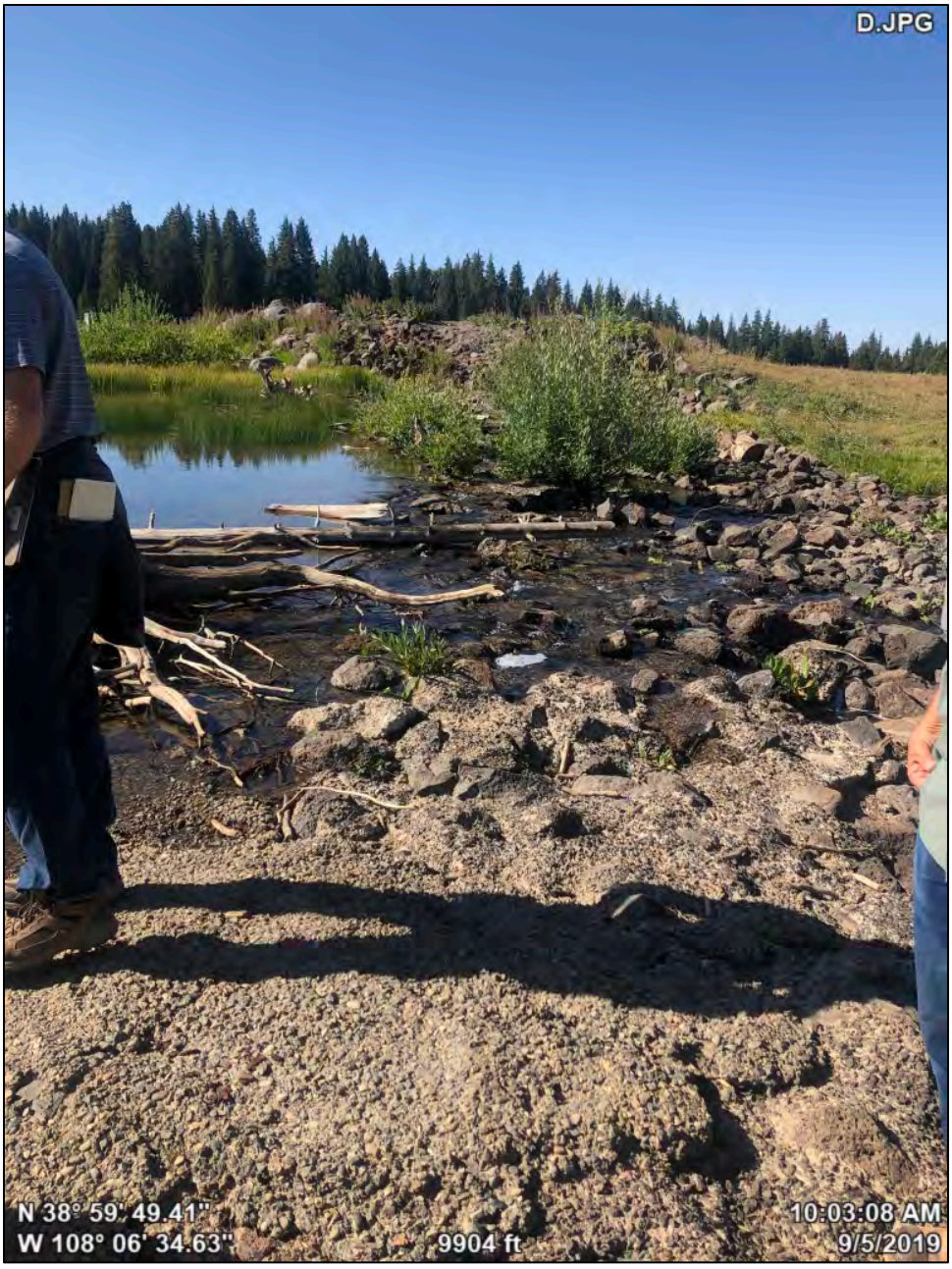
Site Visit Photographs



Spillway Weir

Spillway Weir

Spillway Weir



Spillway Weir

Hogchute Dam Embankment

Carson Lake looking Northeast

Bypass Channel and Foot Bridge

Hogchute Dam Outlet

Hogchute Dam Embankment

Hogchute Dam Outlet

Hogchute Dam Outlet

Hogchute Dam Outlet

Kannah Creek

Carson Lake Spillway Channel

Carson Lake Spillway Channel

Carson Lake Spillway Channel

Carson Lake Spillway Floodplain

Carson Lake Spillway Floodplain

Carson Lake Spillway Channel

Carson Lake Spillway Floodplain

Carson Lake Spillway Channel

Carson Lake looking Southwest

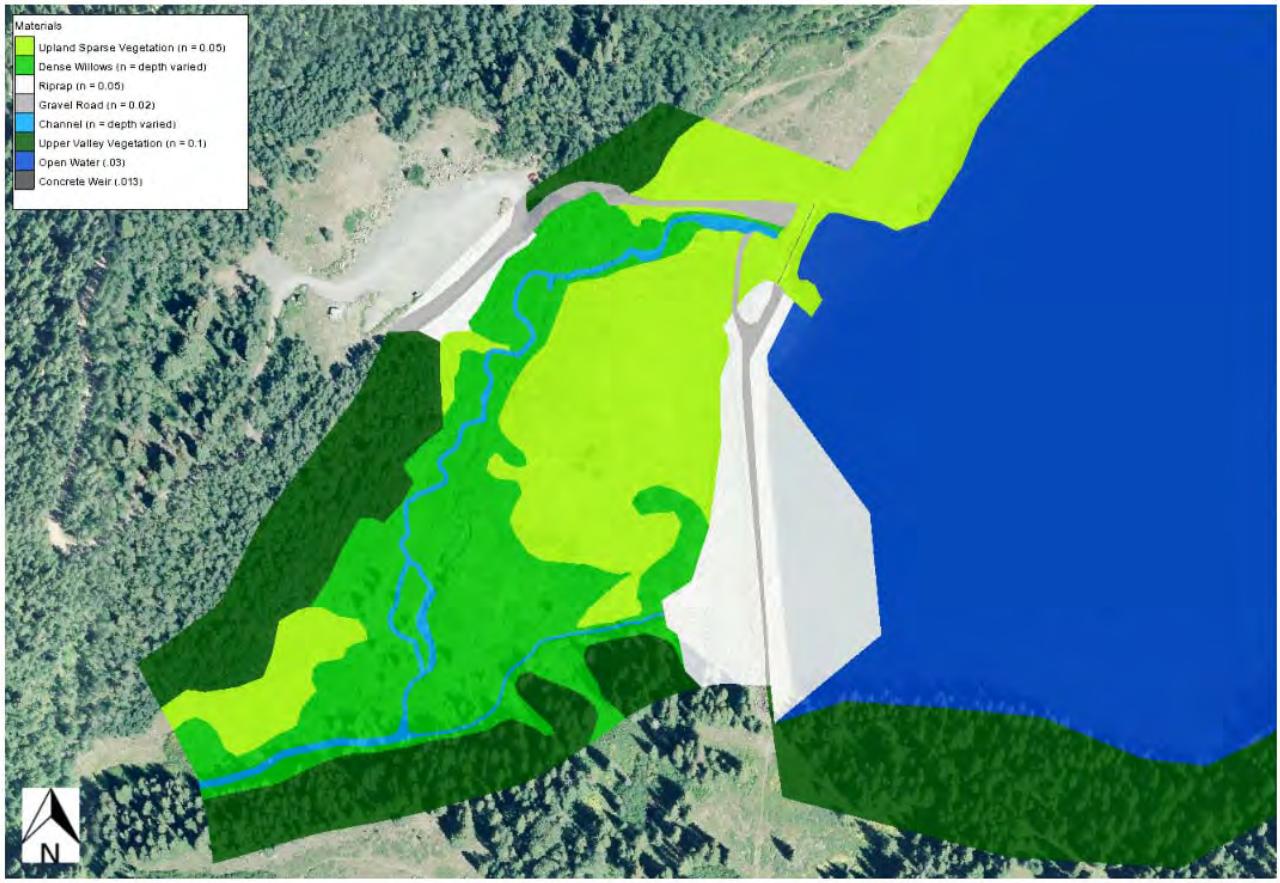
Bypass Channel Overtopping Location

Diversion Structure for Bypass Channel

Appendix I

Spillway Hydraulic Calculations

Materials Coverage Map for Spillway 2-D Hydraulic Model



Hogchute Dam Rehabilitation
Project No: 26-1144.00

Computations: CRH
Checked By: DWR
FEB 2021

Spillway Rock Size Calculations

Isbash:

$$V_C = 11.17\left(Y^{1/6}\right)\left({D_{50,p}}^{1/3}\right) = 11.17\left(5.33^{1/6}\right)\left(0.58^{1/3}\right) = \ 12.3\ ft/s$$

V_c = Critical velocity [ft/s]
D_{50} = Median particle diameter [ft]
Y = Average depth [ft]

$$D_{50,s} = \frac{{V_C}^2}{C^2\left(2g\frac{\gamma_s - \gamma_w}{\gamma_w}\right)} = \frac{12.3^2}{0.86^2\left(2 \times 32.2 \times \frac{165 - 62.4}{62.4}\right)} = \mathbf{1.94\ ft}$$

$D_{50,s}$ = Median stone diameter [ft]
V_c = Critical velocity [ft/s]
C = 0.86 (high turbulence)
g = Gravity, 32.2 [ft/s^2]
γ_s = Stone density, 165 [lb/ft^3]
γ_w = Water density, 62.4 [lb/ft^3]

USBR:

$$D_{50,s} = 0.0122V^{2.06} = \ 0.0122(11.56^{2.06}) = \mathbf{1.89\ ft}$$

$D_{50,s}$ = Median stone diameter [ft]
V = Average channel velocity [ft/s]*
* Used maximum velocity

USACE:

$$q = \frac{Q}{b} = \frac{37.3}{41.2} = 0.906\ m^2/s$$

q = Unit discharge [m^2/s]
Q = Total flow [m^3/s] (100-yr peak)
b = bottom width [m] (shortest berm width)

$$D_{30,s} = \frac{1.95S^{0.555}(Cq)^{2/3}}{g^{1/3}} = \frac{1.95(0.064^{0.555})(1.25 \times 0.906)^{2/3}}{9.81^{1/3}} = 0.21\ m = \mathbf{0.70\ ft}$$

$D_{30,s}$ = 30% finer stone diameter [m]
S = Channel slope
q = Unit discharge [m^2/s]
g = Gravity, 9.81 [m/s^2]
C = flow concentration factor, 1.25

Appendix J

Structural Calculations

CONCRETE BUTTRESS
HOG CHUTE DAM - MESA COUNTY, COLORADO
Stability & Temperature Reinforcement

Ref.:
1) *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2) EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, USACOE, 30 Nov. 2016.
3) EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACOE, 1 December 2005.
4) EM 1110-2-2200, *Gravity Dam Design*, USACOE, 30 June 1995.
5) EM 1110-2-2502, *Retaining and Flood Walls*, USACOE, 29 September 1989.
6) EM 1110-2-6053, *Earthquake Design and Evaluation of Concrete Hydraulic Structures*, USACOE, 1 May 2007.
7) Letter: Geotechnical Design Parameters, Carson Lake, Mesa County, Colorado, from Michael A. Berry, PE, Huddleston-Berry, 27 August 2020 & 15 September 2020 email.
8) Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.

Design

1) Load Cases Per Ref. 3.
Reservoir dewatered w/ cofferdam during construction. Structure is then submerged as the structure is located at the toe on the upstream side of the embankment.

LC 1 - Unusual: Construction
LC 2 - Extreme: Construction with OBE
LC 3 - Usual: Normal Pool
LC 4 - Unusual: Infrequent Flood = Q100
LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
LC 6 - Extreme: Coincident Pool = Normal Pool with MDE
LC 7 - Extreme: Maximum Design Flood = Q10000

2) Required Sliding Factors of Safety (Ref. 3, Table 3-2) and Resultant Location (Ref. 3, Table 3-5)
Usual = 1.7 w/ 100% Base in Compression
Unusual = 1.3 w/ 75% Base in Compression
Extreme = 1.1 w/ Resultant In Base

3) Assume structure will not deflect sufficiently to cause active earth pressure conditions to develop so use use at-rest earth pressures.
Ref. 7: Site PGA = 0.236. k_h is defined as 1/3 of the site modified PGA so k_h = 0.08 for this site.
Ref. 6: Section 7-3, MDE = 2/3 MCE = (2/3)(0.277) = 0.185
Zero vertical earthquake acceleration.

4) Per Ref. 8:
Normal Pool El. = 9895.00 feet
Q10 Pool El. = 9895.45 feet
Q100 Pool El. = 9897.16 feet
Q10000 Pool El. = 9900.40 feet

5) Elevations in NAVD88.

Variables

Variables

Usual case when k_v, δ, and θ are zero.

Concrete Unit Weight
Ref. 4, Section 3-3b
$\gamma_c := 150 \cdot pcf$

Soil Moist Unit Weight
Ref. 7
$\gamma_m := 125 \cdot pcf$

Soil Saturated Unit Weight
Ref. 7
$\gamma_s := 130 \cdot pcf$

Backfill Friction Angle
Ref. 7
$\phi := 32 \cdot deg$

Foundation Friction Angle
Ref. 7
$\phi_{foundation} := 21 \cdot deg$

Horizontal Acceleration (in g's)
Ref. 7
$k_{h_OBE} := 0.08$
$k_{h_MDE} := 0.185$

Water Unit Weight
Ref. 4, Section 3-3c
$\gamma_w := 62.5 \cdot pcf$

Inclination of Soil Surface
(0 if level backfill)
$\beta_a := 21.8 \cdot deg$ [2.5H:1V]
$\beta_p := 0 \cdot deg$

Note: Upstream embankment slope varies. Concrete buttress is based on an approximate 2.75H:1V, but use 2.5H:1V to account for potential variations.

Structure Height
$h := 9 \cdot ft + 1.5 \cdot in$ [against emb.]

Heel Soil Height
$h_{s1} := h$
$h_{s1} = 9.125\ ft$

Toe Soil Height
$h_{s2} := 2 \cdot ft + 6 \cdot in$

Elevations
$EL_{w_NP} := 9895.00 \cdot ft$
$EL_{w_Q100} := 9895.45 \cdot ft$
$EL_{w_Q10000} := 9900.40 \cdot ft$
$EL_{base} := 9846.75 \cdot ft$

Total Width
$Width := 12 \cdot ft + 0 \cdot in$

Total Base
$Base := 17 \cdot ft + 9 \cdot in$

Uniform Surcharge
$q_{surcharge} := 200 \cdot psf$

Allowable Bearing Capacity
Ref. 7
$BearingCap_{allow} := 2000 \cdot psf$
$BearingCap_{allow_eq} := 2666 \cdot psf$

Define Check Function: $Check(x, y) := if(x \geq y, "OK", "NG")$

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Eau Claire, WI

Buttress

Prepared By: TMR Nov 2020
Checked By: AEB Jan 2021

Stability

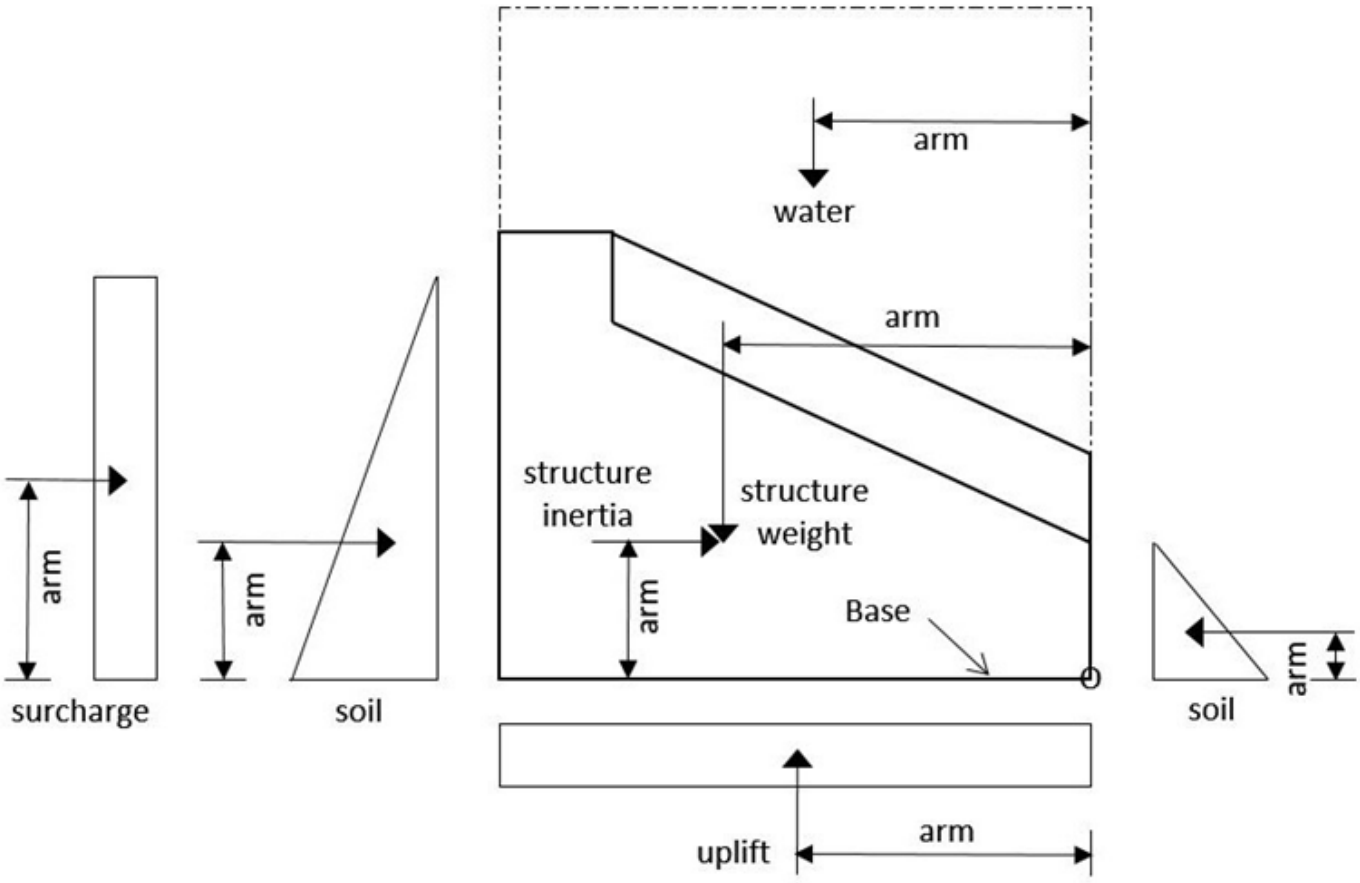

Figure 1 - Loads

Figure 1 & 2 Notes:
Top 'curb' extends along both sides.

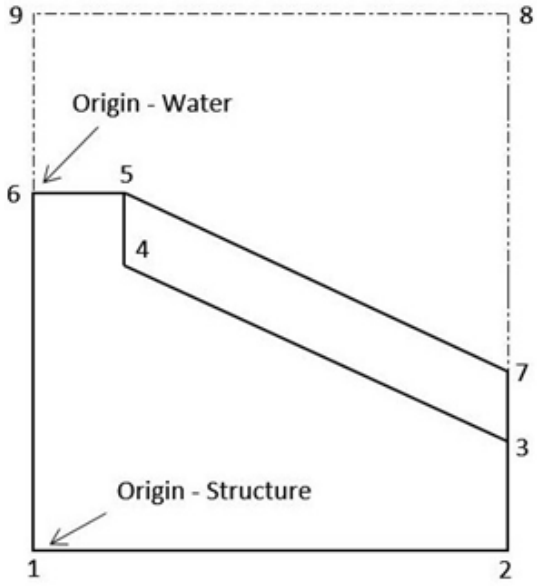

Figure 2 - Concrete and Water

Stability

VERTICAL FORCES

-- see Figure 1 (arms from point O)

Concrete 1: Main Wedge - see Figure 2

$$x_{PTc} := \begin{pmatrix} 0 \\ Base \cdot ft^{-1} \\ Base \cdot ft^{-1} \\ 1.5 \\ 1.5 \\ 0 \\ 0 \end{pmatrix} \cdot ft \qquad y_{PTc} := \begin{pmatrix} EL_{base} \cdot ft^{-1} \\ EL_{base} \cdot ft^{-1} \\ 9849.25 \\ 9855.33 \\ 9858.25 \\ 9858.25 \\ EL_{base} \cdot ft^{-1} \end{pmatrix} \cdot ft$$

$N := 7$ (number of coordinate points in matrix)

$$A_{all} := \sum_{i=1}^{N-1} \left[\frac{1}{2} \left(x_{PTc_i} \cdot y_{PTc_{i+1}} - x_{PTc_{i+1}} \cdot y_{PTc_i} \right) \right]$$

(Area)

$A_{all} = 107.3 \cdot ft^2$

$$c_{xPTc} := \sum_{i=1}^{N-1} \left[\frac{1}{6 \cdot A_{all}} \cdot \left(x_{PTc_i} + x_{PTc_{i+1}} \right) \cdot \left(x_{PTc_i} \cdot y_{PTc_{i+1}} - x_{PTc_{i+1}} \cdot y_{PTc_i} \right) \right] \qquad c_{xPTc} = 6.95\ ft \quad \text{From origin (heel)}$$

$$c_{yPTc} := \sum_{i=1}^{N-1} \left[\frac{1}{6 \cdot A_{all}} \cdot \left(y_{PTc_i} + y_{PTc_{i+1}} \right) \cdot \left(x_{PTc_i} \cdot y_{PTc_{i+1}} - x_{PTc_{i+1}} \cdot y_{PTc_i} \right) \right] \qquad c_{yPTc} = 9850.23\ ft \quad \text{Elevation}$$

$arm_y_{all} := c_{yPTc} - EL_{base}$ $\quad arm_y_{all} = 3.48\ ft$ From base

$armWT_x_{all} := Base - c_{xPTc}$ $\quad armWT_x_{all} = 10.80\ ft$ From toe

$WTc_{all} := \gamma_c \cdot A_{all} \cdot Width$ $\quad WTc_{all} = 193.10 \cdot kip$

$MV_Wc_{all} := WTc_{all} \cdot armWT_x_{all}$ $\quad MV_Wc_{all} = 2085.3 \cdot kip \cdot ft$

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Eau Claire, WI

Buttress

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Stability

-- remove approximate volume for pipe
-- length is about 11.0 feet
-- centroids are approximate

$$WTc_{pipe} := \gamma_c \cdot \left(\frac{\pi}{4}\right) \cdot (2.5 \cdot ft)^2 \cdot (8.0 \cdot ft + 3.0 \cdot ft) \qquad WTc_{pipe} = 8.10 \cdot kip$$

$$armWTc_{pipe_US} := \frac{\left[\gamma_c \cdot \left(\frac{\pi}{4}\right) \cdot (2.5 \cdot ft)^2 \cdot [(8.0 \cdot ft) \cdot (4.0 \cdot ft) + (3.0 \cdot ft) \cdot (9.5 \cdot ft)]\right]}{WTc_{pipe}} \qquad armWTc_{pipe_US} = 5.50\ ft$$

From origin (heel)

$$armWTc_{pipe_DS} := Base - armWTc_{pipe_US} \qquad armWTc_{pipe_DS} = 12.25\ ft$$

From toe

$$MV_Wc_{pipe} := WTc_{pipe} \cdot armWTc_{pipe_DS} \qquad MV_Wc_{pipe} = 99.2 \cdot kip \cdot ft$$

$$arm_y_{pipe} := \frac{\left[\gamma_c \cdot \left(\frac{\pi}{4}\right) \cdot (2.5 \cdot ft)^2 \cdot \left[(8.0 \cdot ft) \cdot \left(9850.0 \cdot ft - EL_{base}\right) + (3.0 \cdot ft) \cdot \left(1.0 \cdot ft + 9850.0 \cdot ft - EL_{base}\right)\right]\right]}{WTc_{pipe}}$$

$$arm_y_{pipe} = 3.52\ ft$$

Stability

Concrete 2: Curbs - see Figure 2

$$x_{PTcc} := \begin{pmatrix} 1.5 \\ Base \cdot ft^{-1} \\ Base \cdot ft^{-1} \\ 1.5 \\ 1.5 \end{pmatrix} \cdot ft \qquad y_{PTcc} := \begin{pmatrix} 9855.33 \\ 9849.25 \\ 9852.17 \\ 9858.25 \\ 9855.33 \end{pmatrix} \cdot ft$$

$N := 5$ (number of coordinate points in matrix)

$$A_{curb} := \sum_{i=1}^{N-1} \left[\frac{1}{2} \left(x_{PTcc_i} \cdot y_{PTcc_{i+1}} - x_{PTcc_{i+1}} \cdot y_{PTcc_i} \right) \right]$$

(Area)

$A_{curb} = 47.4 \cdot ft^2$

$$c_{xPTcc} := \sum_{i=1}^{N-1} \left[\frac{1}{6 \cdot A_{curb}} \cdot \left(x_{PTcc_i} + x_{PTcc_{i+1}} \right) \cdot \left(x_{PTcc_i} \cdot y_{PTcc_{i+1}} - x_{PTcc_{i+1}} \cdot y_{PTcc_i} \right) \right] \qquad c_{xPTcc} = 9.62\ ft$$ From origin (heel)

$$c_{yPTcc} := \sum_{i=1}^{N-1} \left[\frac{1}{6 \cdot A_{curb}} \cdot \left(y_{PTcc_i} + y_{PTcc_{i+1}} \right) \cdot \left(x_{PTcc_i} \cdot y_{PTcc_{i+1}} - x_{PTcc_{i+1}} \cdot y_{PTcc_i} \right) \right] \qquad c_{yPTcc} = 9853.75\ ft$$ Elevation

$arm_y_{curb} := c_{yPTcc} - EL_{base}$ $\qquad arm_y_{curb} = 7.00\ ft$ From base

$armWT_x_{curb} := Base - c_{xPTcc}$ $\qquad armWT_x_{curb} = 8.13\ ft$ From toe

$WTc_{curb} := (2) \cdot \gamma_c \cdot A_{curb} \cdot (1.5 \cdot ft)$ $\qquad WTc_{curb} = 21.35 \cdot kip$

$MV_Wc_{curb} := WTc_{curb} \cdot armWT_x_{curb}$ $\qquad MV_Wc_{curb} = 173.5 \cdot kip \cdot ft$

Concrete Sum

$V_Wc := WTc_{all} - WTc_{pipe} + WTc_{curb}$ $\qquad V_Wc = 206.35 \cdot kip$

$MV_Wc := MV_Wc_{all} - MV_Wc_{pipe} + MV_Wc_{curb}$ $\qquad MV_Wc = 2159.6 \cdot kip \cdot ft$

$$armV_Wc := \frac{MV_Wc}{V_Wc} \qquad armV_Wc = 10.47\ ft$$

Stability

Uplift

-- Uniform

$UP_{NP} := \gamma_w \cdot Width \cdot Base \cdot (EL_{w_NP} - EL_{base})$ $UP_{NP} = 642.33 \cdot kip$

$armUP_{NP} := \frac{Base}{2}$ $armUP_{NP} = 8.875\,ft$

$M_UP_{NP} := UP_{NP} \cdot armUP_{NP}$ $M_UP_{NP} = 5700.7 \cdot kip \cdot ft$

$UP_{Q100} := \gamma_w \cdot Width \cdot Base \cdot (EL_{w_Q100} - EL_{base})$ $UP_{Q100} = 648.32 \cdot kip$

$armUP_{Q100} := \frac{Base}{2}$ $armUP_{Q100} = 8.875\,ft$

$M_UP_{Q100} := UP_{Q100} \cdot armUP_{Q100}$ $M_UP_{Q100} = 5753.8 \cdot kip \cdot ft$

$UP_{Q10000} := \gamma_w \cdot Width \cdot Base \cdot (EL_{w_Q10000} - EL_{base})$ $UP_{Q10000} = 714.22 \cdot kip$

$armUP_{Q10000} := \frac{Base}{2}$ $armUP_{Q10000} = 8.875\,ft$

$M_UP_{Q10000} := UP_{Q10000} \cdot armUP_{Q10000}$ $M_UP_{Q10000} = 6338.7 \cdot kip \cdot ft$

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Buttress

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Stability

Water 1: Full NP, Q100, and Q1000 - see Figure 2

$$x_{PTw1} := \begin{pmatrix} 0 \\ 1.5 \\ 1.5 \\ Base \cdot ft^{-1} \\ Base \cdot ft^{-1} \\ 0 \\ 0 \end{pmatrix} \cdot ft \qquad y_{PTw1} := \begin{pmatrix} 9858.25 \\ 9858.25 \\ 9855.33 \\ 9849.25 \\ EL_{w_NP} \cdot ft^{-1} \\ EL_{w_NP} \cdot ft^{-1} \\ 9858.25 \end{pmatrix} \cdot ft$$

$N := 7$ (number of coordinate points in matrix)

$$A_{NP} := \sum_{i=1}^{N-1} \left[\frac{1}{2} \left(x_{PTw1_i} \cdot y_{PTw1_{i+1}} - x_{PTw1_{i+1}} \cdot y_{PTw1_i} \right) \right]$$

(Area)

$A_{NP} = 749.2 \cdot ft^2$

$$c_{xPTw1} := \sum_{i=1}^{N-1} \left[\frac{1}{6 \cdot A_{NP}} \cdot \left(x_{PTw1_i} + x_{PTw1_{i+1}} \right) \cdot \left(x_{PTw1_i} \cdot y_{PTw1_{i+1}} - x_{PTw1_{i+1}} \cdot y_{PTw1_i} \right) \right]$$

$c_{xPTw1} = 9.15$ ft

From origin (heel)

$armWTw_{NPf} := Base - c_{xPTw1}$ $armWTw_{NPf} = 8.60$ ft From toe

$WTw_{NPf} := \gamma_w \cdot A_{NP} \cdot Width$ $WTw_{NPf} = 561.87 \cdot kip$

$MV_Ww_{NPf} := WTw_{NPf} \cdot armWTw_{NPf}$ $MV_Ww_{NPf} = 4831.8 \cdot kip \cdot ft$

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Stability

$$x_{PTw2} := \begin{pmatrix} 0 \\ 1.5 \\ 1.5 \\ Base \cdot ft^{-1} \\ Base \cdot ft^{-1} \\ 0 \\ 0 \end{pmatrix} \cdot ft \qquad y_{PTw2} := \begin{pmatrix} 9858.25 \\ 9858.25 \\ 9855.33 \\ 9849.25 \\ EL_{w_Q100} \cdot ft^{-1} \\ EL_{w_Q100} \cdot ft^{-1} \\ 9858.25 \end{pmatrix} \cdot ft$$

$N := 7$ (number of coordinate points in matrix)

$$A_{Q100} := \sum_{i=1}^{N-1} \left[\frac{1}{2} \left(x_{PTw2_i} \cdot y_{PTw2_{i+1}} - x_{PTw2_{i+1}} \cdot y_{PTw2_i} \right) \right]$$

(Area)

$A_{Q100} = 757.2 \cdot ft^2$

$$c_{xPTw2} := \sum_{i=1}^{N-1} \left[\frac{1}{6 \cdot A_{Q100}} \cdot \left(x_{PTw2_i} + x_{PTw2_{i+1}} \right) \cdot \left(x_{PTw2_i} \cdot y_{PTw2_{i+1}} - x_{PTw2_{i+1}} \cdot y_{PTw2_i} \right) \right]$$

$c_{xPTw2} = 9.15\ ft$

From origin (heel)

$armWTw_{Q100f} := Base - c_{xPTw2}$ $armWTw_{Q100f} = 8.60\ ft$ From toe

$WTw_{Q100f} := \gamma_w \cdot A_{Q100} \cdot Width$ $WTw_{Q100f} = 567.86 \cdot kip$

$MV_Ww_{Q100f} := WTw_{Q100f} \cdot armWTw_{Q100f}$ $MV_Ww_{Q100f} = 4885.0 \cdot kip \cdot ft$

$$x_{PTw3} := \begin{pmatrix} 0 \\ 1.5 \\ 1.5 \\ Base \cdot ft^{-1} \\ Base \cdot ft^{-1} \\ 0 \\ 0 \end{pmatrix} \cdot ft \qquad y_{PTw3} := \begin{pmatrix} 9858.25 \\ 9858.25 \\ 9855.33 \\ 9849.25 \\ EL_{w_Q10000} \cdot ft^{-1} \\ EL_{w_Q10000} \cdot ft^{-1} \\ 9858.25 \end{pmatrix} \cdot ft$$

$N := 7$ (number of coordinate points in matrix)

$$A_{Q10000} := \sum_{i=1}^{N-1} \left[\frac{1}{2} \left(x_{PTw3_i} \cdot y_{PTw3_{i+1}} - x_{PTw3_{i+1}} \cdot y_{PTw3_i} \right) \right]$$

(Area)

$A_{Q10000} = 845.0 \cdot ft^2$

$$c_{xPTw3} := \sum_{i=1}^{N-1} \left[\frac{1}{6 \cdot A_{Q10000}} \cdot \left(x_{PTw3_i} + x_{PTw3_{i+1}} \right) \cdot \left(x_{PTw3_i} \cdot y_{PTw3_{i+1}} - x_{PTw3_{i+1}} \cdot y_{PTw3_i} \right) \right]$$

$c_{xPTw3} = 9.12\ ft$

From origin (heel)

$armWTw_{Q10000f} := Base - c_{xPTw3}$ $armWTw_{Q10000f} = 8.63\ ft$ From toe

$WTw_{Q10000f} := \gamma_w \cdot A_{Q10000} \cdot Width$ $WTw_{Q10000f} = 633.76 \cdot kip$

$MV_Ww_{Q10000f} := WTw_{Q10000f} \cdot armWTw_{Q10000f}$ $MV_Ww_{Q10000f} = 5469.8 \cdot kip \cdot ft$

Stability

Water 2: Remove Curbs - see Figure 2

$A_{curb} = 47.4 \cdot ft^2$

$arm_y_{curb} = 7.00\ ft$ From base

$armWT_x_{curb} = 8.13\ ft$ From toe

$WTw_{curb} := (2) \cdot \gamma_w \cdot A_{curb} \cdot (1.5 \cdot ft)$ $WTw_{curb} = 8.90 \cdot kip$

$MV_Ww_{curb} := WTw_{curb} \cdot armWT_x_{curb}$ $MV_Ww_{curb} = 72.3 \cdot kip \cdot ft$

Water Sum: NP

$WTw_{NP} := WTw_{NPf} - WTw_{curb}$ $WTw_{NP} = 552.98 \cdot kip$

$MV_Ww_{NP} := MV_Ww_{NPf} - MV_Ww_{curb}$ $MV_Ww_{NP} = 4759.5 \cdot kip \cdot ft$

$armV_Ww_{NP} := \frac{MV_Ww_{NP}}{WTw_{NP}}$ $armV_Ww_{NP} = 8.61\ ft$

Water Sum: Q100

$WTw_{Q100} := WTw_{Q100f} - WTw_{curb}$ $WTw_{Q100} = 558.97 \cdot kip$

$MV_Ww_{Q100} := MV_Ww_{Q100f} - MV_Ww_{curb}$ $MV_Ww_{Q100} = 4812.7 \cdot kip \cdot ft$

$armV_Ww_{Q100} := \frac{MV_Ww_{Q100}}{WTw_{Q100}}$ $armV_Ww_{Q100} = 8.61\ ft$

Water Sum: Q10000

$WTw_{Q10000} := WTw_{Q10000f} - WTw_{curb}$ $WTw_{Q10000} = 624.86 \cdot kip$

$MV_Ww_{Q10000} := MV_Ww_{Q10000f} - MV_Ww_{curb}$ $MV_Ww_{Q10000} = 5397.5 \cdot kip \cdot ft$

$armV_Ww_{Q10000} := \frac{MV_Ww_{Q10000}}{WTw_{Q10000}}$ $armV_Ww_{Q10000} = 8.64\ ft$

Flotation

Flotation - LC 3: Normal, LC 4: Q100, LC 7: Q10000

Ref. 3, Section 3-8.

$W_s := V_Wc$ $\quad W_s = 206.35 \cdot kip$

$W_c := 0 \cdot kip$ $\quad W_c = 0.00 \cdot kip$

$S := 0 \cdot kip$ $\quad S = 0 \cdot kip$

$UP_{NP} = 642.33 \cdot kip$

$UP_{Q100} = 648.32 \cdot kip$

$UP_{Q10000} = 714.22 \cdot kip$

$W_{g_NP} := WTw_{NP}$ $\quad W_{g_NP} = 552.98 \cdot kip$

$W_{g_Q100} := WTw_{Q100}$ $\quad W_{g_Q100} = 558.97 \cdot kip$

$W_{g_Q10000} := WTw_{Q10000}$ $\quad W_{g_Q10000} = 624.86 \cdot kip$

$$SF3_{fl} := \frac{W_s + W_c + S}{UP_{NP} - W_{g_NP}}$$ $\quad SF3_{fl} = 2.31$

$\text{Check}(SF3_{fl}, 1.3) = \text{"OK"}$

$$SF4_{fl} := \frac{W_s + W_c + S}{UP_{Q100} - W_{g_Q100}}$$ $\quad SF4_{fl} = 2.31$

$\text{Check}(SF4_{fl}, 1.2) = \text{"OK"}$

$$SF7_{fl} := \frac{W_s + W_c + S}{UP_{Q10000} - W_{g_Q10000}}$$ $\quad SF7_{fl} = 2.31$

$\text{Check}(SF7_{fl}, 1.1) = \text{"OK"}$

*** Structure meets flotation requirements. ***

Stability

HORIZONTAL FORCES

-- see Figure 1 (arms from base)

Soil:

At-Rest Earth Pressure Coefficients:

$K_{o_a} := (1 - \sin(\phi)) \cdot (1 + \sin(\beta_a))$ $K_{o_a} = 0.645$

$K_{o_p} := (1 - \sin(\phi)) \cdot (1 + \sin(\beta_p))$ $K_{o_p} = 0.470$

$F_{s1m} := \frac{1}{2} \cdot Width \cdot K_{o_a} \cdot \gamma_m \cdot h_{s1}^2$ $F_{s1m} = 40.26 \cdot kip$ $armF_{s1m} := \frac{h_{s1}}{3}$ $armF_{s1m} = 3.042\ ft$

$F_{s1} := \frac{1}{2} \cdot Width \cdot K_{o_a} \cdot (\gamma_s - \gamma_w) \cdot h_{s1}^2$ $F_{s1} = 21.74 \cdot kip$ $armF_{s1} := \frac{h_{s1}}{3}$ $armF_{s1} = 3.042\ ft$

$F_{s2} := \frac{1}{2} \cdot Width \cdot K_{o_p} \cdot (\gamma_s - \gamma_w) \cdot h_{s2}^2$ $F_{s2} = 1.19 \cdot kip$ $armF_{s2} := \frac{h_{s2}}{3}$ $armF_{s2} = 0.833\ ft$

Water: Equal on both sides so do not compute since cancel each other out.
(Construction does not have water present).

Surcharge: Only during construction (LC1) due to embankment height.

$F_{surcharge} := K_{o_a} \cdot q_{surcharge} \cdot Width \cdot h_{s1}$ $F_{surcharge} = 14.12 \cdot kip$

$armF_{surcharge} := \frac{h_{s1}}{2}$ $armF_{surcharge} = 4.56\ ft$

$MH_Sur := F_{surcharge} \cdot armF_{surcharge}$ $MH_Sur = 64.4 \cdot kip \cdot ft$

Stability

Earthquake for Load Cases 2, 5 and 6

The earthquake load cases again are:

LC 2 - Extreme: Construction with OBE
LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
LC 6 - Extreme: Coincident Pool = Normal Pool with MDE

The structure is located at the toe on the upstream side of the embankment.
Therefore, do not perform analyses for LC 5 and LC 6 as the structure is submerged.
The structure will not be a) pushed into the embankment or 2) pushed into the water column.

Mononobe-Okabe Analysis (Ref. 3, Appendix G)

Usual case when k_v, δ, and θ are zero.

Check limits (as a guide) even though we are using at-rest earth pressures.

$\text{Check}(\tan(\phi - \beta_a), k_{h_OBE}) = \text{"OK"}$ $\text{Check}(\tan(\phi - \beta_a), k_{h_MDE}) = \text{"NG"}$ Driving Wedge Limit (Equ. G-7)

$\text{Check}(\tan(\phi + \beta_p), k_{h_OBE}) = \text{"OK"}$ $\text{Check}(\tan(\phi + \beta_p), k_{h_MDE}) = \text{"OK"}$ Resisting Wedge Limit (Equ. G-8)

--> Equ. G-7 is NG which means it will provide a negative number, so a real solution is not possible.

Use alternate method.

Seismic Coefficient Method (Ref. 6, Section 7-3a)

Use the provided coefficient to compute the structure inertial force.

Horizontal Inertia

From vertical weight.

$WTc_{all} = 193.10 \cdot kip$ $WTc_{pipe} = 8.10 \cdot kip$ $WTc_{curb} = 21.35 \cdot kip$

$arm_y_{all} = 3.48\ ft$ $arm_y_{pipe} = 3.52\ ft$ $arm_y_{curb} = 7.00\ ft$

$$armW_H := \frac{WTc_{all} \cdot arm_y_{all} - WTc_{pipe} \cdot arm_y_{pipe} + WTc_{curb} \cdot arm_y_{curb}}{WTc_{all} - WTc_{pipe} + WTc_{curb}} \qquad armW_H = 3.84\ ft$$

$$Inertia_{str_OBE} := k_{h_OBE} \cdot (WTc_{all} - WTc_{pipe} + WTc_{curb}) \qquad Inertia_{str_OBE} = 16.508 \cdot kip$$

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Eau Claire, WI

Buttress

Prepared By: TMR Nov 2020
Checked By: AEB Jan 2021

Stability

SUM OF VERTICAL FORCES

LC1 & LC2 $sumV_{1_2} := V_Wc$ $sumV_{1_2} = 206.35 \cdot kip$

LC3 & LC5 & LC6 $sumV_{3_5_6} := V_Wc + WTw_{NP} - UP_{NP}$ $sumV_{3_5_6} = 116.99 \cdot kip$

LC4 $sumV_4 := V_Wc + WTw_{Q100} - UP_{Q100}$ $sumV_4 = 116.99 \cdot kip$

LC7 $sumV_7 := V_Wc + WTw_{Q10000} - UP_{Q10000}$ $sumV_7 = 116.99 \cdot kip$

SUM OF HORIZONTAL FORCES

LC1 $sumH_1 := F_{s1m} + F_{surcharge}$ $sumH_1 = 54.38 \cdot kip$

LC2 $sumH_2 := F_{s1m} + Inertia_{str_OBE}$ $sumH_2 = 56.77 \cdot kip$

LC3 & LC4 & LC7 $sumH_{3_4_7} := F_{s1} - F_{s2}$ $sumH_{3_4_7} = 20.55 \cdot kip$

LC5 & LC6: N/A so not computed

RESISTING MOMENT (moments about point A)

$M_{R_conc} := MV_Wc$ $M_{R_conc} = 2159.6 \cdot kip \cdot ft$

$M_{R_water_NP} := MV_Ww_{NP}$ $M_{R_water_NP} = 4759.5 \cdot kip \cdot ft$

$M_{R_water_Q100} := MV_Ww_{Q100}$ $M_{R_water_Q100} = 4812.7 \cdot kip \cdot ft$

$M_{R_water_Q10000} := MV_Ww_{Q10000}$ $M_{R_water_Q10000} = 5397.5 \cdot kip \cdot ft$

$M_{R1_2} := M_{R_conc}$ $M_{R1_2} = 2159.6 \cdot kip \cdot ft$

$M_{R3_5_6} := M_{R_conc} + M_{R_water_NP}$ $M_{R3_5_6} = 6919.1 \cdot kip \cdot ft$

$M_{R4} := M_{R_conc} + M_{R_water_Q100}$ $M_{R4} = 6972.2 \cdot kip \cdot ft$

$M_{R7} := M_{R_conc} + M_{R_water_Q10000}$ $M_{R7} = 7557.1 \cdot kip \cdot ft$

Stability

OVERTURNING MOMENT (moments about point A) w/o Uplift

LC1 $M_{O_1} := F_{s1m} \cdot armF_{s1m} + MH_Sur$ $M_{O_1} = 186.9 \cdot kip \cdot ft$

LC2 $M_{O_2} := F_{s1m} \cdot armF_{s1m} + Inertia_{str_OBE} \cdot armW_H$ $M_{O_2} = 185.9 \cdot kip \cdot ft$

LC3 & LC4 & LC7 $M_{O_3_4_7} := F_{s1} \cdot armF_{s1} - F_{s2} \cdot armF_{s2}$ $M_{O_3_4_7} = 65.1 \cdot kip \cdot ft$

LC5 & LC6: N/A so not computed

SLIDING SAFETY FACTORS

For Load Cases 1, 2, 3, 4, and 7 (5 & 6 are N/A).

$$FS1_{sliding} := \frac{(sumV_{1_2}) \cdot \tan(\phi_{foundation})}{sumH_1} \quad FS1_{sliding} = 1.46 \quad Check(FS1_{sliding}, 1.3) = "OK"$$

$$FS2_{sliding} := \frac{(sumV_{1_2}) \cdot \tan(\phi_{foundation})}{sumH_2} \quad FS2_{sliding} = 1.40 \quad Check(FS2_{sliding}, 1.1) = "OK"$$

$$FS3_{sliding} := \frac{(sumV_{3_5_6}) \cdot \tan(\phi_{foundation})}{sumH_{3_4_7}} \quad FS3_{sliding} = 2.19 \quad Check(FS3_{sliding}, 1.7) = "OK"$$

$$FS4_{sliding} := \frac{(sumV_4) \cdot \tan(\phi_{foundation})}{sumH_{3_4_7}} \quad FS4_{sliding} = 2.19 \quad Check(FS4_{sliding}, 1.3) = "OK"$$

$$FS7_{sliding} := \frac{(sumV_7) \cdot \tan(\phi_{foundation})}{sumH_{3_4_7}} \quad FS7_{sliding} = 2.19 \quad Check(FS7_{sliding}, 1.1) = "OK"$$

Stability

BEARING CAPACITY

Ref. 5

R = Resultant from toe & e = eccentricity from base centroid

$R1 := \frac{M_{R1_2} - (M_{O_1})}{sumV_{1_2}}$ $R1 = 9.560\ ft$ $R2 := \frac{M_{R1_2} - (M_{O_2})}{sumV_{1_2}}$ $R2 = 9.565\ ft$

$e1 := \frac{Base}{2} - R1$ $e1 = -0.685\ ft$ $e2 := \frac{Base}{2} - R2$ $e2 = -0.690\ ft$

$R3 := \frac{M_{R3_5_6} - (M_{O_3_4_7} + M_UP_{NP})}{sumV_{3_5_6}}$ $R3 = 9.857\ ft$

$e3 := \frac{Base}{2} - R3$ $e3 = -0.982\ ft$

$R4 := \frac{M_{R4} - (M_{O_3_4_7} + M_UP_{Q100})}{sumV_4}$ $R4 = 9.857\ ft$ $R7 := \frac{M_{R7} - (M_{O_3_4_7} + M_UP_{Q10000})}{sumV_7}$ $R7 = 9.857\ ft$

$e4 := \frac{Base}{2} - R4$ $e4 = -0.982\ ft$ $e7 := \frac{Base}{2} - R7$ $e7 = -0.982\ ft$

where $\frac{Base}{3} = 5.917\ ft$ and $\frac{2 \cdot Base}{3} = 11.833\ ft$

$For1 := if\left[R1 > \left(\frac{1}{3} \cdot Base\right), if\left[R1 > \left(\frac{2}{3} \cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right]$ $For1 = \text{"In Kern"}$

$For2 := if\left[R2 > \left(\frac{1}{3} \cdot Base\right), if\left[R2 > \left(\frac{2}{3} \cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right]$ $For2 = \text{"In Kern"}$

$For3 := if\left[R3 > \left(\frac{1}{3} \cdot Base\right), if\left[R3 > \left(\frac{2}{3} \cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right]$ $For3 = \text{"In Kern"}$

$For4 := if\left[R4 > \left(\frac{1}{3} \cdot Base\right), if\left[R4 > \left(\frac{2}{3} \cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right]$ $For4 = \text{"In Kern"}$

$For7 := if\left[R7 > \left(\frac{1}{3} \cdot Base\right), if\left[R7 > \left(\frac{2}{3} \cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right]$ $For7 = \text{"In Kern"}$

Stability

Percent Base In Compression

Table 4-1, Page 4-5.

$$\%Base_Comp1 := if\left[For1 = "In\ Kern",\ 1,\ \left(\frac{3 \cdot R1}{Base}\right)\right] \qquad \%Base_Comp1 = 100.0 \cdot \% \qquad Check(\%Base_Comp1, 0.75) = "OK"$$

$$\%Base_Comp2 := if\left[For2 = "In\ Kern",\ 1,\ \left(\frac{3 \cdot R2}{Base}\right)\right] \qquad \%Base_Comp2 = 100.0 \cdot \% \qquad Check(\%Base_Comp2, 0.01) = "OK"$$

$$\%Base_Comp3 := if\left[For3 = "In\ Kern",\ 1,\ \left(\frac{3 \cdot R3}{Base}\right)\right] \qquad \%Base_Comp3 = 100.0 \cdot \% \qquad Check(\%Base_Comp3, 1.0) = "OK"$$

$$\%Base_Comp4 := if\left[For4 = "In\ Kern",\ 1,\ \left(\frac{3 \cdot R4}{Base}\right)\right] \qquad \%Base_Comp4 = 100.0 \cdot \% \qquad Check(\%Base_Comp4, 0.75) = "OK"$$

$$\%Base_Comp7 := if\left[For7 = "In\ Kern",\ 1,\ \left(\frac{3 \cdot R7}{Base}\right)\right] \qquad \%Base_Comp7 = 100.0 \cdot \% \qquad Check(\%Base_Comp7, 0.01) = "OK"$$

Heel Pressure (Equ. 3-1)

$$Press1_{Heel} := if\left[For1 = "In\ Kern",\ \left(\frac{sumV_{1_2}}{Base \cdot Width}\right) \cdot \left(1 - \frac{6 \cdot e1}{Base}\right),\ 0 \cdot psf\right] \qquad Press1_{Heel} = 1193 \cdot psf$$

$$Press2_{Heel} := if\left[For2 = "In\ Kern",\ \left(\frac{sumV_{1_2}}{Base \cdot Width}\right) \cdot \left(1 - \frac{6 \cdot e2}{Base}\right),\ 0 \cdot psf\right] \qquad Press2_{Heel} = 1195 \cdot psf$$

$$Press3_{Heel} := if\left[For3 = "In\ Kern",\ \left(\frac{sumV_{3_5_6}}{Base \cdot Width}\right) \cdot \left(1 - \frac{6 \cdot e3}{Base}\right),\ 0 \cdot psf\right] \qquad Press3_{Heel} = 732 \cdot psf$$

$$Press4_{Heel} := if\left[For4 = "In\ Kern",\ \left(\frac{sumV_{4}}{Base \cdot Width}\right) \cdot \left(1 - \frac{6 \cdot e4}{Base}\right),\ 0 \cdot psf\right] \qquad Press4_{Heel} = 732 \cdot psf$$

$$Press7_{Heel} := if\left[For7 = "In\ Kern",\ \left(\frac{sumV_{7}}{Base \cdot Width}\right) \cdot \left(1 - \frac{6 \cdot e7}{Base}\right),\ 0 \cdot psf\right] \qquad Press7_{Heel} = 732 \cdot psf$$

Stability

Toe Pressure (Equ. 3-1 and 3-2)

$$Press1_{Toe} := if\left[For1 = "In\ Kern",\left(\frac{sumV_{1_2}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e1}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{1_2}}{(Base-2\cdot e1)\cdot Width}\right]\right] \qquad Press1_{Toe} = 744\cdot psf$$

$$Press2_{Toe} := if\left[For1 = "In\ Kern",\left(\frac{sumV_{1_2}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e2}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{1_2}}{(Base-2\cdot e2)\cdot Width}\right]\right] \qquad Press2_{Toe} = 743\cdot psf$$

$$Press3_{Toe} := if\left[For3 = "In\ Kern",\left(\frac{sumV_{3_5_6}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e3}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{3_5_6}}{(Base-2\cdot e3)\cdot Width}\right]\right] \qquad Press3_{Toe} = 367\cdot psf$$

$$Press4_{Toe} := if\left[For4 = "In\ Kern",\left(\frac{sumV_{4}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e4}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{4}}{(Base-2\cdot e4)\cdot Width}\right]\right] \qquad Press4_{Toe} = 367\cdot psf$$

$$Press7_{Toe} := if\left[For7 = "In\ Kern",\left(\frac{sumV_{7}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e7}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{7}}{(Base-2\cdot e7)\cdot Width}\right]\right] \qquad Press7_{Toe} = 367\cdot psf$$

Base Pressure Length (Equ. 3-3)

$$Press1_{Length} := if\left[For1 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e1)\right] \qquad Press1_{Length} = 17.750\ ft$$

$$Press2_{Length} := if\left[For2 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e2)\right] \qquad Press2_{Length} = 17.750\ ft$$

$$Press3_{Length} := if\left[For3 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e3)\right] \qquad Press3_{Length} = 17.750\ ft$$

$$Press4_{Length} := if\left[For4 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e4)\right] \qquad Press4_{Length} = 17.750\ ft$$

$$Press7_{Length} := if\left[For7 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e7)\right] \qquad Press7_{Length} = 17.750\ ft$$

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Buttress

Prepared By: TMR Nov 2020
Checked By: AEB Jan 2021

Stability

Summary

$BasePressure1 := \max(Press1_{Heel}, Press1_{Toe})$ $BasePressure1 = 1193 \cdot psf$

$Check(BearingCap_{allow}, BasePressure1) = "OK"$

$BasePressure2 := \max(Press2_{Heel}, Press2_{Toe})$ $BasePressure2 = 1195 \cdot psf$

$Check(BearingCap_{allow_eq}, BasePressure2) = "OK"$

$BasePressure3 := \max(Press3_{Heel}, Press3_{Toe})$ $BasePressure3 = 732 \cdot psf$

$Check(BearingCap_{allow}, BasePressure3) = "OK"$

$BasePressure4 := \max(Press4_{Heel}, Press4_{Toe})$ $BasePressure4 = 732 \cdot psf$

$Check(BearingCap_{allow}, BasePressure4) = "OK"$

$BasePressure7 := \max(Press7_{Heel}, Press7_{Toe})$ $BasePressure7 = 732 \cdot psf$

$Check(BearingCap_{allow}, BasePressure7) = "OK"$

Stability Summary

SUMMARY

LC 1 - Unusual: Construction
LC 2 - Extreme: Construction with OBE
LC 3 - Usual: Normal Pool
LC 4 - Unusual: Infrequent Flood = Q100
LC 5 & LC 6: N/A
LC 7 - Extreme: Maximum Design Flood = Q10000

SLIDING SAFETY FACTORS

$FS1_{sliding} = 1.46$ Check$(FS1_{sliding}, 1.3) =$ "OK"

$FS2_{sliding} = 1.40$ Check$(FS2_{sliding}, 1.1) =$ "OK"

$FS3_{sliding} = 2.19$ Check$(FS3_{sliding}, 1.7) =$ "OK"

$FS4_{sliding} = 2.19$ Check$(FS4_{sliding}, 1.3) =$ "OK"

$FS7_{sliding} = 2.19$ Check$(FS7_{sliding}, 1.1) =$ "OK"

FLOTATION SAFETY FACTORS

-- only LC 3, 4, and 7 are applicable

$SF3_{fl} = 2.31$ Check$(SF3_{fl}, 1.3) =$ "OK"

$SF4_{fl} = 2.31$ Check$(SF4_{fl}, 1.2) =$ "OK"

$SF7_{fl} = 2.31$ Check$(SF7_{fl}, 1.1) =$ "OK"

BEARING CAPACITY

%Base_Comp1 = 100.0·% Check(%Base_Comp1, 0.75) = "OK"

%Base_Comp2 = 100.0·% Check(%Base_Comp2, 0.01) = "OK"

%Base_Comp3 = 100.0·% Check(%Base_Comp3, 1.0) = "OK"

%Base_Comp4 = 100.0·% Check(%Base_Comp4, 0.75) = "OK"

%Base_Comp7 = 100.0·% Check(%Base_Comp7, 0.01) = "OK"

BasePressure1 = 1193·psf Check$(BearingCap_{allow}, BasePressure1) =$ "OK"

BasePressure2 = 1195·psf Check$(BearingCap_{allow_eq}, BasePressure2) =$ "OK"

BasePressure3 = 732·psf Check$(BearingCap_{allow_eq}, BasePressure3) =$ "OK"

BasePressure4 = 732·psf Check$(BearingCap_{allow}, BasePressure4) =$ "OK"

BasePressure7 = 732·psf Check$(BearingCap_{allow}, BasePressure7) =$ "OK"

Concrete

TOP 'CURB' REINFORCEMENT DESIGN

- Two Mats in Both Directions

Temperature Steel Requirement

Reinforcement is ASTM A615, Grade 60.
COE (Ref. 2, Section 2.6): 12" center-to-center spacing of both primary and secondary bars.
COE (Ref. 2, Sections 2.9.1 and 2.9.2): 0.003 for Grade 60, with maximum area equivalent to #9 @ 12" in each face

$$A_{temp_COE} := \frac{0.003 \cdot (12 \cdot in) \cdot (18 \cdot in)}{2} \qquad A_{temp_COE} = 0.324 \cdot in^2$$

#4 @ 7" OC => A = 0.337 in^2
#5 @ 11" OC => A = 0.335 in^2
#6 @ 12" OC => A = 0.442 in^2 <-- USE

ELSE REINFORCEMENT DESIGN

- One mat in vertical and horizontal direction on each exposed face.

Temperature Steel Requirement

Reinforcement is ASTM A615, Grade 60.
COE (Ref. 2, Section 2.6): 12" center-to-center spacing of both primary and secondary bars.
COE (Ref. 2, Sections 2.9.1 and 2.9.2): 0.003 for Grade 60, with maximum area equivalent to #9 @ 12" in each face

#9@12" provides 1.00 in^2. Bars are large and require large development, hooks, etc.

--> Use #7@7" OC (1.03 in^2)

CONCRETE DAM (RETAINING WALL)
HOG CHUTE DAM - MESA COUNTY, COLORADO

Overflow Spillway

Ref.:
1) *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2) *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14).*
3) EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, USACOE, 30 Nov. 2016.
4) EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACOE, 1 December 2005.
5) EM 1110-2-2200, *Gravity Dam Design*, USACOE, 30 June 1995.
6) EM 1110-2-2502, *Retaining and Flood Walls*, USACOE, 29 September 1989.
7) Letter: Geotechnical Design Parameters, Carson Lake, Mesa County, Colorado, from Michael A. Berry, PE, Huddleston-Berry, 27 August 2020 & 15 September 2020 email.
8) Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.

Design

1) Load Cases Per Ref. 4, Table B-1. Do not compute construction conditions (1 and 4) as independent upstream cofferdam is required to retain water during construction. In addition, backfilled evenly.
   LC 1 - Construction & LC 4 - Construction with OBE --> N/A
   LC 2 - Usual: Normal Pool
   LC 3 - Unusual: Infrequent Flood = Q100
   LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
   LC 6 - Extreme: Coincident Pool = Normal Pool with MDE
   LC 7 - Extreme: Maximum Design Flood = Q10000

2) Required Sliding Factors of Safety (Ref. 4, Table 3-2) and Resultant Location (Ref. 4, Table 3-5)
   Usual = 1.7 w/ 100% Base in Compression
   Unusual = 1.3 w/ 75% Base in Compression
   Extreme = 1.1 w/ Resultant In Base

3) Ref. 7: Wall will deflect sufficiently to cause active earth pressure conditions to develop.
   Mononobe-Okabe theory in conjunction with the horizontal seismic coefficient.
   Site PGA = 0.236. k_h is defined as 1/3 of the site modified PGA so k_h = 0.08 for this site.
   Zero vertical earthquake acceleration.

4) Upstream backfill always saturated with reservoir. Downstream backfill always moist due to lack of tailwater for all load cases. Flowing water will not completely saturate downstream side.
   Downstream backfill is armored, but assume top 3.0 feet is ineffective.

5) Soil parameters are the same and soil pressure wedges fully develop on both sides of the wall.

6) Per Ref. 8:
   Normal Pool El. = 9895.00 feet
   Q10 Pool El. = 9895.45 feet
   Q100 Pool El. = 9897.16 feet
   Q10000 Pool El. = 9900.40 feet

7) Crest set at normal pool El. 9895.0 feet. Elevations in NAVD88.

8) Concrete load cases per Ref. 3 and design per Refs. 3 and 4.

Variables

Variables

Usual case when k_v, δ, and θ are zero.

Concrete Unit Weight
Ref. 5, Section 3-3b
$\gamma_c := 150 \cdot pcf$

Water Unit Weight
Ref. 5, Section 3-3c
$\gamma_w := 62.5 \cdot pcf$

Soil Moist Unit Weight
Ref. 7
$\gamma_m := 125 \cdot pcf$

Inclination of Soil Surface
(0 if level backfill)
$\beta := 0 \cdot deg$

Soil Saturated Unit Weight
Ref. 7
$\gamma_s := 130 \cdot pcf$

Soil Buoyant Unit Weight
$\gamma_b := \gamma_s - \gamma_w$
$\gamma_b = 67.5 \cdot pcf$

Backfill Friction Angle
Ref. 7
$\phi := 32 \cdot deg$

Foundation Friction Angle
Ref. 7
$\phi_{foundation} := 21 \cdot deg$

Structure Height
$h := 8.0 \cdot ft$

Normal Water Height
$h_w := h$

Horizontal
Acceleration (in g's)
Ref. 7
$k_{h_OBE} := 0.08$
$k_{h_MDE} := 0.277$

Flood Water Heights
$h_{wf_IF} := h + 2.16 \cdot ft$
$h_{wf_MDF} := h + 5.40 \cdot ft$

Seismic Inertia Angle
(Page G-3)
$\psi_{OBE} := atan(k_{h_OBE})$ $\psi_{OBE} = 4.57 \cdot deg$
$\psi_{MDE} := atan(k_{h_MDE})$ $\psi_{MDE} = 15.48 \cdot deg$

Footing Thickness
$t_{ftg} := 1 \cdot ft + 9 \cdot in$

Stem Thickness
$t_{stem} := 1 \cdot ft + 6 \cdot in$

Heel Soil Height
(soil to stem top)
$h_{s1} := h - t_{ftg}$
$h_{s1} = 6.25\ ft$

Heel Width
$b_{heel} := 4 \cdot ft + 0 \cdot in$

Toe Width
$b_{toe} := 4 \cdot ft + 0 \cdot in$

Total Base
$base := b_{heel} + t_{stem} + b_{toe}$
$base = 9.50\ ft$

Toe Soil Height
(3.0' below stem)
$[h_{s2} \geq 0']$
$h_{s2} := h - t_{ftg} - 3.0 \cdot ft$
$h_{s2} = 3.25\ ft$

Analyze per
foot of width
$b := 12 \cdot in$

Allowable Bearing Capacity
Ref. 7
$BearingCap_{allow} := 2000 \cdot psf$
$BearingCap_{allow_eq} := 2666 \cdot psf$

Variables

Active Earth Pressure Coefficient

$$K_a := \frac{\cos(\phi)^2}{\left(1 + \sqrt{\frac{\sin(\phi)\cdot\sin(\phi-\beta)}{\cos(\beta)}}\right)^2} \qquad K_a = 0.307$$

Passive Earth Pressure Coefficient

$$K_p := \frac{\cos(\phi)^2}{\left(1 - \sqrt{\frac{\sin(\phi)\cdot\sin(\phi-\beta)}{\cos(\beta)}}\right)^2} \qquad K_p = 3.255$$

Concrete Cover: $c := 3\cdot in$ (Ref. 3, Table 2-1)

Resistance Factors (Ref. 2, Table 21.2.1)

$\phi M := 0.90$ $\phi V := 0.75$

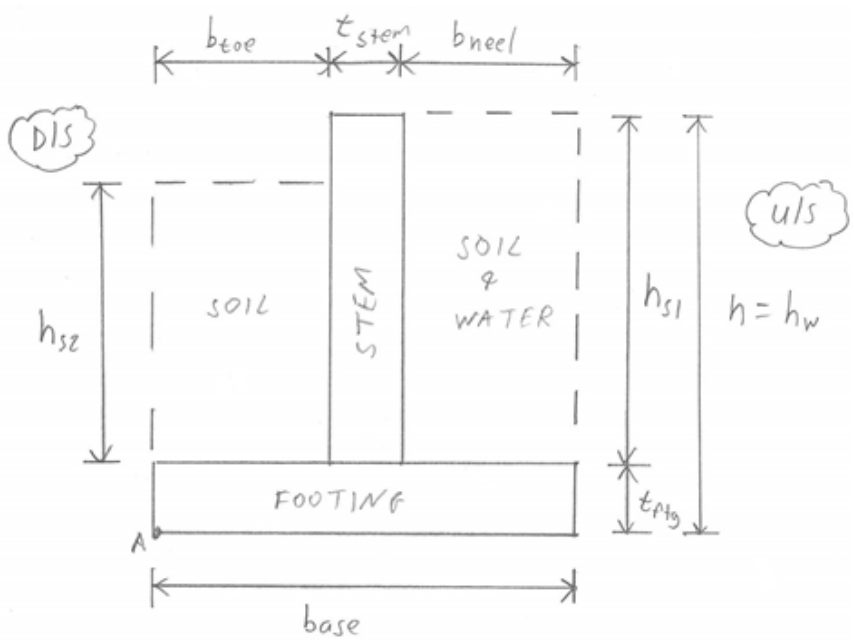
Concrete Compressive Strength: $f'_c := 4000\cdot psi$

Reinforcement Yield Strength: $f_y := 60000\cdot psi$ [ASTM A615, Grade 60]

Ref. 2, 22.2.2.4.3

$$\beta_1 := \text{if}\left[f'_c > 4000\cdot psi, \text{if}\left[\left(0.85 - 0.05\cdot\frac{f'_c - 4000\cdot psi}{1000\cdot psi}\right) > 0.65, \left(0.85 - 0.05\cdot\frac{f'_c - 4000\cdot psi}{1000\cdot psi}\right), 0.65\right], 0.85\right] \qquad \beta_1 = 0.850$$

Define Check Function: $Check(x,y) := \text{if}(x \geq y, \text{"OK"}, \text{"NG"})$



Stability

VERTICAL FORCES (arms from point A)

Concrete:

$W_{c1} := b \cdot \gamma_c \cdot (h - t_{ftg}) \cdot t_{stem}$ $W_{c1} = 1.406 \cdot kip$ $armW_{c1} := b_{toe} + \frac{t_{stem}}{2}$ $armW_{c1} = 4.750\ ft$

$W_{c2} := b \cdot \gamma_c \cdot base \cdot t_{ftg}$ $W_{c2} = 2.494 \cdot kip$ $armW_{c2} := \frac{base}{2}$ $armW_{c2} = 4.750\ ft$

Soil:

$W_{s1} := b \cdot (\gamma_s - \gamma_w) \cdot b_{heel} \cdot h_{s1}$ $W_{s1} = 1.687 \cdot kip$ $armW_{s1} := b_{toe} + \frac{t_{stem}}{2} + \frac{b_{heel}}{2}$ $armW_{s1} = 6.750\ ft$

$W_{s2} := b \cdot \gamma_m \cdot b_{toe} \cdot h_{s2}$ $W_{s2} = 1.625 \cdot kip$ $armW_{s2} := \frac{b_{toe}}{2}$ $armW_{s2} = 2.000\ ft$

Water: only water within soil (ignore above soil during high water)

$Ww_{h1} := b \cdot \gamma_w \cdot b_{heel} \cdot h_{s1}$ $Ww_{h1} = 1.563 \cdot kip$ $armWw_{h1} := base - \frac{b_{heel}}{2}$ $armWw_{h1} = 7.500\ ft$

Uplift:

$UP := \frac{1}{2} \cdot b \cdot \gamma_w \cdot base \cdot h_w$ $UP = 2.375 \cdot kip$ $armUP := \frac{2}{3} \cdot base$ $armUP = 6.333\ ft$

$M_{UP} := UP \cdot armUP$ $M_{UP} = 15.04 \cdot kip \cdot ft$

$UP_{f_IF} := \frac{1}{2} \cdot b \cdot \gamma_w \cdot base \cdot h_{wf_IF}$ $UP_{f_IF} = 3.016 \cdot kip$

$M_{UP_f_IF} := UP_{f_IF} \cdot armUP$ $M_{UP_f_IF} = 19.10 \cdot kip \cdot ft$

$UP_{f_MDF} := \frac{1}{2} \cdot b \cdot \gamma_w \cdot base \cdot h_{wf_MDF}$ $UP_{f_MDF} = 3.978 \cdot kip$

$M_{UP_f_MDF} := UP_{f_MDF} \cdot armUP$ $M_{UP_f_MDF} = 25.19 \cdot kip \cdot ft$

Stability

HORIZONTAL FORCES

Soil:

$F_{s1} := \frac{1}{2} \cdot b \cdot K_a \cdot (\gamma_s - \gamma_w) \cdot (h_{s1} + t_{ftg})^2$ $F_{s1} = 0.664 \cdot kip$ $armF_{s1} := \frac{h_{s1} + t_{ftg}}{3}$ $armF_{s1} = 2.667\ ft$

$F_{s2} := \frac{1}{2} \cdot b \cdot K_p \cdot \gamma_m \cdot (h_{s2} + t_{ftg})^2$ $F_{s2} = 5.085 \cdot kip$ $armF_{s2} := \frac{h_{s2} + t_{ftg}}{3}$ $armF_{s2} = 1.667\ ft$

Water:

$Fw_{h1} := \frac{1}{2} \cdot b \cdot \gamma_w \cdot h_w^2$ $Fw_{h1} = 2.000 \cdot kip$ $armFw_{h1} := \frac{h_w}{3}$ $armFw_{h1} = 2.667\ ft$

$Fw_{h1_f_IF_A} := \frac{1}{2} \cdot b \cdot \gamma_w \cdot h_{wf_IF}^2$ $Fw_{h1_f_IF_A} = 3.226 \cdot kip$ $armFw_{h1_f_IF_A} := \frac{h_{wf_IF}}{3}$ $armFw_{h1_f_IF_A} = 3.387\ ft$

$Fw_{h1_f_IF_B} := \frac{1}{2} \cdot b \cdot \gamma_w \cdot \left[h_{wf_IF} - (h_{s1} + t_{ftg})\right]^2$ $armFw_{h1_f_IF_B} := (h_{s1} + t_{ftg}) + \frac{h_{wf_IF} - (h_{s1} + t_{ftg})}{3}$

$Fw_{h1_f_IF_B} = 0.146 \cdot kip$ $armFw_{h1_f_IF_B} = 8.720\ ft$

$Fw_{h1_f_IF} := Fw_{h1_f_IF_A} - Fw_{h1_f_IF_B}$ $Fw_{h1_f_IF} = 3.080 \cdot kip$

$armFw_{h1_f_IF} := \frac{Fw_{h1_f_IF_A} \cdot armFw_{h1_f_IF_A} - Fw_{h1_f_IF_B} \cdot armFw_{h1_f_IF_B}}{Fw_{h1_f_IF}}$ $armFw_{h1_f_IF} = 3.134\ ft$

$Fw_{h1_f_MDF_A} := \frac{1}{2} \cdot b \cdot \gamma_w \cdot h_{wf_MDF}^2$ $armFw_{h1_f_MDF_A} := \frac{h_{wf_MDF}}{3}$

$Fw_{h1_f_MDF_A} = 5.611 \cdot kip$ $armFw_{h1_f_MDF_A} = 4.467\ ft$

$Fw_{h1_f_MDF_B} := \frac{1}{2} \cdot b \cdot \gamma_w \cdot \left[h_{wf_MDF} - (h_{s1} + t_{ftg})\right]^2$ $armFw_{h1_f_MDF_B} := (h_{s1} + t_{ftg}) + \frac{h_{wf_MDF} - (h_{s1} + t_{ftg})}{3}$

$Fw_{h1_f_MDF_B} = 0.911 \cdot kip$ $armFw_{h1_f_MDF_B} = 9.800\ ft$

$Fw_{h1_f_MDF} := Fw_{h1_f_MDF_A} - Fw_{h1_f_MDF_B}$ $Fw_{h1_f_MDF} = 4.700 \cdot kip$

$armFw_{h1_f_MDF} := \frac{Fw_{h1_f_MDF_A} \cdot armFw_{h1_f_MDF_A} - Fw_{h1_f_MDF_B} \cdot armFw_{h1_f_MDF_B}}{Fw_{h1_f_MDF}}$ $armFw_{h1_f_MDF} = 3.433\ ft$

Stability

SUM OF VERTICAL FORCES

LC2 $sumV := (W_{c1} + W_{c2}) + (W_{s1} + W_{s2}) + (Ww_{h1}) - (UP)$ $sumV = 6.400 \cdot kip$

LC3 $sumV_{f_IF} := (W_{c1} + W_{c2}) + (W_{s1} + W_{s2}) + (Ww_{h1}) - (UP_{f_IF})$ $sumV_{f_IF} = 5.759 \cdot kip$

LC7 $sumV_{f_MDF} := (W_{c1} + W_{c2}) + (W_{s1} + W_{s2}) + (Ww_{h1}) - (UP_{f_MDF})$ $sumV_{f_MDF} = 4.797 \cdot kip$

SUM OF HORIZONTAL FORCES

If sumH < 0, then adjust force in F_{s2} as a triangle shape so sumH = 0.

LC2 $F_{s2_new} := if[(F_{s1} + Fw_{h1}) - (F_{s2}) < 0 \cdot kip, F_{s1} + Fw_{h1}, (F_{s2})]$ $F_{s2_new} = 2.664 \cdot kip$

$sumH := (F_{s1} + Fw_{h1}) - (F_{s2_new})$ $sumH = 0.000 \cdot kip$

LC3 $F_{s2_f_IF_new} := if[(F_{s1} + Fw_{h1_f_IF}) - (F_{s2}) < 0 \cdot kip, F_{s1} + Fw_{h1_f_IF}, (F_{s2})]$ $F_{s2_f_IF_new} = 3.744 \cdot kip$

$sumH_{f_IF} := (F_{s1} + Fw_{h1_f_IF}) - (F_{s2_f_IF_new})$ $sumH_{f_IF} = 0.000 \cdot kip$

LC7 $F_{s2_f_MDF_new} := if[(F_{s1} + Fw_{h1_f_MDF}) - (F_{s2}) < 0 \cdot kip, F_{s1} + Fw_{h1_f_MDF}, (F_{s2})]$ $F_{s2_f_MDF_new} = 5.085 \cdot kip$

$sumH_{f_MDF} := (F_{s1} + Fw_{h1_f_MDF}) - (F_{s2_f_MDF_new})$ $sumH_{f_MDF} = 0.278 \cdot kip$

RESISTING MOMENT (moments about point A)

$M_{R_conc} := W_{c1} \cdot (armW_{c1}) + W_{c2} \cdot (armW_{c2})$ $M_{R_conc} = 18.525 \cdot kip \cdot ft$

$M_{R_soil} := W_{s1} \cdot (armW_{s1}) + W_{s2} \cdot (armW_{s2})$ $M_{R_soil} = 14.641 \cdot kip \cdot ft$

$M_{R_water} := Ww_{h1} \cdot (armWw_{h1})$ $M_{R_water} = 11.719 \cdot kip \cdot ft$

$M_R := M_{R_conc} + M_{R_soil} + M_{R_water}$ $M_R = 44.884 \cdot kip \cdot ft$

OVERTURNING MOMENT (moments about point A) w/o Uplift

LC2 $M_O := [F_{s1} \cdot (armF_{s1}) - F_{s2_new} \cdot (armF_{s2})] + Fw_{h1} \cdot (armFw_{h1})$ $M_O = 2.664 \cdot kip \cdot ft$

LC3 $M_{O_f_IF} := [F_{s1} \cdot (armF_{s1}) - F_{s2_f_IF_new} \cdot (armF_{s2})] + Fw_{h1_f_IF} \cdot (armFw_{h1_f_IF})$ $M_{O_f_IF} = 5.184 \cdot kip \cdot ft$

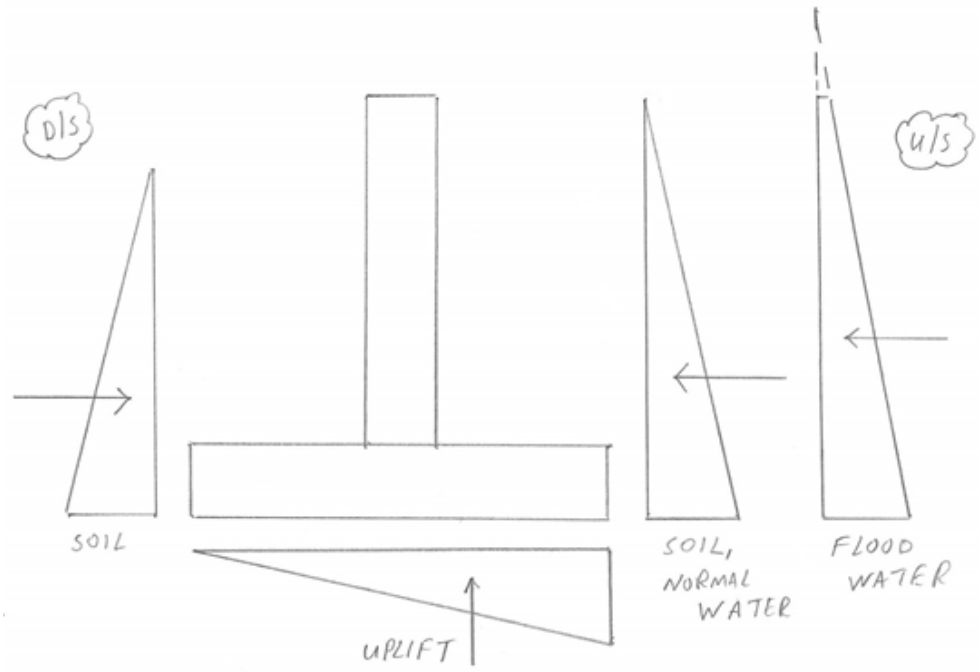
LC7 $M_{O_f_MDF} := [F_{s1} \cdot (armF_{s1}) - F_{s2_f_MDF_new} \cdot (armF_{s2})] + Fw_{h1_f_MDF} \cdot (armFw_{h1_f_MDF})$ $M_{O_f_MDF} = 9.428 \cdot kip \cdot ft$

Ayres Associates Inc
Eau Claire, WI

Concrete Spillway (Wall)

Prepared By: TMR Oct 2020
Checked By: AEB Oct 2020

Stability



SLIDING SAFETY FACTORS

For Load Cases 2, 3, and 7

$$FS2_{sliding} := \text{if}\left[sumH = 0\cdot kip, \text{"Stable"}, \frac{(sumV)\cdot\tan(\phi_{foundation})}{sumH}\right] \qquad FS2_{sliding} = \text{"Stable"}$$

$$FS2_{SL} := \text{if}\left(FS2_{sliding} = \text{"Stable"}, 100, FS2_{sliding}\right) \qquad \text{Check}\left(FS2_{SL}, 1.7\right) = \text{"OK"}$$

$$FS3_{sliding} := \text{if}\left[sumH_{f_IF} = 0\cdot kip, \text{"Stable"}, \frac{(sumV_{f_IF})\cdot\tan(\phi_{foundation})}{sumH_{f_IF}}\right] \qquad FS3_{sliding} = \text{"Stable"}$$

$$FS3_{SL} := \text{if}\left(FS3_{sliding} = \text{"Stable"}, 100, FS3_{sliding}\right) \qquad \text{Check}\left(FS3_{SL}, 1.3\right) = \text{"OK"}$$

$$FS7_{sliding} := \text{if}\left[sumH_{f_MDF} = 0\cdot kip, \text{"Stable"}, \frac{(sumV_{f_MDF})\cdot\tan(\phi_{foundation})}{sumH_{f_MDF}}\right] \qquad FS7_{sliding} = 6.61$$

$$FS7_{SL} := \text{if}\left(FS7_{sliding} = \text{"Stable"}, 100, FS7_{sliding}\right) \qquad \text{Check}\left(FS7_{SL}, 1.1\right) = \text{"OK"}$$

Stability

BEARING CAPACITY

Ref. 6

R = Resultant from toe & e = eccentricity from base centroid

$R2 := \frac{M_R - (M_O + M_{UP})}{sumV}$ $R2 = 4.247\ ft$

$R3 := \frac{M_R - (M_{O_f_IF} + M_{UP_f_IF})}{sumV_{f_IF}}$ $R3 = 3.577\ ft$

$R7 := \frac{M_R - (M_{O_f_MDF} + M_{UP_f_MDF})}{sumV_{f_MDF}}$ $R7 = 2.139\ ft$

$e2 := \frac{base}{2} - R2$ $e2 = 0.503\ ft$

$e3 := \frac{base}{2} - R3$ $e3 = 1.173\ ft$

$e7 := \frac{base}{2} - R7$ $e7 = 2.611\ ft$

where $\frac{base}{3} = 3.167\ ft$ and $\frac{2 \cdot base}{3} = 6.333\ ft$

$For2 := if\left[R2 > \left(\frac{1}{3} \cdot base\right), if\left[R2 > \left(\frac{2}{3} \cdot base\right), "Not\ In\ Kern", "In\ Kern"\right], "Not\ In\ Kern"\right]$ $For2 = "In\ Kern"$

$For3 := if\left[R3 > \left(\frac{1}{3} \cdot base\right), if\left[R3 > \left(\frac{2}{3} \cdot base\right), "Not\ In\ Kern", "In\ Kern"\right], "Not\ In\ Kern"\right]$ $For3 = "In\ Kern"$

$For7 := if\left[R7 > \left(\frac{1}{3} \cdot base\right), if\left[R7 > \left(\frac{2}{3} \cdot base\right), "Not\ In\ Kern", "In\ Kern"\right], "Not\ In\ Kern"\right]$ $For7 = "Not\ In\ Kern"$

Percent Base In Compression

Table 4-1, Page 4-5.

$\%Base_Comp2 := if\left[For2 = "In\ Kern", 1, \left(\frac{3 \cdot R2}{base}\right)\right]$ $\%Base_Comp2 = 100.0 \cdot \%$ $Check(\%Base_Comp2, 1) = "OK"$

$\%Base_Comp3 := if\left[For3 = "In\ Kern", 1, \left(\frac{3 \cdot R3}{base}\right)\right]$ $\%Base_Comp3 = 100.0 \cdot \%$ $Check(\%Base_Comp3, 0.75) = "OK"$

$\%Base_Comp7 := if\left[For7 = "In\ Kern", 1, \left(\frac{3 \cdot R7}{base}\right)\right]$ $\%Base_Comp7 = 67.6 \cdot \%$ $Check(\%Base_Comp7, 0.01) = "OK"$

Stability

Heel Pressure (Equ. 3-1)

$$Press2_{Heel} := if\left[For2 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1-\frac{6\cdot e2}{base}\right),0\cdot psf\right] \qquad Press2_{Heel} = 460\cdot psf$$

$$Press3_{Heel} := if\left[For3 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1-\frac{6\cdot e3}{base}\right),0\cdot psf\right] \qquad Press3_{Heel} = 174\cdot psf$$

$$Press7_{Heel} := if\left[For7 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1-\frac{6\cdot e7}{base}\right),0\cdot psf\right] \qquad Press7_{Heel} = 0\cdot psf$$

Toe Pressure (Equ. 3-1 and 3-2)

$$Press2_{Toe} := if\left[For2 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1+\frac{6\cdot e2}{base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV}{(base-2\cdot e2)\cdot b}\right]\right] \qquad Press2_{Toe} = 888\cdot psf$$

$$Press3_{Toe} := if\left[For3 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1+\frac{6\cdot e3}{base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV}{(base-2\cdot e3)\cdot b}\right]\right] \qquad Press3_{Toe} = 1173\cdot psf$$

$$Press7_{Toe} := if\left[For7 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1+\frac{6\cdot e7}{base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV}{(base-2\cdot e7)\cdot b}\right]\right] \qquad Press7_{Toe} = 1994\cdot psf$$

Base Pressure Length (Equ. 3-3)

$$Press2_{Length} := if\left[For2 = "In\ Kern",base,\left(\frac{3}{2}\right)\cdot(base-2\cdot e2)\right] \qquad Press2_{Length} = 9.500\,ft$$

$$Press3_{Length} := if\left[For3 = "In\ Kern",base,\left(\frac{3}{2}\right)\cdot(base-2\cdot e3)\right] \qquad Press3_{Length} = 9.500\,ft$$

$$Press7_{Length} := if\left[For7 = "In\ Kern",base,\left(\frac{3}{2}\right)\cdot(base-2\cdot e7)\right] \qquad Press7_{Length} = 6.418\,ft$$

Summary

$$BasePressure2 := \max\left(Press2_{Heel},Press2_{Toe}\right) \qquad BasePressure2 = 888\cdot psf$$

$$Check\left(BearingCap_{allow},BasePressure2\right) = "OK"$$

$$BasePressure3 := \max\left(Press3_{Heel},Press3_{Toe}\right) \qquad BasePressure3 = 1173\cdot psf$$

$$Check\left(BearingCap_{allow},BasePressure3\right) = "OK"$$

$$BasePressure7 := \max\left(Press7_{Heel},Press7_{Toe}\right) \qquad BasePressure7 = 1994\cdot psf$$

$$Check\left(BearingCap_{allow},BasePressure7\right) = "OK"$$

Earthquake for Load Cases 5 and 6

Stability

Mononobe-Okabe Analysis (Ref. 4, Appendix G)

Normal Pool. Usual case when k_v, δ, and θ are zero.

$\text{Check}(\tan(\phi - \beta), k_{h_OBE}) = \text{"OK"}$ $\text{Check}(\tan(\phi - \beta), k_{h_MDE}) = \text{"OK"}$ Driving Wedge Limit (Equ. G-7)

$\text{Check}(\tan(\phi + \beta), k_{h_OBE}) = \text{"OK"}$ $\text{Check}(\tan(\phi + \beta), k_{h_MDE}) = \text{"OK"}$ Resisting Wedge Limit (Equ. G-8)

Note: c1 and c2 are the same for both active and passive since equal backfill ϕ

$$c_{1_OBE} := \frac{2\cdot(\tan(\phi) - k_{h_OBE})}{1 + k_{h_OBE}\cdot\tan(\phi)} \qquad c_{1_OBE} = 1.038$$

$$c_{1_MDE} := \frac{2\cdot(\tan(\phi) - k_{h_MDE})}{1 + k_{h_MDE}\cdot\tan(\phi)} \qquad c_{1_MDE} = 0.593$$

(Equ. G-14 & G-17)

$$c_{2_OBE} := \frac{\tan(\phi)\cdot(1 - \tan(\phi)\cdot\tan(\beta)) - (\tan(\beta) + k_{h_OBE})}{\tan(\phi)\cdot(1 + k_{h_OBE}\cdot\tan(\phi))} \qquad c_{2_OBE} = 0.830$$

$$c_{2_MDE} := \frac{\tan(\phi)\cdot(1 - \tan(\phi)\cdot\tan(\beta)) - (\tan(\beta) + k_{h_MDE})}{\tan(\phi)\cdot(1 + k_{h_MDE}\cdot\tan(\phi))} \qquad c_{2_MDE} = 0.475$$

(Equ. G-15 & G-18)

Active

$$\alpha_{A_OBE} := \operatorname{atan}\left(\frac{c_{1_OBE} + \sqrt{c_{1_OBE}^2 + 4\cdot c_{2_OBE}}}{2}\right) \qquad \alpha_{A_OBE} = 57.47\cdot\text{deg}$$

$$\alpha_{A_MDE} := \operatorname{atan}\left(\frac{c_{1_MDE} + \sqrt{c_{1_MDE}^2 + 4\cdot c_{2_MDE}}}{2}\right) \qquad \alpha_{A_MDE} = 46.30\cdot\text{deg}$$

(Equ. G-13)

Passive

$$\alpha_{P_OBE} := \operatorname{atan}\left(\frac{-c_{1_OBE} + \sqrt{c_{1_OBE}^2 + 4\cdot c_{2_OBE}}}{2}\right) \qquad \alpha_{P_OBE} = 27.91\cdot\text{deg}$$

$$\alpha_{P_MDE} := \operatorname{atan}\left(\frac{-c_{1_MDE} + \sqrt{c_{1_MDE}^2 + 4\cdot c_{2_MDE}}}{2}\right) \qquad \alpha_{P_MDE} = 24.39\cdot\text{deg}$$

(Equ. G-16)

--> G-3 Equations and Diagrams for Cohesionless Backfill with and w/o Water Table -> k_v, δ, and θ are zero.

Stability

(c) for driving and (b) for resisting

(1) Driving Force

- static

$$K_{A_OBE} := \left(\frac{1 - \tan(\phi)\cdot\cot(\alpha_{A_OBE})}{1 + \tan(\phi)\cdot\tan(\alpha_{A_OBE})}\right)\cdot\left(\frac{\tan(\alpha_{A_OBE})}{\tan(\alpha_{A_OBE}) - \tan(\beta)}\right) \qquad K_{A_OBE} = 0.304$$

$$K_{A_MDE} := \left(\frac{1 - \tan(\phi)\cdot\cot(\alpha_{A_MDE})}{1 + \tan(\phi)\cdot\tan(\alpha_{A_MDE})}\right)\cdot\left(\frac{\tan(\alpha_{A_MDE})}{\tan(\alpha_{A_MDE}) - \tan(\beta)}\right) \qquad K_{A_MDE} = 0.244$$

$$K_{Ab_OBE} := \left(\frac{1 - \tan(\phi)\cdot\cot(\alpha_{A_OBE})}{1 + \tan(\phi)\cdot\tan(\alpha_{A_OBE})}\right)\cdot\left[1 + \left(\frac{\tan(\alpha_{A_OBE})}{\tan(\alpha_{A_OBE}) - \tan(\beta)} - 1\right)\cdot\frac{\gamma_m}{\gamma_b}\right] \qquad K_{Ab_OBE} = 0.304$$

$$K_{Ab_MDE} := \left(\frac{1 - \tan(\phi)\cdot\cot(\alpha_{A_MDE})}{1 + \tan(\phi)\cdot\tan(\alpha_{A_MDE})}\right)\cdot\left[1 + \left(\frac{\tan(\alpha_{A_MDE})}{\tan(\alpha_{A_MDE}) - \tan(\beta)} - 1\right)\cdot\frac{\gamma_m}{\gamma_b}\right] \qquad K_{Ab_MDE} = 0.244$$

$$P_A = P_{A1} + P_{A2} = \frac{1}{2}\cdot K_A\cdot\gamma\cdot(h - h_s)^2 + \frac{1}{2}\cdot h_s\cdot\left[2\cdot K_A\cdot\gamma\cdot(h - h_s) + K_b\cdot\gamma_b\cdot h_s\right] \qquad \text{(Equ. G-26)}$$

$$P_{A_OBE} := \frac{1}{2}\cdot b\cdot K_{A_OBE}\cdot\gamma_m\cdot(h - h_w)^2 + \frac{1}{2}\cdot b\cdot h_w\cdot\left[2\cdot K_{A_OBE}\cdot\gamma_m\cdot(h - h_w) + K_{Ab_OBE}\cdot\gamma_b\cdot h_w\right] \qquad P_{A_OBE} = 0.656\cdot kip$$

$$P_{A_MDE} := \frac{1}{2}\cdot b\cdot K_{A_MDE}\cdot\gamma_m\cdot(h - h_w)^2 + \frac{1}{2}\cdot b\cdot h_w\cdot\left[2\cdot K_{A_MDE}\cdot\gamma_m\cdot(h - h_w) + K_{Ab_OBE}\cdot\gamma_b\cdot h_w\right] \qquad P_{A_MDE} = 0.656\cdot kip$$

$$P_{ws} = \frac{1}{2}\cdot\gamma_w\cdot h_s^2 \qquad \text{(Equ. G-27)}$$

$$P_{ws} := \frac{1}{2}\cdot b\cdot\gamma_w\cdot h_w^2 \qquad P_{ws} = 2.000\cdot kip$$

Stability

- dynamic

$$\Delta P_{AE} = \Delta P_{AE1} + \Delta P_{AE2} = k_h \cdot \left[\frac{\gamma \cdot h^2}{2 \cdot (\tan(\alpha) - \tan(\beta))}\right] + k_h \cdot \left[\frac{(\gamma_s - \gamma) \cdot h_s^2}{2 \cdot \tan(\alpha)}\right] \quad \text{(Equ. G-28)}$$

$$\Delta P_{AE_OBE} := b \cdot k_{h_OBE} \cdot \left[\frac{\gamma_m \cdot h^2}{2 \cdot (\tan(\alpha_{A_OBE}) - \tan(\beta))}\right] + b \cdot k_{h_OBE} \cdot \left[\frac{(\gamma_s - \gamma_m) \cdot h_w^2}{2 \cdot \tan(\alpha_{A_OBE})}\right} \quad \Delta P_{AE_OBE} = 0.212 \cdot kip$$

$$\Delta P_{AE_MDE} := b \cdot k_{h_MDE} \cdot \left[\frac{\gamma_m \cdot h^2}{2 \cdot (\tan(\alpha_{A_MDE}) - \tan(\beta))}\right] + b \cdot k_{h_MDE} \cdot \left[\frac{(\gamma_s - \gamma_m) \cdot h_w^2}{2 \cdot \tan(\alpha_{A_MDE})}\right] \quad \Delta P_{AE_MDE} = 1.101 \cdot kip$$

- total

$$P_{AE} = P_A + P_{ws} + \Delta P_{AE} \quad \text{(Equ. G-29)}$$

$$P_{AE_OBE} := P_{A_OBE} + P_{ws} + \Delta P_{AE_OBE} \quad P_{AE_OBE} = 2.869 \cdot kip$$

$$P_{AE_MDE} := P_{A_MDE} + P_{ws} + \Delta P_{AE_MDE} \quad P_{AE_MDE} = 3.757 \cdot kip$$

Line of Action for P_{AE}

$$Y_{AE} = \frac{P_A \cdot \left(\frac{h}{3}\right) + P_{ws} \cdot \left(\frac{h}{3}\right) + \Delta P_{AE} \cdot \left(\frac{2 \cdot h}{3}\right)}{P_{AE}} \quad \text{(Equ. G-22 w/mod)}$$

$$Y_{AE_OBE} := \frac{P_{A_OBE} \cdot \left(\frac{h}{3}\right) + P_{ws} \cdot \left(\frac{h_w}{3}\right) + \Delta P_{AE_OBE} \cdot \left(\frac{2 \cdot h}{3}\right)}{P_{AE_OBE}} \quad Y_{AE_OBE} = 2.86 \text{ ft}$$

$$Y_{AE_MDE} := \frac{P_{A_MDE} \cdot \left(\frac{h}{3}\right) + P_{ws} \cdot \left(\frac{h_w}{3}\right) + \Delta P_{AE_MDE} \cdot \left(\frac{2 \cdot h}{3}\right)}{P_{AE_MDE}} \quad Y_{AE_MDE} = 3.45 \text{ ft}$$

Stability

(2) Resisting Force

- static

$$K_{P_OBE} := \left(\frac{1 + \tan(\phi)\cdot\cot(\alpha_{P_OBE})}{1 - \tan(\phi)\cdot\tan(\alpha_{P_OBE})}\right)\cdot\left(\frac{\tan(\alpha_{P_OBE})}{\tan(\alpha_{P_OBE}) - \tan(\beta)}\right) \qquad K_{P_OBE} = 3.258 \qquad \text{(Equ. G-12)}$$

$$K_{P_MDE} := \left(\frac{1 + \tan(\phi)\cdot\cot(\alpha_{P_MDE})}{1 - \tan(\phi)\cdot\tan(\alpha_{P_MDE})}\right)\cdot\left(\frac{\tan(\alpha_{P_MDE})}{\tan(\alpha_{P_MDE}) - \tan(\beta)}\right) \qquad K_{P_MDE} = 3.318 \qquad \text{(Equ. G-12)}$$

$$P_P = \frac{1}{2}\cdot K_P\cdot\gamma\cdot h^2 \qquad \text{(Equ. G-10)}$$

$$P_{P_OBE} := \frac{1}{2}\cdot b\cdot K_{P_OBE}\cdot\gamma_m\cdot(t_{ftg} + h_{s2})^2 \qquad P_{P_OBE} = 5.091\cdot kip$$

$$P_{P_MDE} := \frac{1}{2}\cdot b\cdot K_{P_MDE}\cdot\gamma_m\cdot(t_{ftg} + h_{s2})^2 \qquad P_{P_MDE} = 5.185\cdot kip$$

Pressures:

$$p_{OBE} := K_{P_OBE}\cdot\gamma_m\cdot(t_{ftg} + h_{s2}) \qquad p_{OBE} = 2.0363\cdot ksf$$

$$p_{MDE} := K_{P_MDE}\cdot\gamma_m\cdot(t_{ftg} + h_{s2}) \qquad p_{MDE} = 2.0739\cdot ksf$$

- dynamic

$$\Delta P_{PE} = k_h\cdot\left[\frac{\gamma\cdot h^2}{2\cdot(\tan(\alpha) - \tan(\beta))}\right] \qquad \text{(Equ. G-19)}$$

$$\Delta P_{PE_OBE} := b\cdot k_{h_OBE}\cdot\left[\frac{\gamma_m\cdot(t_{ftg} + h_{s2})^2}{2\cdot(\tan(\alpha_{P_OBE}) - \tan(\beta))}\right] \qquad \Delta P_{PE_OBE} = 0.236\cdot kip$$

$$\Delta P_{PE_MDE} := b\cdot k_{h_MDE}\cdot\left[\frac{\gamma_m\cdot(t_{ftg} + h_{s2})^2}{2\cdot(\tan(\alpha_{P_MDE}) - \tan(\beta))}\right] \qquad \Delta P_{PE_MDE} = 0.954\cdot kip$$

Pressures:

$$\Delta p_{OBE} := \frac{\Delta P_{PE_OBE}}{\frac{1}{2}\cdot b\cdot(t_{ftg} + h_{s2})} \qquad \Delta p_{MDE} := \frac{\Delta P_{PE_MDE}}{\frac{1}{2}\cdot b\cdot(t_{ftg} + h_{s2})}$$

$$\Delta p_{OBE} = 0.0944\cdot ksf \qquad \Delta p_{MDE} = 0.3818\cdot ksf$$

Check above pressures to see if sum causes earth pressures to pull on the structure some distance below the top of the ground line. Since earth pressure cannot pull on the structure, set these pressures to zero.

Stability

- total

$P_{PE} = P_P - \Delta P_{PE}$ (Equ. G-20)

$P_{PE_OBE} := P_{P_OBE} - \Delta P_{PE_OBE}$ $P_{PE_OBE} = 4.855 \cdot kip$

$P_{PE_MDE} := P_{P_MDE} - \Delta P_{PE_MDE}$ $P_{PE_MDE} = 4.230 \cdot kip$

Line of Action for P_{PE} $Y_{PE} = \dfrac{P_P \cdot \left(\frac{h}{3}\right) - \Delta P_{PE} \cdot \left(\frac{2 \cdot h}{3}\right)}{P_{PE}}$ (Equ. G-25)

$Y_{PE_OBE} := \dfrac{P_{P_OBE} \cdot \left(\frac{t_{ftg} + h_{s2}}{3}\right) - \Delta P_{PE_OBE} \cdot \left[\frac{2 \cdot (t_{ftg} + h_{s2})}{3}\right]}{P_{PE_OBE}}$ $Y_{PE_OBE} = 1.59\ ft$

$Y_{PE_MDE} := \dfrac{P_{P_MDE} \cdot \left(\frac{t_{ftg} + h_{s2}}{3}\right) - \Delta P_{PE_MDE} \cdot \left[\frac{2 \cdot (t_{ftg} + h_{s2})}{3}\right]}{P_{PE_MDE}}$ $Y_{PE_MDE} = 1.29\ ft$

Remove 'pulling' pressure from above computations. See figure on next page.
From other computations, OBE and MDE pressure height reduces to h_{rev}.

$h_{rev_OBE} := 4.75 \cdot ft$ $h_{rev_MDE} := 4.2 \cdot ft$

$P_{PE_OBE_rev} := \frac{1}{2} \cdot b \cdot h_{rev_OBE} \cdot p_{OBE}$ $P_{PE_OBE_rev} = 4.836 \cdot kip$

$MP_{PE_OBE_rev} := P_{PE_OBE_rev} \cdot \left(\frac{h_{rev_OBE}}{3}\right)$ $MP_{PE_OBE_rev} = 7.657 \cdot kip \cdot ft$

$P_{PE_MDE_rev} := \frac{1}{2} \cdot b \cdot h_{rev_MDE} \cdot p_{MDE}$ $P_{PE_MDE_rev} = 4.355 \cdot kip$

$MP_{PE_MDE_rev} := P_{PE_MDE_rev} \cdot \left(\frac{h_{rev_MDE}}{3}\right)$ $MP_{PE_MDE_rev} = 6.097 \cdot kip \cdot ft$

Stability

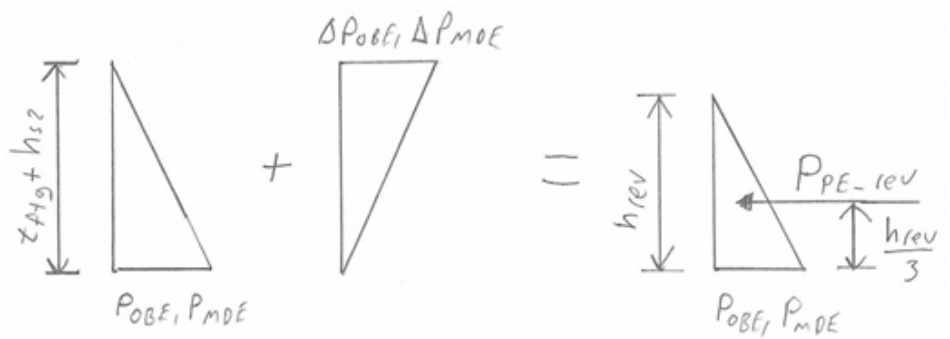


Figure: Reduced Downstream

Horizontal Inertia

From vertical weight.

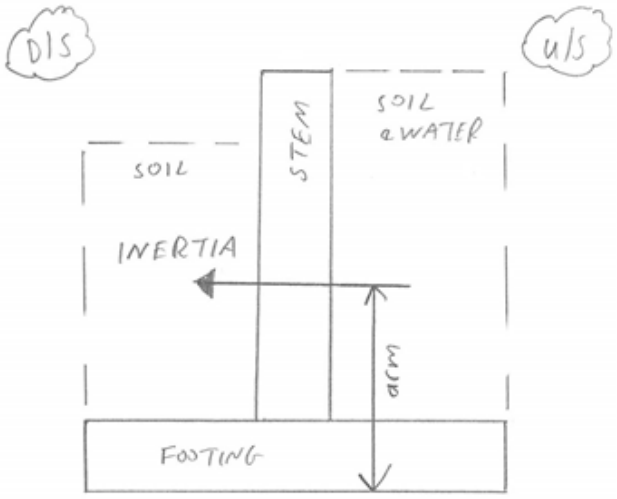
$armW_{c1H} := t_{ftg} + \frac{h - t_{ftg}}{2}$ $armW_{c1H} = 4.875\ ft$

$armW_{c2H} := \frac{t_{ftg}}{2}$ $armW_{c2H} = 0.875\ ft$

$armW_{s1H} := t_{ftg} + \frac{h_{s1}}{2}$ $armW_{s1H} = 4.875\ ft$

$armW_{s2H} := t_{ftg} + \frac{h_{s2}}{2}$ $armW_{s2H} = 3.375\ ft$

$armWw_{h1H} := t_{ftg} + \frac{h_{s1}}{2}$ $armWw_{h1H} = 4.875\ ft$



$$armW_H := \frac{W_{c1}\cdot armW_{c1H} + W_{c2}\cdot armW_{c2H} + W_{s1}\cdot armW_{s1H} + W_{s2}\cdot armW_{s2H} + Ww_{h1}\cdot armWw_{h1H}}{W_{c1} + W_{c2} + W_{s1} + W_{s2} + Ww_{h1}} \qquad armW_H = 3.460\ ft$$

$Inertia_{str_OBE} := k_{h_OBE}\cdot\left[\left(W_{c1} + W_{c2}\right) + \left(W_{s1} + W_{s2}\right) + \left(Ww_{h1}\right)\right]$ $Inertia_{str_OBE} = 0.702\cdot kip$

$Inertia_{str_MDE} := k_{h_MDE}\cdot\left[\left(W_{c1} + W_{c2}\right) + \left(W_{s1} + W_{s2}\right) + \left(Ww_{h1}\right)\right]$ $Inertia_{str_MDE} = 2.431\cdot kip$

Stability

SUM OF VERTICAL FORCES

LC5 & LC6 $sumV := (W_{c1} + W_{c2}) + (W_{s1} + W_{s2}) + (Ww_{h1}) - (UP)$ $sumV = 6.400 \cdot kip$

SUM OF HORIZONTAL FORCES

If sumH < 0, then adjust force in F_{s2} as a triangle shape so sumH = 0.

LC5 $P_{PE_OBE_rev_new} := if[P_{AE_OBE} - P_{PE_OBE_rev} + Inertia_{str_OBE} < 0 \cdot kip, P_{AE_OBE} + Inertia_{str_OBE}, (P_{PE_OBE_rev})]$

$P_{PE_OBE_rev_new} = 3.571 \cdot kip$

$sumH_{OBE} := P_{AE_OBE} - P_{PE_OBE_rev_new} + Inertia_{str_OBE}$ $sumH_{OBE} = 0.000 \cdot kip$

LC6 $P_{PE_MDE_rev_new} := if[P_{AE_MDE} - P_{PE_MDE_rev} + Inertia_{str_MDE} < 0 \cdot kip, P_{AE_MDE} + Inertia_{str_MDE}, (P_{PE_MDE_rev})]$

$P_{PE_MDE_rev_new} = 4.355 \cdot kip$

$sumH_{MDE} := P_{AE_MDE} - P_{PE_MDE_rev_new} + Inertia_{str_MDE}$ $sumH_{MDE} = 1.833 \cdot kip$

RESISTING MOMENT (moments about point A)

$M_{R_conc} := W_{c1} \cdot (armW_{c1}) + W_{c2} \cdot (armW_{c2})$ $M_{R_conc} = 18.525 \cdot kip \cdot ft$

$M_{R_soil} := W_{s1} \cdot (armW_{s1}) + W_{s2} \cdot (armW_{s2})$ $M_{R_soil} = 14.641 \cdot kip \cdot ft$

$M_{R_water} := Ww_{h1} \cdot (armWw_{h1})$ $M_{R_water} = 11.719 \cdot kip \cdot ft$

$M_R := M_{R_conc} + M_{R_soil} + M_{R_water}$ $M_R = 44.884 \cdot kip \cdot ft$

OVERTURNING MOMENT (moments about point A) w/o Uplift

LC5 $M_{O_OBE} := P_{AE_OBE} \cdot Y_{AE_OBE} - P_{PE_OBE_rev_new} \cdot \left(\frac{h_{rev_OBE}}{3}\right) + Inertia_{str_OBE} \cdot armW_H$ $M_{O_OBE} = 4.991 \cdot kip \cdot ft$

LC6 $M_{O_MDE} := P_{AE_MDE} \cdot Y_{AE_MDE} - P_{PE_MDE_rev_new} \cdot \left(\frac{h_{rev_MDE}}{3}\right) + Inertia_{str_MDE} \cdot armW_H$ $M_{O_MDE} = 15.270 \cdot kip \cdot ft$

Stability

SLIDING SAFETY FACTORS

For Load Cases 5 and 6

$FS5_{sliding} := if\left[sumH_{OBE} = 0 \cdot kip, "Stable", \frac{(sumV) \cdot \tan(\phi_{foundation})}{sumH_{OBE}}\right]$

$FS5_{sliding}$ = Stable

$FS5_{sliding} = 2.40 \times 10^{16}$

$FS5_{SL} := if(FS5_{sliding} = "Stable", 100, FS5_{sliding})$

$Check(FS5_{SL}, 1.3) = "OK"$

$FS6_{sliding} := if\left[sumH_{MDE} = 0 \cdot kip, "Stable", \frac{(sumV) \cdot \tan(\phi_{foundation})}{sumH_{MDE}}\right]$

$FS6_{sliding} = 1.34$

$FS6_{SL} := if(FS6_{sliding} = "Stable", 100, FS6_{sliding})$

$Check(FS6_{SL}, 1.1) = "OK"$

Stability

BEARING CAPACITY

Ref. 6

R = Resultant from toe & e = eccentricity from base centroid

$R5 := \frac{M_R - (M_{O_OBE} + M_{UP})}{sumV}$ $R5 = 3.883\ ft$

$R6 := \frac{M_R - (M_{O_MDE} + M_{UP})}{sumV}$ $R6 = 2.277\ ft$

$e5 := \frac{base}{2} - R5$ $e5 = 0.867\ ft$

$e6 := \frac{base}{2} - R6$ $e6 = 2.473\ ft$

where $\frac{base}{3} = 3.167\ ft$ and $\frac{2 \cdot base}{3} = 6.333\ ft$

$For5 := if\left[R5 > \left(\frac{1}{3} \cdot base\right), if\left[R5 > \left(\frac{2}{3} \cdot base\right), "Not\ In\ Kern", "In\ Kern"\right], "Not\ In\ Kern"\right]$ $For5 = "In\ Kern"$

$For6 := if\left[R6 > \left(\frac{1}{3} \cdot base\right), if\left[R6 > \left(\frac{2}{3} \cdot base\right), "Not\ In\ Kern", "In\ Kern"\right], "Not\ In\ Kern"\right]$ $For6 = "Not\ In\ Kern"$

Percent Base In Compression

Table 4-1, Page 4-5.

$\%Base_Comp5 := if\left[For5 = "In\ Kern", 1, \left(\frac{3 \cdot R5}{base}\right)\right]$ $\%Base_Comp5 = 100.0 \cdot \%$ $Check(\%Base_Comp5, 0.75) = "OK"$

$\%Base_Comp6 := if\left[For6 = "In\ Kern", 1, \left(\frac{3 \cdot R6}{base}\right)\right]$ $\%Base_Comp6 = 71.9 \cdot \%$ $Check(\%Base_Comp6, 0.01) = "OK"$

Stability

Heel Pressure (Equ. 3-1)

$$Press5_{Heel} := if\left[For5 = "In\ Kern",\left(\frac{sumV}{base \cdot b}\right)\cdot\left(1-\frac{6\cdot e5}{base}\right),0\cdot psf\right] \qquad Press5_{Heel} = 305\cdot psf$$

$$Press6_{Heel} := if\left[For6 = "In\ Kern",\left(\frac{sumV}{base \cdot b}\right)\cdot\left(1-\frac{6\cdot e6}{base}\right),0\cdot psf\right] \qquad Press6_{Heel} = 0\cdot psf$$

Toe Pressure (Equ. 3-1 and 3-2)

$$Press5_{Toe} := if\left[For5 = "In\ Kern",\left(\frac{sumV}{base \cdot b}\right)\cdot\left(1+\frac{6\cdot e5}{base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV}{(base-2\cdot e5)\cdot b}\right]\right] \qquad Press5_{Toe} = 1043\cdot psf$$

$$Press6_{Toe} := if\left[For6 = "In\ Kern",\left(\frac{sumV}{base \cdot b}\right)\cdot\left(1+\frac{6\cdot e6}{base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV}{(base-2\cdot e6)\cdot b}\right]\right] \qquad Press6_{Toe} = 1874\cdot psf$$

Base Pressure Length (Equ. 3-3)

$$Press5_{Length} := if\left[For5 = "In\ Kern",base,\left(\frac{3}{2}\right)\cdot(base-2\cdot e5)\right] \qquad Press5_{Length} = 9.500\ ft$$

$$Press6_{Length} := if\left[For6 = "In\ Kern",base,\left(\frac{3}{2}\right)\cdot(base-2\cdot e6)\right] \qquad Press6_{Length} = 6.831\ ft$$

Summary

$$BasePressure5 := \max\left(Press5_{Heel},Press5_{Toe}\right) \qquad BasePressure5 = 1043\cdot psf$$

$$Check\left(BearingCap_{allow_eq},BasePressure5\right) = "OK"$$

$$BasePressure6 := \max\left(Press6_{Heel},Press6_{Toe}\right) \qquad BasePressure6 = 1874\cdot psf$$

$$Check\left(BearingCap_{allow_eq},BasePressure6\right) = "OK"$$

Concrete

CONCRETE STRUCTURE

Load Cases per Ref. 3, Section 3.3

Lp = permanent loads
Lt = temporary (intermittent static) loads -> i.e. floods
Ld = dynamic (impulse) loads -> i.e. earthquake

Lp = 1] concrete (D), soil (EV & EH)
2] vertical and horizontal normal water (Hs)
3] normal uplift (Hs)

Lt = 1] vertical and horizontal flood (Hs)
2] flood uplift (Hs)

Ld = horizontal earthquake loads (EQ)

Load Cases (see previous) and Load Factors

LC 2 - Usual: Normal Pool
Equ. 3-2: U = 2.2 [D + EV + EH + Hs]

LC 3 - Unusual: Infrequent Flood = Q100
Equ. 3-3: U = 1.6 [D + EV + EH + Hs]

LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
Equ. 3-7: U = 1.5 [D + EV + EH + Hs + EQ]

LC 6 - Extreme: Coincident Pool = Normal Pool with MDE
Equ. 3-8 and Table 3-1:
U = (1.2 add to effect or 0.9 subtract) [D] + (1.5 active or 0.5 passive) [EH] + (1.35 add to effect or 1.0 subtract) [EV]
+ 1.0 [Hs] + 1.25 EQ

LC 7 - Extreme: Maximum Design Flood = Q10000
Equ. 3-6 and Table 3-1:
U = (1.2 add to effect or 0.9 subtract) [D] + (1.5 active or 0.5 passive) [EH] + (1.35 add to effect or 1.0 subtract) [EV]
+ 1.3 [Hs]

where:

D = Dead Load
EH = Moist or Effective Lateral Earth Pressures
EV = Vertical Earth
EQ = Earthquake
Hs = Hydrostatic (Lateral and Vertical)

Concrete - Stem

HORIZONTAL FORCES @ STEM BASE

Soil:

- Ignore resisting

$F_{s1b} := \frac{1}{2}\cdot b\cdot K_a\cdot(\gamma_s - \gamma_w)\cdot(h_{s1})^2$ $F_{s1b} = 0.405\cdot kip$ $armF_{s1b} := \frac{h_{s1}}{3}$ $armF_{s1b} = 2.083\ ft$

Water:

$Fw_{h1b} := \frac{1}{2}\cdot b\cdot\gamma_w\cdot(h_w - t_{ftg})^2$ $Fw_{h1b} = 1.221\cdot kip$ $armFw_{h1b} := \frac{h_w - t_{ftg}}{3}$ $armFw_{h1b} = 2.083\ ft$

$Fw_{h1_f_IF_Ab} := \frac{1}{2}\cdot b\cdot\gamma_w\cdot(h_{wf_IF} - t_{ftg})^2$ $armFw_{h1_f_IF_Ab} := \frac{h_{wf_IF} - t_{ftg}}{3}$

$Fw_{h1_f_IF_Ab} = 2.210\cdot kip$ $armFw_{h1_f_IF_Ab} = 2.803\ ft$

$Fw_{h1_f_IF_Bb} := \frac{1}{2}\cdot b\cdot\gamma_w\cdot\left[h_{wf_IF} - (h_{s1} + t_{ftg})\right]^2$ $armFw_{h1_f_IF_Bb} := (h_{s1}) + \frac{h_{wf_IF} - (h_{s1} + t_{ftg})}{3}$

$Fw_{h1_f_IF_Bb} = 0.146\cdot kip$ $armFw_{h1_f_IF_Bb} = 6.970\ ft$

$Fw_{h1_f_IFb} := Fw_{h1_f_IF_Ab} - Fw_{h1_f_IF_Bb}$ $Fw_{h1_f_IFb} = 2.064\cdot kip$

$armFw_{h1_f_IFb} := \frac{Fw_{h1_f_IF_Ab}\cdot armFw_{h1_f_IF_Ab} - Fw_{h1_f_IF_Bb}\cdot armFw_{h1_f_IF_Bb}}{Fw_{h1_f_IFb}}$ $armFw_{h1_f_IFb} = 2.509\ ft$

$Fw_{h1_f_MDF_Ab} := \frac{1}{2}\cdot b\cdot\gamma_w\cdot(h_{wf_MDF} - t_{ftg})^2$ $armFw_{h1_f_MDF_Ab} := \frac{h_{wf_MDF} - t_{ftg}}{3}$

$Fw_{h1_f_MDF_Ab} = 4.241\cdot kip$ $armFw_{h1_f_MDF_Ab} = 3.883\ ft$

$Fw_{h1_f_MDF_Bb} := \frac{1}{2}\cdot b\cdot\gamma_w\cdot\left[h_{wf_MDF} - (h_{s1} + t_{ftg})\right]^2$ $armFw_{h1_f_MDF_Bb} := (h_{s1}) + \frac{h_{wf_MDF} - (h_{s1} + t_{ftg})}{3}$

$Fw_{h1_f_MDF_Bb} = 0.911\cdot kip$ $armFw_{h1_f_MDF_Bb} = 8.050\ ft$

$Fw_{h1_f_MDFb} := Fw_{h1_f_MDF_Ab} - Fw_{h1_f_MDF_Bb}$ $Fw_{h1_f_MDFb} = 3.330\cdot kip$

$armFw_{h1_f_MDFb} := \frac{Fw_{h1_f_MDF_Ab}\cdot armFw_{h1_f_MDF_Ab} - Fw_{h1_f_MDF_Bb}\cdot armFw_{h1_f_MDF_Bb}}{Fw_{h1_f_MDFb}}$ $armFw_{h1_f_MDFb} = 2.743\ ft$

Concrete - Stem

Earthquake --> (1) Driving Force

- static

$$P_{A_OBEb} := \frac{1}{2}\cdot b\cdot K_{A_OBE}\cdot\gamma_m\cdot\left[\left(h - t_{ftg}\right) - \left(h_w - t_{ftg}\right)\right]^2 \ldots + \frac{1}{2}\cdot b\cdot\left(h_w - t_{ftg}\right)\cdot\left[2\cdot K_{A_OBE}\cdot\gamma_m\cdot\left[\left(h - t_{ftg}\right) - \left(h_w - t_{ftg}\right)\right] + K_{Ab_OBE}\cdot\gamma_b\cdot\left(h_w - t_{ftg}\right)\right] \qquad P_{A_OBEb} = 0.401\cdot kip$$

$$P_{A_MDEb} := \frac{1}{2}\cdot b\cdot K_{A_MDE}\cdot\gamma_m\cdot\left[\left(h - t_{ftg}\right) - \left(h_w - t_{ftg}\right)\right]^2 \ldots + \frac{1}{2}\cdot b\cdot\left(h_w - t_{ftg}\right)\cdot\left[2\cdot K_{A_MDE}\cdot\gamma_m\cdot\left[\left(h - t_{ftg}\right) - \left(h_w - t_{ftg}\right)\right] + K_{Ab_OBE}\cdot\gamma_b\cdot\left(h_w - t_{ftg}\right)\right] \qquad P_{A_MDEb} = 0.401\cdot kip$$

$$P_{wsb} := \frac{1}{2}\cdot b\cdot\gamma_w\cdot\left(h_w - t_{ftg}\right)^2 \qquad P_{wsb} = 1.221\cdot kip$$

- dynamic

$$\Delta P_{AE_OBEb} := b\cdot k_{h_OBE}\cdot\left[\frac{\gamma_m\cdot\left(h - t_{ftg}\right)^2}{2\cdot\left(\tan\left(\alpha_{A_OBE}\right) - \tan(\beta)\right)}\right] + b\cdot k_{h_OBE}\cdot\left[\frac{\left(\gamma_s - \gamma_m\right)\cdot\left(h_w - t_{ftg}\right)^2}{2\cdot\tan\left(\alpha_{A_OBE}\right)}\right] \qquad \Delta P_{AE_OBEb} = 0.130\cdot kip$$

$$\Delta P_{AE_MDEb} := b\cdot k_{h_MDE}\cdot\left[\frac{\gamma_m\cdot\left(h - t_{ftg}\right)^2}{2\cdot\left(\tan\left(\alpha_{A_MDE}\right) - \tan(\beta)\right)}\right] + b\cdot k_{h_MDE}\cdot\left[\frac{\left(\gamma_s - \gamma_m\right)\cdot\left(h_w - t_{ftg}\right)^2}{2\cdot\tan\left(\alpha_{A_MDE}\right)}\right] \qquad \Delta P_{AE_MDEb} = 0.672\cdot kip$$

- total

$$P_{AE_OBEb} := P_{A_OBEb} + P_{wsb} + \Delta P_{AE_OBEb} \qquad P_{AE_OBEb} = 1.751\cdot kip$$

$$P_{AE_MDEb} := P_{A_MDEb} + P_{wsb} + \Delta P_{AE_MDEb} \qquad P_{AE_MDEb} = 2.293\cdot kip$$

Line of Action for P_{AE}

$$Y_{AE_OBEb} := \frac{P_{A_OBEb}\cdot\left(\frac{h - t_{ftg}}{3}\right) + P_{wsb}\cdot\left(\frac{h_w - t_{ftg}}{3}\right) + \Delta P_{AE_OBEb}\cdot\left[\frac{2\cdot\left(h - t_{ftg}\right)}{3}\right]}{P_{AE_OBEb}} \qquad Y_{AE_OBEb} = 2.24\ ft$$

$$Y_{AE_MDEb} := \frac{P_{A_MDEb}\cdot\left(\frac{h - t_{ftg}}{3}\right) + P_{wsb}\cdot\left(\frac{h_w - t_{ftg}}{3}\right) + \Delta P_{AE_MDEb}\cdot\left[\frac{2\cdot\left(h - t_{ftg}\right)}{3}\right]}{P_{AE_MDEb}} \qquad Y_{AE_MDEb} = 2.69\ ft$$

Concrete - Stem

Horizontal Inertia

From vertical weight.

$armW_{c1Hb} := \frac{h - t_{ftg}}{2}$ $armW_{c1Hb} = 3.125\ ft$ $armW_{s1Hb} := \frac{h_{s1}}{2}$ $armW_{s1Hb} = 3.125\ ft$

$armWw_{h1Hb} := \frac{h_{s1}}{2}$ $armWw_{h1Hb} = 3.125\ ft$

$armW_{Hb} := \frac{W_{c1} \cdot armW_{c1Hb} + W_{s1} \cdot armW_{s1Hb} + Ww_{h1} \cdot armWw_{h1Hb}}{W_{c1} + W_{s1} + Ww_{h1}}$ $armW_{Hb} = 3.125\ ft$

$Inertia_{str_OBEb} := k_{h_OBE} \cdot \left[\left(W_{c1} \right) + \left(W_{s1} \right) + \left(Ww_{h1} \right) \right]$ $Inertia_{str_OBEb} = 0.373 \cdot kip$

$Inertia_{str_MDEb} := k_{h_MDE} \cdot \left[\left(W_{c1} \right) + \left(W_{s1} \right) + \left(Ww_{h1} \right) \right]$ $Inertia_{str_MDEb} = 1.290 \cdot kip$

Concrete - Stem

FACTORED STEM HORIZONTAL FORCES @ STEM BASE w/o Resisting

$LC2_sumH_b := 2.2\cdot(F_{s1b} + Fw_{h1b})$ $\quad LC2_sumH_b = 3.577\cdot kip$

$LC3_sumH_{f_IFb} := 1.6\cdot(F_{s1b} + Fw_{h1_f_IFb})$ $\quad LC3_sumH_{f_IFb} = 3.951\cdot kip$

$LC5_sumH_{OBEb} := 1.5\cdot(P_{AE_OBEb} + Inertia_{str_OBEb})$ $\quad LC5_sumH_{OBEb} = 3.185\cdot kip$

$LC6_sumH_{MDEb} := 1.5\cdot(P_{A_MDEb}) + 1.0\cdot(P_{wsb}) + 1.25\cdot(\Delta P_{AE_MDEb} + Inertia_{str_MDEb})$ $\quad LC6_sumH_{MDEb} = 4.274\cdot kip$

$LC7_sumH_{f_MDFb} := 1.5\cdot(F_{s1b}) + 1.3\cdot(Fw_{h1_f_MDFb})$ $\quad LC7_sumH_{f_MDFb} = 4.937\cdot kip$

FACTORED STEM MOMENTS (from Horizontal Forces) @ STEM BASE w/o Resisting

$LC2_sumM_b := 2.2\cdot(F_{s1b}\cdot armF_{s1b} + Fw_{h1b}\cdot armFw_{h1b})$ $\quad LC2_sumM_b = 7.451\cdot kip\cdot ft$

$LC3_sumM_{f_IFb} := 1.6\cdot(F_{s1b}\cdot armF_{s1b} + Fw_{h1_f_IFb}\cdot armFw_{h1_f_IFb})$ $\quad LC3_sumM_{f_IFb} = 9.638\cdot kip\cdot ft$

$LC5_sumM_{OBEb} := 1.5\cdot(P_{AE_OBEb}\cdot Y_{AE_OBEb} + Inertia_{str_OBEb}\cdot armW_{Hb})$ $\quad LC5_sumM_{OBEb} = 7.622\cdot kip\cdot ft$

$$LC6_sumM_{MDEb} := 1.5\cdot\left[P_{A_MDEb}\cdot\left(\frac{h - t_{ftg}}{3}\right)\right] + 1.0\cdot\left[P_{wsb}\cdot\left(\frac{h_w - t_{ftg}}{3}\right)\right] \ldots + 1.25\cdot\left[\Delta P_{AE_MDEb}\cdot\left[\frac{2\cdot(h - t_{ftg})}{3}\right] + Inertia_{str_MDEb}\cdot(armW_{Hb})\right]$$

$LC6_sumM_{MDEb} = 12.333\cdot kip\cdot ft$

$LC7_sumM_{f_MDFb} := 1.5\cdot(F_{s1b}\cdot armF_{s1b}) + 1.3\cdot(Fw_{h1_f_MDFb}\cdot armFw_{h1_f_MDFb})$ $\quad LC7_sumM_{f_MDFb} = 13.141\cdot kip\cdot ft$

Controlling Forces

Factored Base Shear: $V_u := \max(LC2_sumH_b, LC3_sumH_{f_IFb}, LC5_sumH_{OBEb}, LC6_sumH_{MDEb}, LC7_sumH_{f_MDFb})$

$V_u = 4.937\cdot kip$

Factored Base Moment: $M_u := \max(LC2_sumM_b, LC3_sumM_{f_IFb}, LC5_sumM_{OBEb}, LC6_sumM_{MDEb}, LC7_sumM_{f_MDFb})$

$M_u = 13.141\cdot kip\cdot ft$

Concrete - Stem

STEM REINFORCEMENT DESIGN

- Two Mats in Both Directions

Check Temperature Steel Requirement

COE (Ref. 3, Section 2.6): 12" center-to-center spacing of both primary and secondary bars.
COE (Ref. 3, Sections 2.9.1 and 2.9.2): 0.003 for Grade 60, with maximum area equivalent to #9 @ 12" in each face

$A_{temp_COE} := \frac{0.003 \cdot b \cdot t_{stem}}{2}$ $A_{temp_COE} = 0.324 \cdot in^2$

#4 @ 7" OC => A = 0.337 in²
#5 @ 11" OC => A = 0.335 in²
#6 @ 12" OC => A = 0.442 in²

Check Moment Steel Requirement

$d := t_{stem} - c - \frac{1}{2} \cdot \left(\frac{6}{8} in\right)$ $d = 14.6250 \cdot in$

$R := \frac{M_u}{\phi M \cdot b \cdot d^2}$ $R = 68.27 \cdot psi$

$\rho := \frac{0.85 \cdot f'_c}{f_y} \cdot \left[1 - \sqrt{\left[1 - \left(\frac{2 \cdot R}{0.85 \cdot f'_c}\right)\right]}\right]$ $\rho = 0.00115$

Ref. 2, 9.6.1.2 $\rho_{min} := if\left[\left(\frac{200 \cdot psi}{f_y}\right) > \left(\frac{3\sqrt{f'_c \cdot psi}}{f_y}\right), \left(\frac{200 \cdot psi}{f_y}\right), \left(\frac{3\sqrt{f'_c \cdot psi}}{f_y}\right)\right]$ $\rho_{min} = 0.00333$

Ref. 2, 9.6.1.3 $\frac{4}{3} \cdot \rho = 0.00153$

$\rho\rho := if\left(\rho_{min} > \rho, if\left(\rho_{min} > \frac{4}{3} \cdot \rho, \frac{4}{3} \cdot \rho, \rho_{min}\right), \rho\right)$ $\rho\rho = 0.00153$

Ref. 3, Appendix C $\rho_{balanced} := \left(\frac{0.85 \cdot f'_c \cdot \beta_1}{f_y}\right) \cdot \left(\frac{87000 \cdot psi}{87000 \cdot psi + f_y}\right)$ $\rho_{balanced} = 0.02851$

$\rho_{max} := 0.25 \cdot \rho_{balanced}$ [Ref. 3, Chapter 3, Section 3.5] $\rho_{max} = 0.00713$

$rein_ratio := if(\rho_{max} > \rho\rho, "OK", "NG")$ $rein_ratio = "OK"$

Concrete - Stem

$A_s := \rho\rho \cdot b \cdot d$ $\quad A_s = 0.269 \cdot in^2$ / foot

$A_{smin} := \rho_{min} \cdot b \cdot d$ $\quad A_{smin} = 0.585 \cdot in^2$ / foot

****

$BarNo := 6$ $\quad B := BarNo$ * see temperature requirements

$Spacing := 12 \cdot in$

****

$$db := if\left(B = 4, \frac{4}{8} \cdot in, if\left(B = 5, \frac{5}{8} \cdot in, if\left(B = 6, \frac{6}{8} \cdot in, if\left(B = 7, \frac{7}{8} \cdot in, if\left(B = 8, \frac{8}{8} \cdot in, if(B = 9, 1.128 \cdot in, 1.270 \cdot in)\right)\right)\right)\right)\right)$$

$$BarArea := \frac{\pi \cdot db^2}{4} \qquad BarArea = 0.442 \cdot in^2$$

$$A_{sprov} := \frac{BarArea \cdot (12 \cdot in)}{Spacing} \qquad A_{sprov} = 0.442 \cdot in^2 \text{ / foot}$$

Check: $A_{sprov} = 0.442 \cdot in^2$ / foot >? $A_s = 0.269 \cdot in^2$ / foot $\quad Check(A_{sprov}, A_s) = "OK"$

$$d_w := t_{stem} - c - \frac{db}{2} \qquad d_w = 14.625 \cdot in$$

$$\phi M_n := \phi M \cdot \left[A_{sprov} \cdot f_y \cdot \left(d_w - \frac{a}{2}\right)\right] \qquad a := \frac{A_{sprov} \cdot f_y}{0.85 \cdot f'_c \cdot b} \qquad a = 0.650 \cdot in$$

$\phi M_n = 28.43 \cdot kip \cdot ft$ >? $M_u = 13.14 \cdot kip \cdot ft$ $\quad Check(\phi M_n, M_u) = "OK"$

SHEAR

Shear Strength of Normal-Weight Concrete (Ref. 2, 22.5.5.1)

$$\phi V_c := \phi V \cdot \left(2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d_w\right)$$

$\phi V_c = 16.65 \cdot kip$

$Check(\phi V_c, V_u) = "OK"$

Maximum Shear

$V_u = 4.94 \cdot kip$

Minimum Shear Reinforcement (Ref. 2, 7.6.3.2)
None required if M_n and V_n can be developed.

Concrete - Footing

FOOTING DESIGN

Vertical Concrete, Soil and Water Weight Forces and Moment Arms (measured from stem face):

Heel:

$W_{ch} := b \cdot \gamma_c \cdot b_{heel} \cdot t_{ftg}$

$W_{ch} = 1.050 \cdot kip$

$W_{s1} = 1.687 \cdot kip$

$Ww_{h1} = 1.563 \cdot kip$

$arm_CSW_h := \frac{b_{heel}}{2}$

$arm_CSW_h = 2.000\ ft$

Toe:

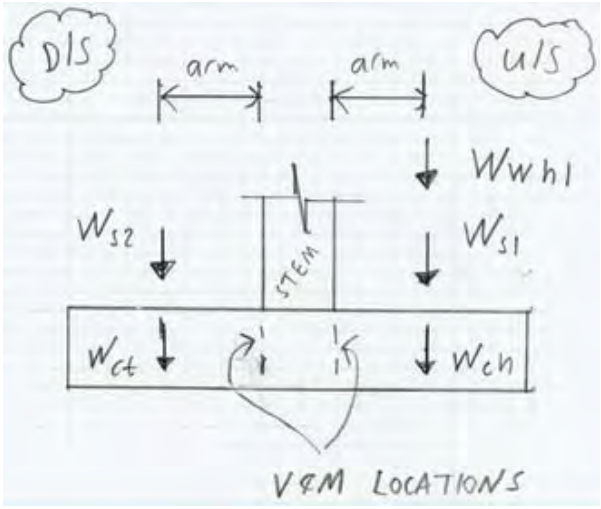
$W_{ct} := b \cdot \gamma_c \cdot b_{toe} \cdot t_{ftg}$

$W_{ct} = 1.050 \cdot kip$

$W_{s2} = 1.625 \cdot kip$

$arm_CSW_t := \frac{b_{toe}}{2}$

$arm_CSW_t = 2.000\ ft$



Concrete - Footing

Base Pressure Upward Forces

-- at face of stem

$Press2_{Heel} = 460 \cdot psf$ $Press3_{Heel} = 174 \cdot psf$ $Press5_{Heel} = 305 \cdot psf$

$Press2_{Toe} = 888 \cdot psf$ $Press3_{Toe} = 1173 \cdot psf$ $Press5_{Toe} = 1043 \cdot psf$

$Press2_{Length} = 9.500\ ft$ $Press3_{Length} = 9.500\ ft$ $Press5_{Length} = 9.500\ ft$

$$pt2 := Press2_{Toe} + \left(\frac{b_{toe}}{Press2_{Length}}\right) \cdot \left(Press2_{Heel} - Press2_{Toe}\right) \qquad pt2 = 0.707 \cdot ksf$$

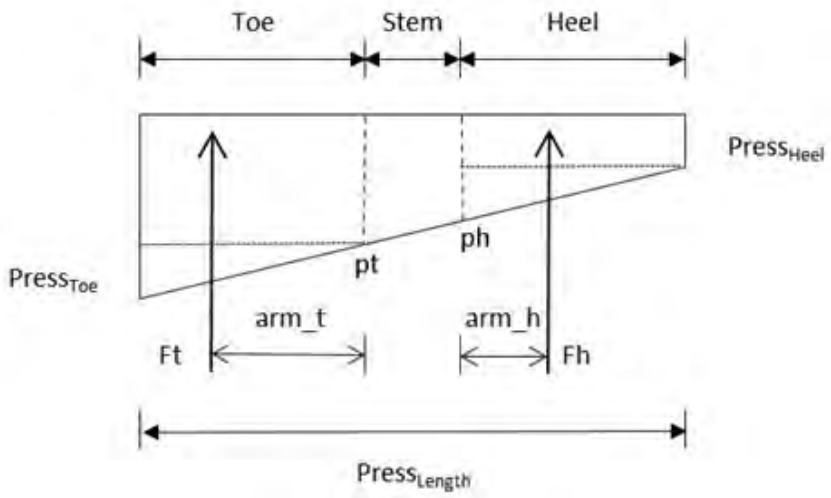
$$ph2 := Press2_{Toe} + \left(\frac{Press2_{Length} - b_{heel}}{Press2_{Length}}\right) \cdot \left(Press2_{Heel} - Press2_{Toe}\right) \qquad ph2 = 0.640 \cdot ksf$$

$$pt3 := Press3_{Toe} + \left(\frac{b_{toe}}{Press3_{Length}}\right) \cdot \left(Press3_{Heel} - Press3_{Toe}\right) \qquad pt3 = 0.753 \cdot ksf$$

$$ph3 := Press3_{Toe} + \left(\frac{Press3_{Length} - b_{heel}}{Press3_{Length}}\right) \cdot \left(Press3_{Heel} - Press3_{Toe}\right) \qquad ph3 = 0.595 \cdot ksf$$

$$pt5 := Press5_{Toe} + \left(\frac{b_{toe}}{Press5_{Length}}\right) \cdot \left(Press5_{Heel} - Press5_{Toe}\right) \qquad pt5 = 0.732 \cdot ksf$$

$$ph5 := Press5_{Toe} + \left(\frac{Press5_{Length} - b_{heel}}{Press5_{Length}}\right) \cdot \left(Press5_{Heel} - Press5_{Toe}\right) \qquad ph5 = 0.615 \cdot ksf$$



Concrete - Footing

Pressure Forces and Moment Arms (measured from stem face):

$Ft_2 := b \cdot b_{toe} \cdot \left[\frac{1}{2} \cdot \left(Press2_{Toe} + pt2\right)\right]$ $Ft_2 = 3.191 \cdot kip$

$Fh_2 := b \cdot b_{heel} \cdot \left[\frac{1}{2} \cdot \left(Press2_{Heel} + ph2\right)\right]$ $Fh_2 = 2.199 \cdot kip$

$arm_t_2 := \frac{b_{toe} \cdot \left(2 \cdot Press2_{Toe} + pt2\right)}{3 \cdot \left(Press2_{Toe} + pt2\right)}$ $arm_t_2 = 2.08\ ft$

$arm_h_2 := \frac{b_{heel} \cdot \left(2 \cdot Press2_{Heel} + ph2\right)}{3 \cdot \left(Press2_{Heel} + ph2\right)}$ $arm_h_2 = 1.89\ ft$

$Ft_3 := b \cdot b_{toe} \cdot \left[\frac{1}{2} \cdot \left(Press3_{Toe} + pt3\right)\right]$ $Ft_3 = 3.851 \cdot kip$

$Fh_3 := b \cdot b_{heel} \cdot \left[\frac{1}{2} \cdot \left(Press3_{Heel} + ph3\right)\right]$ $Fh_3 = 1.539 \cdot kip$

$arm_t_3 := \frac{b_{toe} \cdot \left(2 \cdot Press3_{Toe} + pt3\right)}{3 \cdot \left(Press3_{Toe} + pt3\right)}$ $arm_t_3 = 2.15\ ft$

$arm_h_3 := \frac{b_{heel} \cdot \left(2 \cdot Press3_{Heel} + ph3\right)}{3 \cdot \left(Press3_{Heel} + ph3\right)}$ $arm_h_3 = 1.64\ ft$

$Ft_5 := b \cdot b_{toe} \cdot \left[\frac{1}{2} \cdot \left(Press5_{Toe} + pt5\right)\right]$ $Ft_5 = 3.549 \cdot kip$

$Fh_5 := b \cdot b_{heel} \cdot \left[\frac{1}{2} \cdot \left(Press5_{Heel} + ph5\right)\right]$ $Fh_5 = 1.840 \cdot kip$

$arm_t_5 := \frac{b_{toe} \cdot \left(2 \cdot Press5_{Toe} + pt5\right)}{3 \cdot \left(Press5_{Toe} + pt5\right)}$ $arm_t_5 = 2.12\ ft$

$arm_h_5 := \frac{b_{heel} \cdot \left(2 \cdot Press5_{Heel} + ph5\right)}{3 \cdot \left(Press5_{Heel} + ph5\right)}$ $arm_h_5 = 1.77\ ft$

Concrete - Footing

LOAD CASES 2, 3, and 5: FACTORED FOOTING VERTICAL FORCES @ STEM FACE – for (1) heel and (2) toe

+ is downward for top reinforcement
- is upward for bottom reinforcement

Heel

$LC2_sumV_{heel} := 2.2\cdot\left(W_{ch} + W_{s1} + Ww_{h1} - Fh_2\right)$ $LC2_sumV_{heel} = 4.623\cdot kip$

$LC3_sumV_{heel} := 1.6\cdot\left(W_{ch} + W_{s1} + Ww_{h1} - Fh_3\right)$ $LC3_sumV_{heel} = 4.418\cdot kip$

$LC5_sumV_{heel} := 1.5\cdot\left(W_{ch} + W_{s1} + Ww_{h1} - Fh_5\right)$ $LC5_sumV_{heel} = 3.689\cdot kip$

$LC2_sumM_{heel} := 2.2\cdot\left[\left(W_{ch} + W_{s1} + Ww_{h1}\right)\cdot\left(arm_CSW_h\right) - Fh_2\cdot\left(arm_h_2\right)\right]$ $LC2_sumM_{heel} = 9.774\cdot kip\cdot ft$

$LC3_sumM_{heel} := 1.6\cdot\left[\left(W_{ch} + W_{s1} + Ww_{h1}\right)\cdot\left(arm_CSW_h\right) - Fh_3\cdot\left(arm_h_3\right)\right]$ $LC3_sumM_{heel} = 9.733\cdot kip\cdot ft$

$LC5_sumM_{heel} := 1.5\cdot\left[\left(W_{ch} + W_{s1} + Ww_{h1}\right)\cdot\left(arm_CSW_h\right) - Fh_5\cdot\left(arm_h_5\right)\right]$ $LC5_sumM_{heel} = 8.000\cdot kip\cdot ft$

Toe

$LC2_sumV_{toe} := 2.2\cdot\left(W_{ct} + W_{s2} - Ft_2\right)$ $LC2_sumV_{toe} = -1.134\cdot kip$

$LC3_sumV_{toe} := 1.6\cdot\left(W_{ct} + W_{s2} - Ft_3\right)$ $LC3_sumV_{toe} = -1.881\cdot kip$

$LC5_sumV_{toe} := 1.5\cdot\left(W_{ct} + W_{s2} - Ft_5\right)$ $LC5_sumV_{toe} = -1.311\cdot kip$

$LC2_sumM_{toe} := 2.2\cdot\left[\left(W_{ct} + W_{s2}\right)\cdot\left(arm_CSW_t\right) - Ft_2\cdot\left(arm_t_2\right)\right]$ $LC2_sumM_{toe} = -2.798\cdot kip\cdot ft$

$LC3_sumM_{toe} := 1.6\cdot\left[\left(W_{ct} + W_{s2}\right)\cdot\left(arm_CSW_t\right) - Ft_3\cdot\left(arm_t_3\right)\right]$ $LC3_sumM_{toe} = -4.659\cdot kip\cdot ft$

$LC5_sumM_{toe} := 1.5\cdot\left[\left(W_{ct} + W_{s2}\right)\cdot\left(arm_CSW_t\right) - Ft_5\cdot\left(arm_t_5\right)\right]$ $LC5_sumM_{toe} = -3.243\cdot kip\cdot ft$

Concrete - Footing

Controlling Forces for LC 2, 3, and 5

- maximum from these cases and all faces

Factored Shear:

$$V_{u_ftg_2_3_5} := \max\left(\left|LC2_sumV_{heel}\right|, \left|LC3_sumV_{heel}\right|, \left|LC5_sumV_{heel}\right|, \left|LC2_sumV_{toe}\right|, \left|LC3_sumV_{toe}\right|, \left|LC5_sumV_{toe}\right|\right)$$

$$V_{u_ftg_2_3_5} = 4.623 \cdot kip$$

Factored Moment:

$$M_{u_ftg_2_3_5} := \max\left(\left|LC2_sumM_{heel}\right|, \left|LC3_sumM_{heel}\right|, \left|LC5_sumM_{heel}\right|, \left|LC2_sumM_{toe}\right|, \left|LC3_sumM_{toe}\right|, \left|LC5_sumM_{toe}\right|\right)$$

$$M_{u_ftg_2_3_5} = 9.774 \cdot kip \cdot ft$$

Concrete - Footing

LOAD CASES 6 and 7: BASE PRESSURES w/ FACTORED LOADS

SUM OF VERTICAL FORCES

LC6 $sumV := 1.2\cdot(W_{c1} + W_{c2}) + 1.35\cdot(W_{s1} + W_{s2}) + (1.0\cdot Ww_{h1}) - (1.0\cdot UP)$ $sumV = 8.339\cdot kip$

LC7 $sumV_{f_MDF} := 1.2\cdot(W_{c1} + W_{c2}) + 1.35\cdot(W_{s1} + W_{s2}) + (1.3\cdot Ww_{h1}) - (1.3\cdot UP_{f_MDF})$ $sumV_{f_MDF} = 6.012\cdot kip$

SUM OF HORIZONTAL FORCES

If sumH < 0, then adjust force in F_{s2} as a triangle shape so sumH = 0.

LC6 $partA6 := 1.5\cdot P_{A_MDE} - 0.5\cdot P_{PE_MDE_rev} + 1.0\cdot(P_{ws}) + 1.25\cdot(\Delta P_{AE_MDE} + Inertia_{str_MDE})$ $partA6 = 5.221\cdot kip$

$partB6 := 1.5\cdot P_{A_MDE} + 1.0\cdot(P_{ws}) + 1.25\cdot(\Delta P_{AE_MDE} + Inertia_{str_MDE})$ $partB6 = 7.399\cdot kip$

$P_{PE_MDE_rev_new} := if[partA6 < 0\cdot kip, partB6, (0.5\cdot P_{PE_MDE_rev})]$ $P_{PE_MDE_rev_new} = 2.178\cdot kip$

$sumH_{MDE} := 1.5\cdot P_{A_MDE} - P_{PE_MDE_rev_new} + 1.0\cdot(P_{ws}) + 1.25\cdot(\Delta P_{AE_MDE} + Inertia_{str_MDE})$ $sumH_{MDE} = 5.221\cdot kip$

LC7 $F_{s2_f_MDF_new} := if[(1.5\cdot F_{s1} + 1.3\cdot Fw_{h1_f_MDF}) - (0.5\cdot F_{s2}) < 0\cdot kip, 1.5\cdot F_{s1} + 1.3\cdot Fw_{h1_f_MDF}, (0.5\cdot F_{s2})]$

$F_{s2_f_MDF_new} = 2.543\cdot kip$

$sumH_{f_MDF} := (1.5\cdot F_{s1} + 1.3\cdot Fw_{h1_f_MDF}) - (F_{s2_f_MDF_new})$ $sumH_{f_MDF} = 4.563\cdot kip$

RESISTING MOMENT (moments about point A)

$M_{R_conc} := W_{c1}\cdot(armW_{c1}) + W_{c2}\cdot(armW_{c2})$ $M_{R_conc} = 18.525\cdot kip\cdot ft$

$M_{R_soil} := W_{s1}\cdot(armW_{s1}) + W_{s2}\cdot(armW_{s2})$ $M_{R_soil} = 14.641\cdot kip\cdot ft$

$M_{R_water} := Ww_{h1}\cdot(armWw_{h1})$ $M_{R_water} = 11.719\cdot kip\cdot ft$

$M_{R6} := 1.2\cdot M_{R_conc} + 1.35\cdot M_{R_soil} + 1.0\cdot M_{R_water}$ $M_{R6} = 53.714\cdot kip\cdot ft$

$M_{R7} := 1.2\cdot M_{R_conc} + 1.35\cdot M_{R_soil} + 1.3\cdot M_{R_water}$ $M_{R7} = 57.229\cdot kip\cdot ft$

Concrete - Footing

OVERTURNING MOMENT (moments about point A) w/o Uplift

LC6 $M_{O_MDE} := 1.5\cdot\left[P_{A_MDE}\cdot\left(\frac{h}{3}\right)\right] - P_{PE_MDE_rev_new}\cdot\left(\frac{h_{rev_MDE}}{3}\right) + 1.0\cdot\left[P_{ws}\cdot\left(\frac{h_w}{3}\right)\right] \ldots$
$+ 1.25\cdot\left[\Delta P_{AE_MDE}\cdot\left(\frac{2\cdot h}{3}\right) + Inertia_{str_MDE}\cdot\left(armW_H\right)\right]$

$M_{O_MDE} = 22.764\cdot kip\cdot ft$

LC7 $M_{O_f_MDF} := 1.5\cdot F_{s1}\cdot\left(armF_{s1}\right) - F_{s2_f_MDF_new}\cdot\left(armF_{s2}\right) + 1.3\cdot Fw_{h1_f_MDF}\cdot\left(armFw_{h1_f_MDF}\right)$

$M_{O_f_MDF} = 19.390\cdot kip\cdot ft$

UPLIFT MOMENT (moments about point A)

$M6_{UP} := 1.0\cdot M_{UP}$ $M6_{UP} = 15.042\cdot kip\cdot ft$

$M7_{UP} := 1.3\cdot M_{UP_f_MDF}$ $M7_{UP} = 32.753\cdot kip\cdot ft$

BEARING CAPACITY

Ref. 6

R = Resultant from toe & e = eccentricity from base centroid

$R6 := \frac{M_{R6} - \left(M_{O_MDE} + M6_{UP}\right)}{sumV}$ $R7 := \frac{M_{R7} - \left(M_{O_f_MDF} + M7_{UP}\right)}{sumV_{f_MDF}}$

$R6 = 1.908\ ft$ $R7 = 0.846\ ft$

$e6 := \frac{base}{2} - R6$ $e7 := \frac{base}{2} - R7$

$e6 = 2.842\ ft$ $e7 = 3.904\ ft$

where $\frac{base}{3} = 3.167\ ft$ and $\frac{2\cdot base}{3} = 6.333\ ft$

$For6 := if\left[R6 > \left(\frac{1}{3}\cdot base\right), if\left[R6 > \left(\frac{2}{3}\cdot base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right]$ $For6 = \text{"Not In Kern"}$

$For7 := if\left[R7 > \left(\frac{1}{3}\cdot base\right), if\left[R7 > \left(\frac{2}{3}\cdot base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right]$ $For7 = \text{"Not In Kern"}$

Concrete - Footing

Percent Base In Compression

Table 4-1, Page 4-5.

$$\%Base_Comp6c := if\left[For6 = "In\ Kern",1,\left(\frac{3\cdot R6}{base}\right)\right] \qquad \%Base_Comp6c = 60.2\cdot\% \qquad Check(\%Base_Comp6c, 0.01) = "OK"$$

$$\%Base_Comp7c := if\left[For7 = "In\ Kern",1,\left(\frac{3\cdot R7}{base}\right)\right] \qquad \%Base_Comp7c = 26.7\cdot\% \qquad Check(\%Base_Comp7c, 0.01) = "OK"$$

Heel Pressure (Equ. 3-1)

$$Press6c_{Heel} := if\left[For6 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1-\frac{6\cdot e6}{base}\right),0\cdot psf\right] \qquad Press6c_{Heel} = 0\cdot psf$$

$$Press7c_{Heel} := if\left[For7 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1-\frac{6\cdot e7}{base}\right),0\cdot psf\right] \qquad Press7c_{Heel} = 0\cdot psf$$

Toe Pressure (Equ. 3-1 and 3-2)

$$Press6c_{Toe} := if\left[For6 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1+\frac{6\cdot e6}{base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV}{(base-2\cdot e6)\cdot b}\right]\right] \qquad Press6c_{Toe} = 2915\cdot psf$$

$$Press7c_{Toe} := if\left[For7 = "In\ Kern",\left(\frac{sumV}{base\cdot b}\right)\cdot\left(1+\frac{6\cdot e7}{base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV}{(base-2\cdot e7)\cdot b}\right]\right] \qquad Press7c_{Toe} = 6572\cdot psf$$

Base Pressure Length (Equ. 3-3)

$$Press6c_{Length} := if\left[For6 = "In\ Kern",base,\left(\frac{3}{2}\right)\cdot(base-2\cdot e6)\right] \qquad Press6c_{Length} = 5.723\ ft$$

$$Press7c_{Length} := if\left[For7 = "In\ Kern",base,\left(\frac{3}{2}\right)\cdot(base-2\cdot e7)\right] \qquad Press7c_{Length} = 2.538\ ft$$

Concrete - Footing

Base Pressure Upward Forces

-- at face of stem

$Press6c_{Heel} = 0 \cdot psf$ $\quad$ $Press7c_{Heel} = 0 \cdot psf$

$Press6c_{Toe} = 2915 \cdot psf$ $\quad$ $Press7c_{Toe} = 6572 \cdot psf$

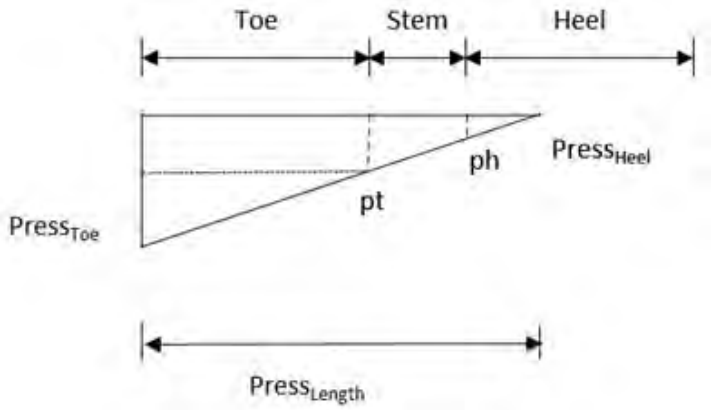
$Press6c_{Length} = 5.723\ ft$ $\quad$ $Press7c_{Length} = 2.538\ ft$

$$pt6 := Press6c_{Toe} + \left(\frac{b_{toe}}{Press6c_{Length}}\right) \cdot \left(Press6c_{Heel} - Press6c_{Toe}\right) \qquad pt6 = 0.877 \cdot ksf$$

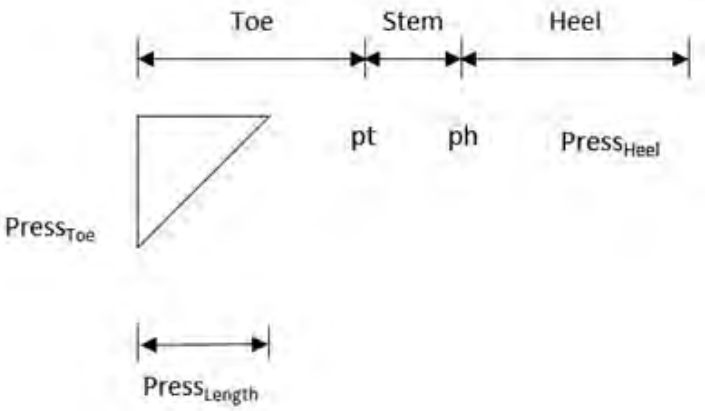
$$ph6 := Press6c_{Toe} + \left[\frac{\left(b_{toe} + t_{stem}\right)}{Press6c_{Length}}\right] \cdot \left(Press6c_{Heel} - Press6c_{Toe}\right) \qquad ph6 = 0.113 \cdot ksf$$

Note: pt7 and ph7 are N/A since pressure length is just on portion of toe.

Load Case 6



Load Case 7



Concrete - Footing

Pressure Forces and Moment Arms (measured from stem face):

$Ft_6 := b \cdot b_{toe} \cdot \left[\frac{1}{2} \cdot \left(Press6c_{Toe} + pt6\right)\right]$ $Ft_6 = 7.584 \cdot kip$

$Fh_6 := b \cdot b_{heel} \cdot \left[\frac{1}{2} \cdot \left(Press6c_{Heel} + ph6\right)\right]$ $Fh_6 = 0.227 \cdot kip$

$arm_t_6 := \frac{b_{toe} \cdot \left(2 \cdot Press6c_{Toe} + pt6\right)}{3 \cdot \left(Press6c_{Toe} + pt6\right)}$ $arm_t_6 = 2.36\,ft$

$arm_h_6 := \frac{\left(Press6c_{Length} - b_{toe} - t_{stem}\right) \cdot \left(2 \cdot Press6c_{Heel} + ph6\right)}{3 \cdot \left(Press6c_{Heel} + ph6\right)}$ $arm_h_6 = 0.07\,ft$

$Ft_7 := b \cdot Press7c_{Length} \cdot \left[\frac{1}{2} \cdot \left(Press7c_{Toe} + 0 \cdot psf\right)\right]$ $Ft_7 = 8.339 \cdot kip$

$Fh_7 := 0 \cdot kip$ $Fh_7 = 0.000 \cdot kip$

$arm_t_7 := b_{toe} - \frac{Press7c_{Length}}{3}$ $arm_t_7 = 3.15\,ft$

$arm_h_7 := 0 \cdot ft$ $arm_h_7 = 0.00 \cdot ft$

Concrete - Footing

LOAD CASES 6 and 7: FACTORED FOOTING VERTICAL FORCES @ STEM FACE -- for (1) heel and (2) toe

+ is downward for top reinforcement
- is upward for bottom reinforcement

Note: Fh and Ft already factored.

Heel

$LC6a_sumV_{heel} := 1.2 \cdot W_{ch} + 1.35 \cdot W_{s1} + 1.0 \cdot Ww_{h1} - Fh_6$ $LC6a_sumV_{heel} = 4.874 \cdot kip$

$LC7a_sumV_{heel} := 1.2 \cdot W_{ch} + 1.35 \cdot W_{s1} + 1.3 \cdot Ww_{h1} - Fh_7$ $LC7a_sumV_{heel} = 5.569 \cdot kip$

$LC6b_sumV_{heel} := 0.9 \cdot W_{ch} + 1.0 \cdot W_{s1} + 1.0 \cdot Ww_{h1} - Fh_6$ $LC6b_sumV_{heel} = 3.968 \cdot kip$

$LC7b_sumV_{heel} := 0.9 \cdot W_{ch} + 1.0 \cdot W_{s1} + 1.3 \cdot Ww_{h1} - Fh_7$ $LC7b_sumV_{heel} = 4.664 \cdot kip$

$V_{u_heel_6_7} := \max(|LC6a_sumV_{heel}|, |LC7a_sumV_{heel}|, |LC6b_sumV_{heel}|, |LC7b_sumV_{heel}|)$ $V_{u_heel_6_7} = 5.569 \cdot kip$

$LC6a_sumM_{heel} := (1.2 \cdot W_{ch} + 1.35 \cdot W_{s1} + 1.0 \cdot Ww_{h1}) \cdot (arm_CSW_h) - Fh_6 \cdot (arm_h_6)$ $LC6a_sumM_{heel} = 10.184 \cdot kip \cdot ft$

$LC7a_sumM_{heel} := (1.2 \cdot W_{ch} + 1.35 \cdot W_{s1} + 1.3 \cdot Ww_{h1}) \cdot (arm_CSW_h) - Fh_7 \cdot (arm_h_7)$ $LC7a_sumM_{heel} = 11.139 \cdot kip \cdot ft$

$LC6b_sumM_{heel} := (0.9 \cdot W_{ch} + 1.0 \cdot W_{s1} + 1.0 \cdot Ww_{h1}) \cdot (arm_CSW_h) - Fh_6 \cdot (arm_h_6)$ $LC6b_sumM_{heel} = 8.373 \cdot kip \cdot ft$

$LC7b_sumM_{heel} := (0.9 \cdot W_{ch} + 1.0 \cdot W_{s1} + 1.3 \cdot Ww_{h1}) \cdot (arm_CSW_h) - Fh_7 \cdot (arm_h_7)$ $LC7b_sumM_{heel} = 9.328 \cdot kip \cdot ft$

$M_{u_heel_6_7} := \max(|LC6a_sumM_{heel}|, |LC7a_sumM_{heel}|, |LC6b_sumM_{heel}|, |LC7b_sumM_{heel}|)$ $M_{u_heel_6_7} = 11.139 \cdot kip \cdot ft$

Concrete - Footing

Toe

$LC6a_sumV_{toe} := 1.2 \cdot W_{ct} + 1.35 \cdot W_{s2} - Ft_6$ $LC6a_sumV_{toe} = -4.130 \cdot kip$

$LC7a_sumV_{toe} := 1.2 \cdot W_{ct} + 1.35 \cdot W_{s2} - Ft_7$ $LC7a_sumV_{toe} = -4.886 \cdot kip$

$LC6b_sumV_{toe} := 0.9 \cdot W_{ct} + 1.0 \cdot W_{s2} - Ft_6$ $LC6b_sumV_{toe} = -5.014 \cdot kip$

$LC7b_sumV_{toe} := 0.9 \cdot W_{ct} + 1.0 \cdot W_{s2} - Ft_7$ $LC7b_sumV_{toe} = -5.769 \cdot kip$

$V_{u_toe_6_7} := \max(|LC6a_sumV_{toe}|, |LC7a_sumV_{toe}|, |LC6b_sumV_{toe}|, |LC7b_sumV_{toe}|)$ $V_{u_toe_6_7} = 5.769 \cdot kip$

$LC6a_sumM_{toe} := (1.2 \cdot W_{ct} + 1.35 \cdot W_{s2}) \cdot (arm_CSW_t) - Ft_6 \cdot (arm_t_6)$ $LC6a_sumM_{toe} = -10.976 \cdot kip \cdot ft$

$LC7a_sumM_{toe} := (1.2 \cdot W_{ct} + 1.35 \cdot W_{s2}) \cdot (arm_CSW_t) - Ft_7 \cdot (arm_t_7)$ $LC7a_sumM_{toe} = -19.395 \cdot kip \cdot ft$

$LC6b_sumM_{toe} := (0.9 \cdot W_{ct} + 1.0 \cdot W_{s2}) \cdot (arm_CSW_t) - Ft_6 \cdot (arm_t_6)$ $LC6b_sumM_{toe} = -12.744 \cdot kip \cdot ft$

$LC7b_sumM_{toe} := (0.9 \cdot W_{ct} + 1.0 \cdot W_{s2}) \cdot (arm_CSW_t) - Ft_7 \cdot (arm_t_7)$ $LC7b_sumM_{toe} = -21.163 \cdot kip \cdot ft$

$M_{u_toe_6_7} := \max(|LC6a_sumM_{toe}|, |LC7a_sumM_{toe}|, |LC6b_sumM_{toe}|, |LC7b_sumM_{toe}|)$ $M_{u_toe_6_7} = 21.163 \cdot kip \cdot ft$

Controlling Forces for LC 6 and 7

- maximum from these cases and all faces

Factored Shear:

$V_{u_ftg_6_7} := \max(V_{u_heel_6_7}, V_{u_toe_6_7})$ $V_{u_ftg_6_7} = 5.769 \cdot kip$

Factored Moment:

$M_{u_ftg_6_7} := \max(M_{u_heel_6_7}, M_{u_toe_6_7})$ $M_{u_ftg_6_7} = 21.163 \cdot kip \cdot ft$

Controlling Forces for LC 2, 3, 5, 6, and 7

- maximum from all cases and faces

Factored Shear:

$V_{u_ftg} := \max\left(V_{u_ftg_2_3_5}, V_{u_ftg_6_7}\right)$ $V_{u_ftg} = 5.769 \cdot kip$

Factored Moment:

$M_{u_ftg} := \max\left(M_{u_ftg_2_3_5}, M_{u_ftg_6_7}\right)$ $M_{u_ftg} = 21.163 \cdot kip \cdot ft$

Concrete

FOOTING REINFORCEMENT DESIGN

- Two Mats in Both Directions

Check Temperature Steel Requirement

COE (Ref. 3, Section 2.6): 12" center-to-center spacing of both primary and secondary bars.
COE (Ref. 3, Sections 2.9.1 and 2.9.2): 0.003 for Grade 60, with maximum area equivalent to #9 @ 12" in each face

$A_{temp_COE} := \frac{0.003 \cdot b \cdot t_{ftg}}{2}$ $A_{temp_COE} = 0.378 \cdot in^2$

#5 @ 9" OC => A = 0.409 in^2
#6 @ 12" OC => A = 0.442 in^2

Check Moment Steel Requirement

$d := t_{ftg} - (c + 1 \cdot in) - \frac{1}{2} \cdot \left(\frac{6}{8} in\right)$ (conservatively add 1" to cover) $d = 16.6250 \cdot in$

$R := \frac{M_{u_ftg}}{\phi M \cdot b \cdot d^2}$ $R = 85.07 \cdot psi$

$\rho := \frac{0.85 \cdot f'_c}{f_y} \cdot \left[1 - \sqrt{\left[1 - \left(\frac{2 \cdot R}{0.85 \cdot f'_c}\right)\right]}\right]$ $\rho = 0.00144$

Ref. 2, 9.6.1.2 $\rho_{min} := if\left[\left(\frac{200 \cdot psi}{f_y}\right) > \left(\frac{3\sqrt{f'_c \cdot psi}}{f_y}\right), \left(\frac{200 \cdot psi}{f_y}\right), \left(\frac{3\sqrt{f'_c \cdot psi}}{f_y}\right)\right]$ $\rho_{min} = 0.00333$

Ref. 2, 9.6.1.3 $\frac{4}{3} \cdot \rho = 0.00191$

$\rho\rho := if\left(\rho_{min} > \rho, if\left(\rho_{min} > \frac{4}{3} \cdot \rho, \frac{4}{3} \cdot \rho, \rho_{min}\right), \rho\right)$ $\rho\rho = 0.00191$

Ref. 3, Appendix C $\rho_{balanced} := \left(\frac{0.85 \cdot f'_c \cdot \beta_1}{f_y}\right) \cdot \left(\frac{87000 \cdot psi}{87000 \cdot psi + f_y}\right)$ $\rho_{balanced} = 0.02851$

$\rho_{max} := 0.25 \cdot \rho_{balanced}$ [Ref. 3, Chapter 3, Section 3.5] $\rho_{max} = 0.00713$

$rein_ratio := if(\rho_{max} > \rho\rho, "OK", "NG")$ $rein_ratio = "OK"$

Concrete

$A_s := \rho\rho \cdot b \cdot d$ $\quad A_s = 0.382 \cdot in^2$ / foot

$A_{smin} := \rho_{min} \cdot b \cdot d$ $\quad A_{smin} = 0.665 \cdot in^2$ / foot

****

BarNo := 6 $\quad$ B := BarNo $\quad$ * see temperature requirements

Spacing := 12·in

****

$$db := if\left(B = 4, \frac{4}{8}\cdot in, if\left(B = 5, \frac{5}{8}\cdot in, if\left(B = 6, \frac{6}{8}\cdot in, if\left(B = 7, \frac{7}{8}\cdot in, if\left(B = 8, \frac{8}{8}\cdot in, if(B = 9, 1.128\cdot in, 1.270\cdot in)\right)\right)\right)\right)\right)$$

$$BarArea := \frac{\pi \cdot db^2}{4} \qquad BarArea = 0.442 \cdot in^2$$

$$A_{sprov} := \frac{BarArea \cdot (12\cdot in)}{Spacing} \qquad A_{sprov} = 0.442 \cdot in^2 \text{ / foot}$$

Check: $A_{sprov} = 0.442 \cdot in^2$ / foot >? $A_s = 0.382 \cdot in^2$ / foot $\quad Check(A_{sprov}, A_s) = "OK"$

$$d_w := t_{ftg} - (c + 1\cdot in) - \frac{db}{2} \qquad d_w = 16.625 \cdot in$$

$$\phi M_n := \phi M \cdot \left[A_{sprov} \cdot f_y \cdot \left(d_w - \frac{a}{2}\right)\right] \qquad a := \frac{A_{sprov} \cdot f_y}{0.85 \cdot f'_c \cdot b} \qquad a = 0.650 \cdot in$$

$\phi M_n = 32.41 \cdot kip \cdot ft$ >? $M_{u_ftg} = 21.16 \cdot kip \cdot ft$ $\quad Check(\phi M_n, M_{u_ftg}) = "OK"$

SHEAR

Shear Strength of Normal-Weight Concrete (Ref. 2, 22.5.5.1)

$$\phi V_c := \phi V \cdot \left(2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d_w\right)$$

$\phi V_c = 18.93 \cdot kip$

$Check(\phi V_c, V_{u_ftg}) = "OK"$

Maximum Shear

$V_{u_ftg} = 5.77 \cdot kip$

Minimum Shear Reinforcement (Ref. 2, 7.6.3.2)
None required if M_n and V_n can be developed.

Summary

SUMMARY

LC 1 & 4: N/A
LC 2 - Usual: Normal Pool
LC 3 - Unusual: Infrequent Flood = Q100
LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
LC 6 - Extreme: Coincident Pool = Normal Pool with MDE
LC 7 - Extreme: Maximum Design Flood = Q10000

SLIDING SAFETY FACTORS

$FS2_{sliding}$ = "Stable" | $Check(FS2_{SL}, 1.7)$ = "OK"

$FS6_{sliding} = 1.34$ | $Check(FS6_{SL}, 1.1)$ = "OK"

$FS3_{sliding}$ = "Stable" | $Check(FS3_{SL}, 1.3)$ = "OK"

$FS7_{sliding} = 6.61$ | $Check(FS7_{SL}, 1.1)$ = "OK"

$FS5_{sliding} = 2.40 \times 10^{16}$ | $Check(FS5_{SL}, 1.3)$ = "OK"

$FS5_{sliding}$ = Stable

BEARING CAPACITY

$\%Base_Comp2 = 100.0 \cdot \%$ | $Check(\%Base_Comp2, 1)$ = "OK"

$\%Base_Comp3 = 100.0 \cdot \%$ | $Check(\%Base_Comp3, 0.75)$ = "OK"

$\%Base_Comp5 = 100.0 \cdot \%$ | $Check(\%Base_Comp5, 0.75)$ = "OK"

$\%Base_Comp6 = 71.9 \cdot \%$ | $Check(\%Base_Comp6, 0.01)$ = "OK"

$\%Base_Comp7 = 67.6 \cdot \%$ | $Check(\%Base_Comp7, 0.01)$ = "OK"

$BasePressure2 = 888 \cdot psf$ | $Check(BearingCap_{allow}, BasePressure2)$ = "OK"

$BasePressure3 = 1173 \cdot psf$ | $Check(BearingCap_{allow}, BasePressure3)$ = "OK"

$BasePressure5 = 1043 \cdot psf$ | $Check(BearingCap_{allow_eq}, BasePressure5)$ = "OK"

$BasePressure6 = 1874 \cdot psf$ | $Check(BearingCap_{allow_eq}, BasePressure6)$ = "OK"

$BasePressure7 = 1994 \cdot psf$ | $Check(BearingCap_{allow}, BasePressure7)$ = "OK"

REINFORCEMENT

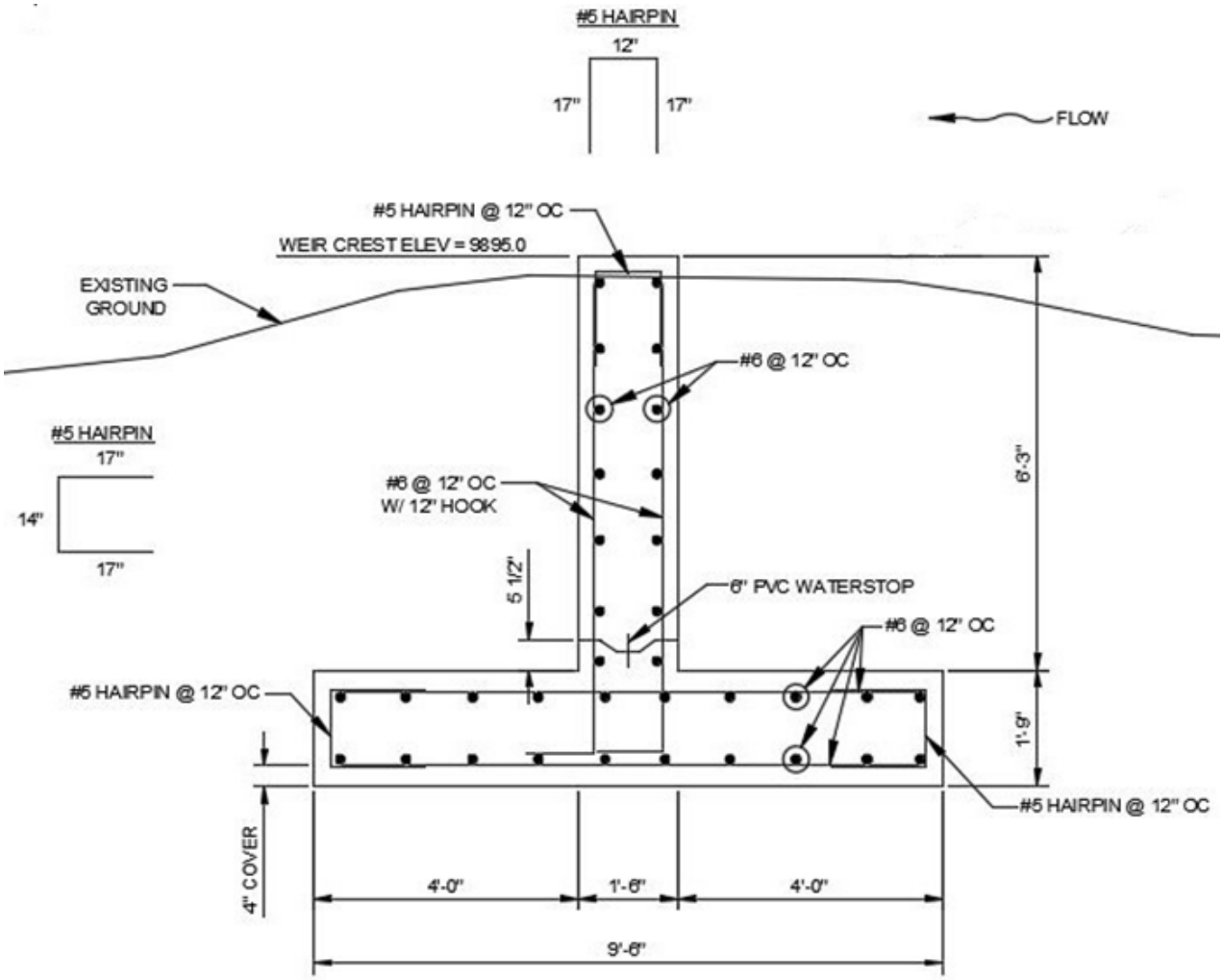
Stem: 1'-6" thick w/ #6@12" OC, EW, EF
Footing: 1'-9" thick w/ #6@12" OC, EW, EF

Ayres Associates Inc
Eau Claire, WI

Concrete Spillway (Wall)

Prepared By: TMR Oct 2020
Checked By: AEB Oct 2020

3" COVER UNLESS NOTED



SLIDE GATE TRASHRACK DESIGN

HOG CHUTE DAM - MESA COUNTY, COLORADO

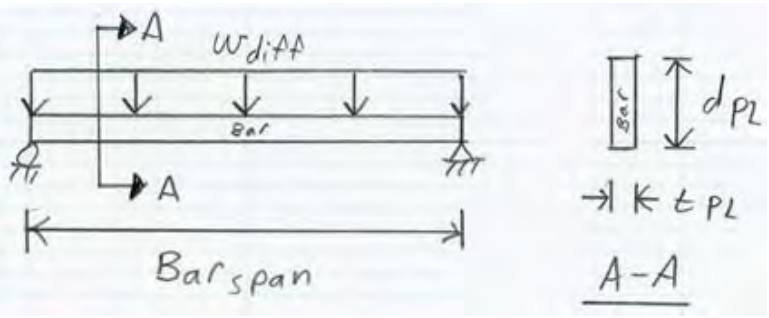
Ref.:
1) *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2) *Project Review Guide*, Colorado Dam Safety, 21 January 2020.
3) *Steel Construction Manual*, AISC, Fourteenth Edition, 2011.
4) *Specification for Structural Steel Buildings*, AISC, 7 July 2016.
5) *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-10.
6) *Design Standard No. 6: Hydraulic and Mechanical Equipment, Chapter 12 – Trashracks and Trashrack Cleaning Devices*, U.S. Department of the Interior, Bureau of Reclamation, Denver, December 2006.
7) *Steel Design Guide 1: Base Plate and Anchor Rod Design*, 1996, AISC, 2nd, 2006.
8) Ayres Drawings.

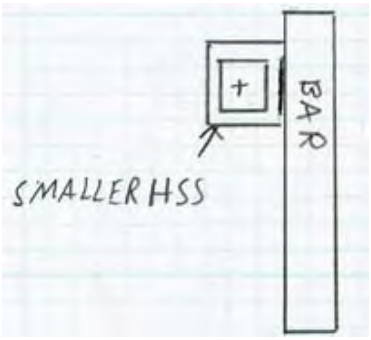
Summary

Per 'Whipps' Stainless Steel Slide Gate online Catalog, Model 955, Page 32:
Out-to-out frame = Width + 16"
Thickness of gate is unknown.
=> 36" gate so total is 52"
=> allow distance from gate frame to trashrack frame
=> use 5.0' clear span (allows 4" side clearance), add member thickness for bar design

Bar Load Figures 1 (left) and 2 (right)

- bar spacing not shown





Design Summary

1) Ref. 2: II-H2b (pg 31) [Taken from Ref. 7, 12.2.5]. Head is greater that 20 feet so use design differential of 20 feet.

2) Use AISC ASD per Ref. 4. See Ref. 7, 12.2.6.

3) Long bars are simply supported. Long (left figure) controls for bar design over cantilever (right figure) [see below].

Load Combinations

- Load Combinations for Allowable Stress (Ref. 5, 2.4.1)
  F is fluid load with same factor as dead load D.
  No other description so one controlling combination: 1. D

Data

Steel Density: (Ref. 3) $\gamma_{steel} := 490 \cdot pcf$

Water Density: (Ref. 3) $\gamma_{water} := 62.4 \cdot pcf$

Head Differential: $\Delta H := 20 \cdot ft$

Steel Properties

Plate: ASTM A572, Grade 50

Yield Strength: $F_y := 50 \cdot ksi$

Modulus Of Elasticity: $E := 29000 \cdot ksi$

Tensile Strength: $F_u := 65 \cdot ksi$

Define Check Function: $Check(x, y) := if(x \geq y, "OK", "NG")$

Bar Design

Simple supports.

Pressure: $\omega_{diff} := \gamma_{water} \cdot \Delta H$ $\omega_{diff} = 1248.0 \cdot psf$

Plate Properties (thickness and depth): $t_{PL} := \frac{3}{8} \cdot in$ $d_{PL} := 4 \cdot in$

Ref. 6, 12.2.7: $Check(d_{PL}, 4 \cdot t_{PL}) = "OK"$ $Check(12 \cdot t_{PL}, d_{PL}) = "OK"$

Bar Spacing: $Bar_{space} := 4 \cdot in + t_{PL}$ (centerline-to-centerline -> 4" clear)

Bar Span: $L_{bar} := 5 \cdot ft + 7 \cdot in$ (5.0' clear plus frame thickness for centerline)

Ref. 6, 12.2.7: $Check(70 \cdot t_{PL}, L_{bar}) = "NG"$ If NG, use round stock lateral supports between 18" and 30". Round stock is ASTM A36.

Braced Span: $Lb_{bar} := 0.75 \cdot L_{bar}$ Place round stock lateral supports at 1/3 points, but use 75% of full span as braced for moment capacity as stiffness of entire rack not verified

Ref. 6, 12.2.7: $Check\left(70 \cdot t_{PL}, \frac{L_{bar}}{3}\right) = "OK"$

Section Modulus: $S_{PL} := \frac{d_{PL}^2 \cdot t_{PL}}{6}$ $S_{PL} = 1.000 \cdot in^3$

Moment of Inertia: $I_{PL} := \frac{d_{PL}^3 \cdot t_{PL}}{12}$ $I_{PL} = 2.000 \cdot in^4$

Shear: $V_{PL} := \frac{\omega_{diff} \cdot (Bar_{space}) \cdot L_{bar}}{2} + \frac{\gamma_{steel} \cdot L_{bar} \cdot t_{PL} \cdot d_{PL}}{2}$

$V_{PL} = 1.284 \cdot kip$

Cantilever: (Does Not Control)

$V_{PL_cant} := \omega_{diff} \cdot (Bar_{space}) \cdot (2.0 \cdot ft) = 0.910 \cdot kip$

Moment: $M_{PL} := \frac{\omega_{diff} \cdot (Bar_{space}) \cdot L_{bar}^2}{8} + \frac{\gamma_{steel} \cdot L_{bar}^2 \cdot t_{PL} \cdot d_{PL}}{8}$

$M_{PL} = 21.515 \cdot kip \cdot in$

$M_{PL_cant} := \frac{\omega_{diff} \cdot (Bar_{space}) \cdot (2.0 \cdot ft)^2}{2} = 10.920 \cdot kip \cdot in$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

TRASHRACK DESIGN

Trash Rack.xmcd
Page 4 of 20

Prepared By: TMR Dec 2020
Checked By: AEB Jan 2021

FLEXURE

per Ref. 4: F11 – Page 16.1-65

1) Yielding - bent about major axis

$$ratio := \frac{Lb_{bar} \cdot d_{PL}}{t_{PL}^{2}} \qquad ratio = 1429$$

$$M_y := F_y \cdot S_{PL} \qquad M_y = 4.167 \cdot kip \cdot ft$$

$$Z := 1.5 \cdot S_{PL} \qquad Z = 1.500 \cdot in^3$$

(shape factor = 1.5
F11 comm., page 16.1-347)

$$M_{n1} := \min\left(F_y \cdot Z, 1.6 \cdot M_y\right) \qquad M_{n1} = 6.250 \cdot kip \cdot ft$$

2) Lateral-Torsional Buckling

$$C_b := 1.0$$

$$F_{cr} := \frac{1.9 \cdot E \cdot C_b}{ratio} \qquad F_{cr} = 38.55 \cdot ksi$$

$$M_{n2a} := C_b \cdot \left[1.52 - 0.274 \cdot (ratio) \cdot \frac{F_y}{E}\right] \cdot M_y \qquad M_{n2a} = 3.520 \cdot kip \cdot ft$$

$$M_n := \begin{cases} M_{n1} & \text{if } ratio \le \frac{0.08 \cdot E}{F_y} \\ \min\left(M_{n2a}, M_{n1}\right) & \text{if } \frac{0.08 \cdot E}{F_y} < ratio \le \frac{1.9 \cdot E}{F_y} \\ \min\left(F_{cr} \cdot S_{PL}, M_{n1}\right) & \text{if } ratio > \frac{1.9 \cdot E}{F_y} \end{cases}$$

$$\frac{0.08 \cdot E}{F_y} = 46.4$$

$$\frac{1.9 \cdot E}{F_y} = 1102$$

$$M_n = 3.212 \cdot kip \cdot ft$$

Allowable Flexural Strength (F1)

$\Omega_b := 1.67$ (ASD) $\qquad M_{allow} := \frac{M_n}{\Omega_b}$ $\qquad \boxed{M_{allow} = 23.083 \cdot kip \cdot in}$ $\qquad \boxed{Check\left(M_{allow}, M_{PL}\right) = \text{"OK"}}$

Ref. 6, 12.2.7, Equ 2: $\qquad \boxed{Check\left(0.6 \cdot F_y, \frac{M_{PL}}{S_{PL}}\right) = \text{"OK"}}$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

TRASHRACK DESIGN

Trash Rack.xmcd
Page 5 of 20

Prepared By: TMR Dec 2020
Checked By: AEB Jan 2021

SHEAR

per Ref. 4: G4 -- Page 16.1-74

Shear Depth: $shear_depth := d_{PL}$ $shear_depth = 4.000 \cdot in$

Web Shear Area: $A_w := shear_depth \cdot t_{PL}$ $A_w = 1.500 \cdot in^2$

Web Plate Buckling Coefficient: $k_v := 5$ [G4]

Web Shear Coefficient (G2.2):

$$C_{v2} := \begin{cases} 1.0 & \text{if } \frac{shear_depth}{t_{PL}} \le 1.10 \cdot \sqrt{\frac{k_v \cdot E}{F_y}} \\ \frac{1.10 \cdot \sqrt{\frac{k_v \cdot E}{F_y}}}{\left(\frac{shear_depth}{t_{PL}}\right)} & \text{if } \frac{shear_depth}{t_{PL}} > 1.10 \cdot \sqrt{\frac{k_v \cdot E}{F_y}} \end{cases}$$

$C_{v2} = 1.000$

$\frac{shear_depth}{t_{PL}} = 10.67$

$1.10 \cdot \sqrt{\frac{k_v \cdot E}{F_y}} = 59.24$

$\frac{1.10 \cdot \sqrt{\frac{k_v \cdot E}{F_y}}}{\left(\frac{shear_depth}{t_{PL}}\right)} = 5.553$

Nominal Shear Strength (Equ. G4-1)

$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v2}$ $V_n = 45.00 \cdot kip$

Allowable Shear Strength (G1)

$\Omega_v := 1.67$ (ASD) $V_{allow} := \frac{V_n}{\Omega_v}$ $V_{allow} = 26.95 \cdot kip$ $Check(V_{allow}, V_{PL}) = \text{"OK"}$

Deflection

$\Delta_{max} := \frac{5 \cdot \omega_{diff} \cdot Bar_{space} \cdot L_{bar}^4}{384 \cdot E \cdot I_{PL}}$ $\Delta_{max} = 0.172 \cdot in$

$Check\left(\frac{L_{bar}}{360}, \Delta_{max}\right) = \text{"OK"}$

Frame Design

Thickness of gate is unknown.
The bearing bars transfer load to a steel frame. Bearing bars span the shortest distance or parallel to the dam axis.
This frame is supported on 'columns' with a base plate that is then anchored to the concrete intake.

Gap between concrete and frame support may need additional bars to reduce gap.
Place lateral (side) differential head pressure on load frame member. This assumes entire area inside frame is 'dewatered' due to the 100% blockage. Place entire load on the frame members.

Use STAAD.Pro computer software for frame design with fixed members and simple supports.

Ref. 6, 12.2.5: "All trashracks that are installed must be designed to yield before any damage is sustained to the support structure if they ever become totally obstructed."

Therefore, increase loads by 5% in frame model. Use ASD design with 1.05 load combination.

Use ASTM A500, Grade C, Square or Rectangle shapes w/ Fy = 50 ksi and Fu = 62 ksi.

Use 5'-0" inside frame clear along width and 10'-0" inside frame clear along height.

Pressure: $\omega_{diff} = 1248.0 \cdot psf$

Frame Loads

TOP:
Use 5'-0" clear and 7" member thickness.
Members 1 to 3 and 11 to 13 in Y-direction.
Members 25 and 26 in Y-direction is member thickness.

$\omega_{frame_TOP} := \frac{1}{2} \cdot \omega_{diff} \cdot [5.0 \cdot ft + 2 \cdot (7 \cdot in)]$ $\omega_{frame_TOP} = 3.848 \cdot \frac{kip}{ft}$ (Y)

$\omega_{frame_TOP25} := \omega_{diff} \cdot (5 \cdot in)$ $\omega_{frame_TOP25} = 0.520 \cdot \frac{kip}{ft}$ (Y)

SIDE:
Use total height (excluding bearing bars) of 2.0'.
Will need bearing bars on perimeter to close gap between frame and concrete.

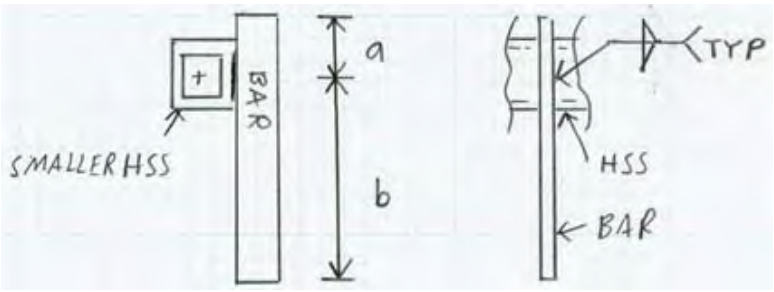
Uniform load on side based on 7" & 5" member thickness.
Members 1 to 3 and 11 to 13 in X-direction (7").
Members 25 and 26 have bars so full height in Z-direction. Include eccentric moment.

$\omega_{frame_SIDE1to3_11to13} := \omega_{diff} \cdot (7 \cdot in)$ $\omega_{frame_SIDE1to3_11to13} = 0.728 \cdot \frac{kip}{ft}$ (X)

$\omega_{frame_SIDE25} := \omega_{diff} \cdot (2 \cdot ft + 0 \cdot in)$ $\omega_{frame_SIDE25} = 2.496 \cdot \frac{kip}{ft}$ (Z)

$a_{25} := 3.5 \cdot in$ $b_{25} := 24 \cdot in - a_{25}$ $b_{25} = 20.5 \cdot in$

$M_{X_frame_25} := \omega_{diff} \cdot (1 \cdot ft) \cdot \left(\frac{b_{25}^2}{2} - \frac{a_{25}^2}{2} \right)$ $M_{X_frame_25} = 1.768 \cdot kip \cdot ft$ (per ft of width)



Smaller HSS - Members 25 and 26

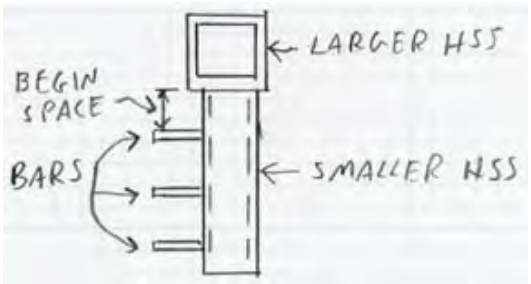
SUPPORTS:
Place reactions on support from side bearing bars (X-direction only).
Use 3'-9" for end spans and 3'-6.5" for middle span.
Members 201 to 204 and 211 to 214.

$\omega_{support_SIDE_X_201_204} := \omega_{diff} \cdot \left(\frac{3 \cdot ft + 9 \cdot in}{2} \right)$ $\omega_{support_SIDE_X_201_204} = 2.340 \cdot \frac{kip}{ft}$

$\omega_{support_SIDE_X_202_203} := \omega_{diff} \cdot \left(\frac{3 \cdot ft + 9 \cdot in}{2} + \frac{3 \cdot ft + 6.5 \cdot in}{2} \right)$ $\omega_{support_SIDE_X_202_203} = 4.550 \cdot \frac{kip}{ft}$

$\omega_{support_SIDE_Z_201_211} := \omega_{diff} \cdot (5 \cdot in + 2 \cdot in)$ $\omega_{support_SIDE_Z_201_211} = 0.728 \cdot \frac{kip}{ft}$

(5" HSS thickness plus about 2" tributary)



Section of a Support - Members 201 to 204 and 211 to 214

NOTES:
LC 1: Use full forces on all exterior faces.
LC 2: Use half of force on two sides, full else.
Other partial loadings N/A due to greater equalization of pressure.

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Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

TRASHRACK DESIGN

Trash Rack.xmcd
Page 8 of 20

Prepared By: TMR Dec 2020
Checked By: AEB Jan 2021

BASE PLATE & BEARING

-- Ref. 3, Page 14-6 and Ref. 7, Section 3.1.3, Page 16.
-- Axially loaded only.
-- per Ref. 8: m and n are 0.95 times the depth and width of HSS. λ term is not used.

Axial: $P_{ac} := 15.28 \cdot kip$ (JOINTS 2 & 12 w/ 1.05 LF)

Width 'B' along beam bf: $B := 9 \cdot in$ Beam Depth: $d := 5 \cdot in$ (largest)

Length 'N' along beam d: $N := 11 \cdot in$ Beam Width: $b := 5 \cdot in$

Base Plate Thickness: $t := \frac{1}{2} \cdot in$ (1/2" minimum to match member wall thickness)

Plate Yield Strength: $F_{yPL} := 50 \cdot ksi$

Critical Cantilever Dimension:

$m := \frac{N - 0.95 \cdot d}{2} = 3.13 \cdot in$ $n := \frac{B - 0.95 \cdot b}{2} = 2.13 \cdot in$

$l := \max(m, n) = 3.13 \cdot in$

Minimum Thickness:

$t_{min} := l \cdot \left(\sqrt{\frac{3.33 \cdot P_{ac}}{F_{yPL} \cdot B \cdot N}} \right)$ $t_{min} = 0.317 \cdot in$ $\boxed{Check(t, t_{min}) = "OK"}$

Allowable Bearing Pressure [Conservatively use only 1" on each side of column for base side.]
Ref.: *Building Code Requirements For Structural Concrete (ACI 318-14) And Commentary (ACI 318R-14)*

Allowable bearing strength: ACI 14.5.6.1: ϕ(0.85f'c) where ϕ = 0.65 (ACI 21.2.1)
Live Load (LL) Factor = 1.6 (ACI Table 5.3.1), (0.65)(0.85)/1.6 = 0.345 => $0.35f'_c$

Concrete Compressive Strength: $f'_c := 4000 \cdot psi$

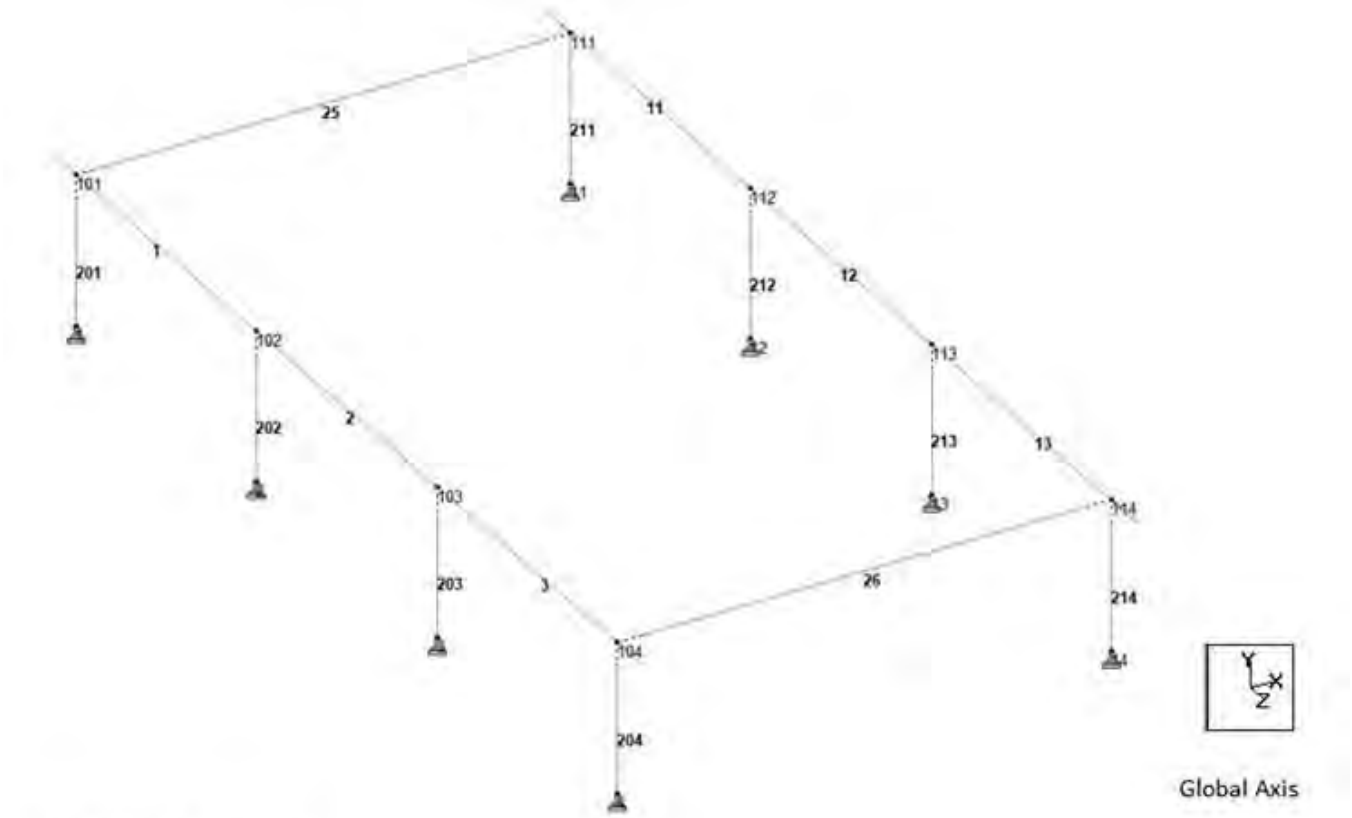
$Bearing_{Allow} := 0.35 \cdot f'_c$ $Bearing_{Allow} = 1400\,psi$

$Pressure := \frac{P_{ac}}{(d + 2 \cdot in) \cdot (b + 2 \cdot in)}$ $Pressure = 312\,psi$ $\boxed{Check(Bearing_{Allow}, Pressure) = "OK"}$

or

$P_p := 0.85 \cdot f'_c \cdot (d + 2 \cdot in) \cdot (b + 2 \cdot in)$ $P_p = 166.60 \cdot kip$ (Equ. J8-1)

$\Omega_c := 2.31$ (ASD) $P_{p_allow} := \frac{P_p}{\Omega_c}$ $\boxed{P_{p_allow} = 72.12 \cdot kip}$ $\boxed{Check(P_{p_allow}, P_{ac}) = "OK"}$



STAAD.Pro File: Frame.std

	26 (LC 101) *JT 104/114*	26 (LC102) *JT 114*	201 (LC101) *JT 101*	202 (LC102) *JT 101*	203 (LC102) *JT 103*	204 (LC102) *JT 104*	214 (LC102) *JT 114*
Axial (kip)	9.33	8.55	10.35	15.22	14.90	9.32	10.60
VY (kip)	1.44	2.09	3.89	0.55	0.86	3.06	3.73
VZ (kip)	6.57	6.39	0.83	0.49	1.03	1.25	1.51
Torsion (k-in)	55.88	55.90	0.00	0.00	0.00	0.00	0.00
MY (k-in)	70.70	63.13	23.27	8.32	17.59	30.58	30.32
MZ (k-in)	7.52	27.11	36.60	48.37	43.00	22.38	48.61

Frame Force Summary (from STAAD.Pro)

HSS-to-HSS Moment Connections

Ref.: *Design Guide No. 24: Hollow Structural Section Connections*, AISC, March 2015.

HSS 5x5x0.5 to HSS 7x7x0.5 -- ASTM A500, Grade C

$H := 7\cdot in$ $H_b := 5\cdot in$ $F_y := 50\cdot ksi$ $E = 29000\cdot ksi$

$B := H$ $B_b := H_b$ $F_u := 62\cdot ksi$ $\theta := 90\cdot deg$

$t := 0.465\cdot in$ $t_b := 0.465\cdot in$ t is design thickness

$A := 11.6\cdot in^2$ $A_b := 7.88\cdot in^2$

$S := 23.0\cdot in^3$ $S_b := 10.4\cdot in^3$ $\eta := \frac{H_b}{B}$ $\eta = 0.71$

$\beta := \frac{B_b}{B}$ $\beta = 0.71$ < 0.85 so limits required Checks

STAAD.Pro Model: Frame Member 26, Joint 114

-- Computation only shown for controlling case. See figure for force summary.

$Axial1 := 8.55\cdot kip$ $T1 := 55.90\cdot kip\cdot in$ (Both Planes)

$VY1 := 2.09\cdot kip$ (VY = Out-Of-Plane) $MY1 := 63.13\cdot kip\cdot in$ (MY = In-Plane)

$VZ1 := 6.39\cdot kip$ (VZ = In-Plane) $MZ1 := 27.11\cdot kip\cdot in$ (MZ = Out-Of-Plane)

Shear due to Torsion for In-Plane and Out-Of-Plane. Add V to Axial forces.

$VY1_T := \frac{1}{2}\cdot\left(\frac{T1}{\min(H_b, B_b)}\right)$ $VY1_T = 5.59\cdot kip$ $VZ1_T := VY1_T$ $VZ1_T = 5.59\cdot kip$

$V1 := \sqrt{(VY1 + VY1_T)^2 + (VZ1 + VZ1_T)^2}$ $V1 = 14.23\cdot kip$

STAAD.Pro Model: Support Member 214, Joint 114

-- Computation only shown for controlling case. See figure for force summary.

$Axial2 := 10.60 \cdot kip$ $T2 := 0.00 \cdot kip \cdot in$ (Both Planes)

$VY2 := 3.73 \cdot kip$ (VY = Out-Of-Plane) $MY2 := 30.32 \cdot kip \cdot in$ (MY = In-Plane)

$VZ2 := 1.51 \cdot kip$ (VZ = In-Plane) $MZ2 := 48.61 \cdot kip \cdot in$ (MZ = Out-Of-Plane)

Shear due to Torsion for In-Plane and Out-Of-Plane. Add V to Axial forces.

$VY2_T := \frac{1}{2} \cdot \left(\frac{T2}{\min(H_b, B_b)} \right)$ $VY2_T = 0.00 \cdot kip$ $VZ2_T := VY2_T$ $VZ2_T = 0.00 \cdot kip$

$V2 := \sqrt{(VY2 + VY2_T)^2 + (VZ2 + VZ2_T)^2}$ $V2 = 4.02 \cdot kip$

Tables 8-2 and 9-2 - Limits of Applicability
from Tables 8-2A and 9-2A

Branch Angle: Table 8-2
"OK" since angle is 90 deg. for Table 9-2

$Check(\theta, 30 \cdot deg) = "OK"$

Chord Wall Slenderness:

$Check\left(35, \frac{B}{t}\right) = "OK"$ $Check\left(35, \frac{H}{t}\right) = "OK"$

Branch Wall Slenderness:

$Check\left(35, \frac{B_b}{t_b}\right) = "OK"$ $Check\left(35, \frac{H_b}{t_b}\right) = "OK"$

$Check\left(1.25 \cdot \sqrt{\frac{E}{F_y}}, \frac{B_b}{t_b}\right) = "OK"$ $Check\left(1.25 \cdot \sqrt{\frac{E}{F_y}}, \frac{H_b}{t_b}\right) = "OK"$

Width Ratio:

$Check\left(\frac{B_b}{B}, 0.25\right) = "OK"$

Aspect Ratio:

$Check\left(\frac{H}{B}, 0.5\right) = "OK"$ $Check\left(2.0, \frac{H}{B}\right) = "OK"$

$Check\left(\frac{H_b}{B_b}, 0.5\right) = "OK"$ $Check\left(2.0, \frac{H_b}{B_b}\right) = "OK"$

Material Strength and Ductility:

"OK" since ASTM A500, Grade C

Table 8-2. Nominal Strengths of Rectangular HSS-to-HSS Truss Connections -> Type T

(Equs. K2-14, K2-15, K2-16, K2-17, K2-18 & K2-19 are N/A due to β)

Chord Wall Plastification

P_r = Axial M_r = MY or MZ $F_c = 0.6F_y$

$$U := \frac{\max(Axial1 + V1, Axial2 + V2)}{A\cdot(0.6\cdot F_y)} + \frac{\max(MY1, MZ1, MY2, MZ2)}{S\cdot(0.6\cdot F_y)} \qquad U = 0.16 \qquad \text{(Equ. K2-12)}$$

$$Q_f := \min\left[1.0, 1.3 - 0.4\cdot\left(\frac{U}{\beta}\right)\right] \qquad Q_f = 1.00 \qquad \text{for chord (connecting surface)} \qquad \text{(Equ. K2-10)}$$

$$P_{n_cwp} := \left(\frac{1}{\sin(\theta)}\right)\cdot F_y\cdot t^2\cdot\left[\frac{2\cdot\eta}{(1-\beta)} + \frac{4}{\sqrt{1-\beta}}\right]\cdot Q_f \qquad \boxed{P_{n_cwp} = 134.96\cdot kip} \qquad \text{(Equ. K2-13)}$$

$$\boxed{Check\left(P_{n_cwp}, \max(Axial1 + V1, Axial2 + V2)\right) = "OK"}$$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

TRASHRACK DESIGN

Trash Rack.xmcd
Page 13 of 20

Prepared By: TMR Dec 2020
Checked By: AEB Jan 2021

Table 9-2. Nominal Capacities of Rectangular HSS-to-HSS Moment Connections -> Type T

(Equs. K3-12, K3-13, K3-16 & K3-17 are N/A due to β)

P_r = Axial M_r = MY or MZ $F_c = 0.6F_y$

$$U := \frac{\max(Axial1 + V1, Axial2 + V2)}{A\cdot(0.6\cdot F_y)} + \frac{\max(MY1, MZ1, MY2, MZ2)}{S\cdot(0.6\cdot F_y)} \qquad U = 0.16$$ (Equ. K3-10)

$$Q_f := \min\left[1.0, 1.3 - 0.4\cdot\left(\frac{U}{\beta}\right)\right] \qquad Q_f = 1.00$$ (Equ. K3-9)

for chord (connecting surface)
Compute for controlling plane
and use for all.

$\Omega_{b_all} := 1.50$ (ASD)

Chord Wall Plastification – In-Plane Bending

$$M_{n_K3.11} := F_y\cdot t^2\cdot H_b\cdot\left[\frac{1}{2\cdot\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{(1-\beta)}\right]\cdot Q_f \qquad M_{n_K3.11} = 375.2\cdot kip\cdot in$$ (Equ. K3-11)

Chord Wall Plastification – Out-of-Plane Bending

$$M_{n_K3.15} := F_y\cdot t^2\cdot\left[\frac{0.5\cdot H_b\cdot(1+\beta)}{(1-\beta)} + \sqrt{\frac{2\cdot B\cdot B_b\cdot(1+\beta)}{(1-\beta)}}\right]\cdot Q_f \qquad M_{n_K3.15} = 383.7\cdot kip\cdot in$$ (Equ. K3-15)

Chord Distortional Failure

$$M_{n_K3.19} := 2\cdot F_y\cdot t\cdot\left[H_b\cdot t + \sqrt{B\cdot H\cdot t\cdot(B+H)}\right] \qquad M_{n_K3.19} = 938.6\cdot kip\cdot in$$ (Equ. K3-19)

Allowable Moment Capacity

$$M_{allow_K3} := \min\left(\frac{M_{n_K3.11}}{\Omega_{b_all}}, \frac{M_{n_K3.15}}{\Omega_{b_all}}, \frac{M_{n_K3.19}}{\Omega_{b_all}}\right) \qquad M_{allow_K3} = 250.2\cdot kip\cdot in$$

$Check(M_{allow_K3}, \max(MY1, MZ1, MY2, MZ2)) = "OK"$

Combination - Interaction

$$interaction1 := \left(\frac{Axial1 + V1}{P_{n_cwp}}\right) + \left(\frac{MY1}{\frac{M_{n_K3.11}}{\Omega_{b_all}}}\right) + \left(\frac{MZ1}{\frac{M_{n_K3.15}}{\Omega_{b_all}}}\right) \qquad interaction1 = 0.527$$ (Equ. K3-21)

$Check(1.0, interaction1) = "OK"$

$$interaction2 := \left(\frac{Axial2 + V2}{P_{n_cwp}}\right) + \left(\frac{MY2}{\frac{M_{n_K3.11}}{\Omega_{b_all}}}\right) + \left(\frac{MZ2}{\frac{M_{n_K3.15}}{\Omega_{b_all}}}\right) \qquad interaction2 = 0.420$$ (Equ. K3-21)

$Check(1.0, interaction2) = "OK"$

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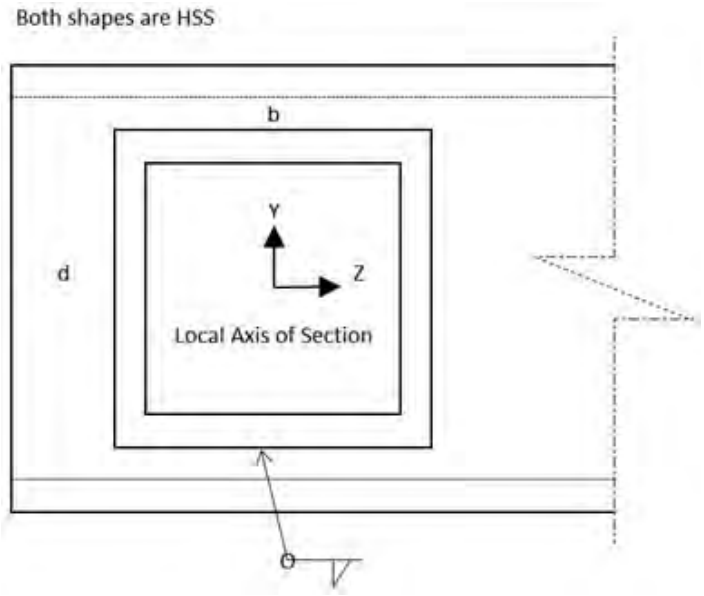
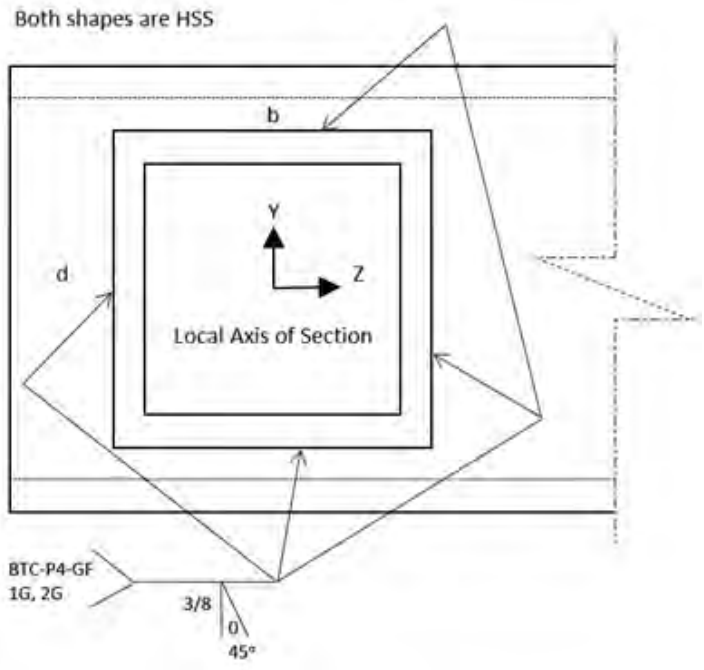
Check Weld of Smaller HSS to Larger HSS

Ref. 4: K5. Welds of Plates and Branches to Rectangular HSS



Support Connection Detail (smaller HSS to larger HSS) - Fillet



Frame Connection Detail (smaller HSS to larger HSS) - Prequalified Partial Penetration Groove Weld
Ref. 3, Table 8-2, Page 8-55

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

TRASHRACK DESIGN

Trash Rack.xmcd
Page 15 of 20

Prepared By: TMR Dec 2020
Checked By: AEB Jan 2021

Frame LOAD 1 – Table K5.1 - Effective Weld Properties for Connections to Retangular HSS

$\theta := 90 \cdot deg$ $\beta := \frac{B_b}{B}$ $\beta = 0.71$ $\gamma := \frac{B}{2 \cdot t}$ $\gamma = 7.53$

$t1_w := \frac{3}{8} \cdot in$ <-- effective partial penetration groove weld throat

$B_e := \min\left[B_b, B_b \cdot \left(\frac{10 \cdot t}{B}\right) \cdot \left(\frac{F_y \cdot t}{F_y \cdot t_b}\right)\right]$ $B_e = 3.32 \cdot in$ (Equ. K1-1)

$B_{ee} := if\left(\beta > 0.85 \vee \theta > 50 \cdot deg, \min\left(\frac{B_e}{2}, \frac{B_b}{4}\right), B_e\right)$ $B_{ee} = 1.25 \cdot in$

$L1_e := \frac{2 \cdot H_b}{\sin(\theta)} + 2 \cdot B_{ee}$ $L1_e = 12.50 \cdot in$ (Equ. K5-5)

$S1_{ip} := \left(\frac{t1_w}{3}\right) \cdot \left(\frac{H_b}{\sin(\theta)}\right)^2 + t1_w \cdot B_{ee} \cdot \left(\frac{H_b}{\sin(\theta)}\right)$ $S1_{ip} = 5.469 \cdot in^3$ (Equ. K5-6)

$S1_{op} := t1_w \cdot \left(\frac{H_b}{\sin(\theta)}\right) \cdot B_b + \left(\frac{t1_w}{3}\right) \cdot B_b^2 + t1_w \cdot B_{ee} \cdot \left(\frac{H_b}{\sin(\theta)}\right) - \frac{\left(\frac{t1_w}{3}\right) \cdot (B_b - B_{ee})^3}{B_b}$ $S1_{op} = 13.525 \cdot in^3$ (Equ. K5-7)

- Nominal Stress of Weld Metal (Table J2.5, pg. 16.1-124)

Electrode: $F_{EXX} := 70 \cdot ksi$ Stress: $F_{nw} := 0.60 \cdot F_{EXX}$ $F_{nw} = 42.00 \cdot ksi$

$\Omega_w := 2.00$ (ASD) $Pa_n := \frac{F_{nw} \cdot t1_w \cdot L1_e}{\Omega_w}$ $Pa_n = 98.44 \cdot kip$ (Equ. K5-1)

$Check(Pa_n, Axial1) = "OK"$

$Ma_{n_ip} := \frac{F_{nw} \cdot S1_{ip}}{\Omega_w}$ $Ma_{n_ip} = 114.84 \cdot kip \cdot in$ (Equ. K5-2)

$Check(Ma_{n_ip}, MY1) = "OK"$

$Ma_{n_op} := \frac{F_{nw} \cdot S1_{op}}{\Omega_w}$ $Ma_{n_op} = 284.03 \cdot kip \cdot in$ (Equ. K5-3)

$Check(Ma_{n_op}, MZ1) = "OK"$

$interaction_weld1 := \left(\frac{Axial1 + V1}{Pa_n}\right) + \left(\frac{MY1}{Ma_{n_ip}}\right) + \left(\frac{MZ1}{Ma_{n_op}}\right)$ $interaction_weld1 = 0.877$

$Check(1.0, interaction_weld1) = "OK"$

Note: 5/16" Single Pass Fillet Weld is NG

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

TRASHRACK DESIGN

Trash Rack.xmcd
Page 16 of 20

Prepared By: TMR Dec 2020
Checked By: AEB Jan 2021

Support LOAD 2 -- Table K5.1 - Effective Weld Properties for Connections to Retangular HSS

$\theta := 90 \cdot deg$

$\beta := \frac{B_b}{B}$ $\beta = 0.71$ $\gamma := \frac{B}{2 \cdot t}$ $\gamma = 7.53$

$t2_w := \left(\frac{\sqrt{2}}{2}\right) \cdot \left(\frac{5}{16} \cdot in\right)$ <-- effective fillet weld throat

$B_e := \min\left[B_b, B_b \cdot \left(\frac{10 \cdot t}{B}\right) \cdot \left(\frac{F_y \cdot t}{F_y \cdot t_b}\right)\right]$ $B_e = 3.32 \cdot in$ (Equ. K1-1)

$B_{ee} := if\left(\beta > 0.85 \vee \theta > 50 \cdot deg, \min\left(\frac{B_e}{2}, \frac{B_b}{4}\right), B_e\right)$ $B_{ee} = 1.25 \cdot in$

$L2_e := \frac{2 \cdot H_b}{\sin(\theta)} + 2 \cdot B_{ee}$ $L2_e = 12.50 \cdot in$ (Equ. K5-5)

$S2_{ip} := \left(\frac{t2_w}{3}\right) \cdot \left(\frac{H_b}{\sin(\theta)}\right)^2 + t2_w \cdot B_{ee} \cdot \left(\frac{H_b}{\sin(\theta)}\right)$ $S2_{ip} = 3.222 \cdot in^3$ (Equ. K5-6)

$S2_{op} := t2_w \cdot \left(\frac{H_b}{\sin(\theta)}\right) \cdot B_b + \left(\frac{t2_w}{3}\right) \cdot {B_b}^2 + t2_w \cdot B_{ee} \cdot \left(\frac{H_b}{\sin(\theta)}\right) - \frac{\left(\frac{t2_w}{3}\right) \cdot (B_b - B_{ee})^3}{B_b}$ $S2_{op} = 7.970 \cdot in^3$ (Equ. K5-7)

- Nominal Stress of Weld Metal (Table J2.5, pg. 16.1-124)

Electrode: $F_{EXX} := 70 \cdot ksi$ Stress: $F_{nw} := 0.60 \cdot F_{EXX}$ $F_{nw} = 42.00 \cdot ksi$

$\Omega_w := 2.00$ (ASD) $Pa_n := \frac{F_{nw} \cdot t2_w \cdot L2_e}{\Omega_w}$ $Pa_n = 58.00 \cdot kip$ (Equ. K5-1)

$Check(Pa_n, Axial2) = "OK"$

$Ma_{n_ip} := \frac{F_{nw} \cdot S2_{ip}}{\Omega_w}$ $Ma_{n_ip} = 67.67 \cdot kip \cdot in$ (Equ. K5-2)

$Check(Ma_{n_ip}, MY2) = "OK"$

$Ma_{n_op} := \frac{F_{nw} \cdot S2_{op}}{\Omega_w}$ $Ma_{n_op} = 167.37 \cdot kip \cdot in$ (Equ. K5-3)

$Check(Ma_{n_op}, MZ2) = "OK"$

$interaction_weld2 := \left(\frac{Axial2 + V2}{Pa_n}\right) + \left(\frac{MY2}{Ma_{n_ip}}\right) + \left(\frac{MZ2}{Ma_{n_op}}\right)$ $interaction_weld2 = 0.991$

$Check(1.0, interaction_weld2) = "OK"$

Alternate Fillet Weld On Smaller HSS Considering Moment and Shear In

- assumes full cross-section is effective

- Weld Strength for Shear (Tension N/A) (Table J2.5, pg. 16.1-124)

Electrode: $F_{EXX} = 70 \cdot ksi$

$F_w := 0.60 \cdot F_{EXX}$ $F_w = 42.00 \cdot ksi$ $\Omega_V := 2.00$ (ASD)

$Weld_Allow_V := \frac{F_w}{\Omega_V}$ $Weld_Allow_V = 21.00 \cdot ksi$

- Base Metal Strength

Shear (J4.2, page 16.1-137):

a) Yielding $R_{n_J4.2a} := 0.60 \cdot F_y$ $R_{n_J4.2a} = 30 \cdot ksi$ $\Omega_{J4.2a} := 1.50$ (ASD)

$Allow_R_{n_J4.2a} := \frac{R_{n_J4.2a}}{\Omega_{J4.2a}}$ $Allow_R_{n_J4.2a} = 20.00 \cdot ksi$

b) Rupture $R_{n_J4.2b} := 0.60 \cdot F_u$ $R_{n_J4.2b} = 37.2 \cdot ksi$ $\Omega_{J4.2b} := 2.00$ (ASD)

$Allow_R_{n_J4.2b} := \frac{R_{n_J4.2b}}{\Omega_{J4.2b}}$ $Allow_R_{n_J4.2b} = 18.60 \cdot ksi$

$Base_Allow_V := \min(Allow_R_{n_J4.2a}, Allow_R_{n_J4.2b})$ $Base_Allow_V = 18.60 \cdot ksi$

Minimum Fillet Weld On Web (Table J2.4)

To 1/4" inclusive: 1/8"
Over 1/4" to 1/2": 3/16"
Over 1/2" to 3/4": 1/4"
Over 3/4": 5/16"

Weld Size, a_w $a_w := \frac{1}{4} \cdot in$

$R_{nwm_allow} := (1) \cdot \left(\frac{\sqrt{2}}{2}\right) \cdot a_w \cdot Weld_Allow_V$ $R_{nwm_allow} = 3.712 \cdot \frac{kip}{in}$ x 1 since one side

Base Metal: Controlling Metal Thickness, t $t_{contol} := \min(t, t_b)$ $t_{contol} = 0.465 \cdot in$

$R_{nbm_allow} := t_{contol} \cdot (Base_Allow_V)$ $R_{nbm_allow} = 8.649 \cdot \frac{kip}{in}$

Design Strength: $R_{nw} := \min(R_{nwm_allow}, R_{nbm_allow})$ $R_{nw} = 3.712 \cdot \frac{kip}{in}$

Weld Properties

$B_b = 5 \cdot in \qquad H_b = 5 \cdot in$

Length and Section Based on 1" unit Width:

$L_w := \min(2 \cdot B_b + 2 \cdot H_b, L1_e) \qquad L_w = 12.50 \cdot in$

$S_w := \min\left[\left(B_b \cdot H_b + \frac{H_b^2}{3}\right), \left[\left(\frac{H_b}{\sin(\theta)}\right)^2 + B_{ee} \cdot \left(\frac{H_b}{\sin(\theta)}\right)\right]\right] \qquad S_w = 31.25 \cdot in^2$

Weld Forces 1

$R1_x := \frac{Axial1}{L_w} \qquad R1_x = 0.684 \cdot \frac{kip}{in}$

$R1_y := \frac{VY1}{L_w} \qquad R1_y = 0.167 \cdot \frac{kip}{in} \qquad R1_{yy} := \frac{MZ1}{S_w} \qquad R1_{yy} = 0.868 \cdot \frac{kip}{in}$

$R1_z := \frac{VZ1}{L_w} \qquad R1_z = 0.511 \cdot \frac{kip}{in} \qquad R1_{zz} := \frac{MY1}{S_w} \qquad R1_{zz} = 2.02 \cdot \frac{kip}{in}$

$R1_{yyy} := \frac{VY1_T}{L_w} \qquad R1_{yyy} = 0.447 \cdot \frac{kip}{in} \qquad R1_{zzz} := \frac{VZ1_T}{L_w} \qquad R1_{zzz} = 0.447 \cdot \frac{kip}{in}$

Resultant: $R1_w := \sqrt{(R1_x)^2 + (R1_y + R1_{yy} + R1_{yyy})^2 + (R1_z + R1_{zz} + R1_{zzz})^2} \qquad \boxed{R1_w = 3.396 \cdot \frac{kip}{in}}$

$\boxed{Check\left[\left(R_{nw}\right), R1_w\right] = \text{"OK"}}$

Frame long members: HSS 7x7x0.5
Frame short members and support: HSS 5x5x0.5
5/16" fillet weld on supports to long frame members and supports to base plate.
3/8" partial penetration groove weld on short frame member to long frame member.

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

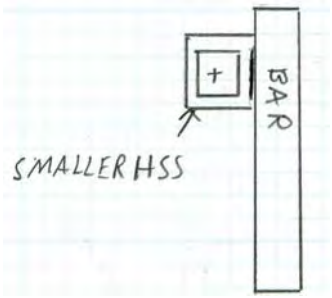
TRASHRACK DESIGN

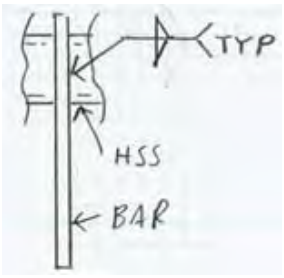
Trash Rack.xmcd
Page 19 of 20

Prepared By: TMR Dec 2020
Checked By: AEB Jan 2021

Cantilever Bar Weld

- assume maximum forces





Minimum Fillet Weld On Web (Table J2.4)

To 1/4" inclusive: 1/8"
Over 1/4" to 1/2": 3/16"
Over 1/2" to 3/4": 1/4"
Over 3/4": 5/16"

Weld Size, a_w $a_{w_cant} := \frac{3}{16}\cdot in$

$R_{nwm_allow_cant} := (2)\cdot\left(\frac{\sqrt{2}}{2}\right)\cdot a_{w_cant}\cdot Weld_Allow_V$ $R_{nwm_allow_cant} = 5.568\cdot\frac{kip}{in}$ x 2 since two sides

Base Metal: Controlling Metal Thickness, t $t_{contol} := \min(t_{PL}, t_b)$ $t_{contol} = 0.375\cdot in$

$R_{nbm_allow_cant} := t_{contol}\cdot(Base_Allow_V)$ $R_{nbm_allow_cant} = 6.975\cdot\frac{kip}{in}$

Design Strength: $R_{nw_cant} := \min(R_{nwm_allow_cant}, R_{nbm_allow_cant})$ $R_{nw_cant} = 5.568\cdot\frac{kip}{in}$

Forces, Properties, and Result

$V_{PL_cant} = 0.910\cdot kip$ $M_{PL_cant} = 10.920\cdot kip\cdot in$

$L_{w_cant} := H_b - 2\cdot(1.5t_b)$ $L_{w_cant} = 3.605\cdot in$ (welded on flat with 1.5 radius)

$S_{PL_cant_weld} := \frac{L_{w_cant}^2}{3}$ $S_{PL_cant_weld} = 4.332\cdot in^2$

Resultant: $R1_{w_cant} := \sqrt{\left(\frac{V_{PL_cant}}{L_{w_cant}}\right)^2 + \left(\frac{M_{PL_cant}}{S_{PL_cant_weld}}\right)^2}$ $R1_{w_cant} = 2.533\cdot\frac{kip}{in}$

$Check(R_{nw_cant}, R1_{w_cant}) = "OK"$

Support Reactions for Adhesive Anchors

Support Type 1) Supports for Members 201 (Jt 1) , 204 (Jt 4), 211 (Jt 11), and 214 (Jt 14) are the same. Joint 14 controls.
Support Type 2) Supports for Members 202 (Jt 2), 203 (Jt 3), 212 (Jt 12), and 213 (Jt 13) are the same. Joint 2 controls.

Design like supports for maximum forces. Ignore axial force since all are in compression.
Forces from Staad.Pro are global.

$VX1_s := 1.89 \cdot kip$ $VX2_s := 5.93 \cdot kip$

$VZ1_s := 1.95 \cdot kip$ $VZ2_s := 0.47 \cdot kip$

ADHESIVE ANCHORS

See directions from Staad.Pro to Hilti PROFIS.

	STAAD.Pro	Hilti PROFIS
Vertical:	Y	Z
Lateral:	X	X
Lateral:	Z	Y

Use Hilti RE-500 V3 adhesive.
See results from Hilti PROFIS Anchor Program.
Use Load Factor of 1.6 for anchor design in concrete per ACI (*Building Code Requirements For Structural Concrete (ACI 318-14) And Commentary (ACI 318R-14)*) using blockage as a live load in Section 5.3.

$VX1_{s_u} := (1.6) \cdot VX1_s$ $VX1_{s_u} = 3024\,lbf$ $VX2_{s_u} := (1.6) \cdot VX2_s$ $VX2_{s_u} = 9488\,lbf$

$VZ1_{s_u} := (1.6) \cdot VZ1_s$ $VZ1_{s_u} = 3120\,lbf$ $VZ2_{s_u} := (1.6) \cdot VZ2_s$ $VZ2_{s_u} = 752\,lbf$

Support Type 2 controls so use same size anchors for all connections.

3/4" Diameter, Galvanized Hilti HAS-E-55 Rod (ASTM F1554, Grade 55) w/ 5" embedment with Hilti RE-500 V3 adhesive

www.hilti.us

Profis Anchor 2.9.1

Company:	Ayres - TMR	Page:	1
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

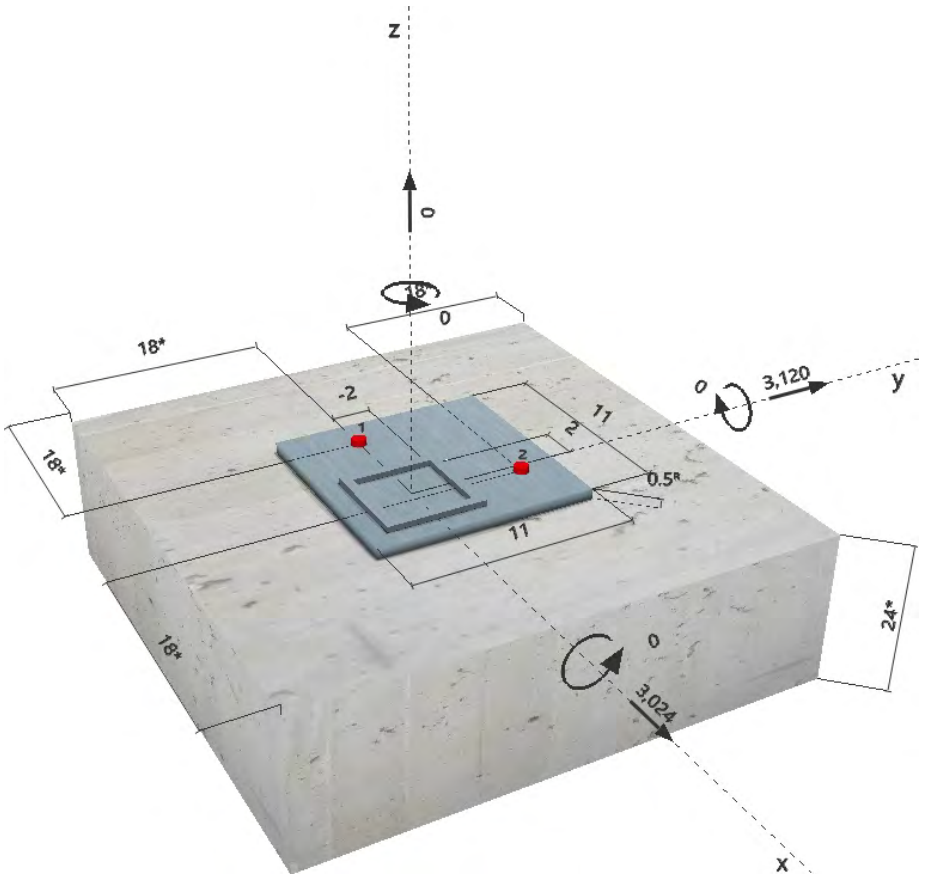
Specifier's comments: Joints 1, 4, 11, 14 (Dec 2020)

1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + HAS-E-55 (ASTM F1554 Gr.55) 3/4**
Effective embedment depth:	$h_{ef,opti}$ = 3.500 in. ($h_{ef,limit}$ = 11.250 in.)
Material:	ASTM A 1554 Grade 55
Evaluation Service Report:	ESR-3814
Issued I Valid:	1/1/2020 | 1/1/2021
Proof:	Design method ACI 318-14 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 11.000 in. x 11.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Square HSS (AISC), HSS5X5X.500; (L x W x T) = 5.000 in. x 5.000 in. x 0.500 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 24.000 in., Temp. short/long: 40/80 °F
Installation:	**hammer drilled hole, Installation condition: Submerged**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

R - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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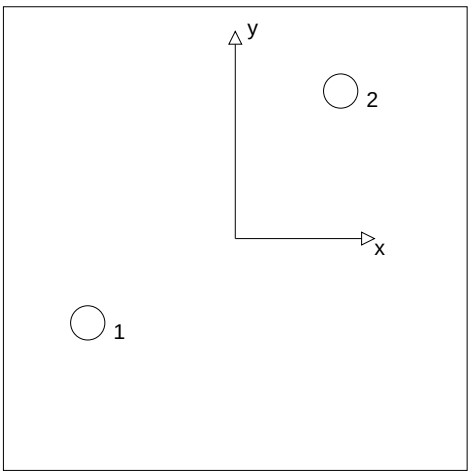
Company:	Ayres - TMR	Page:	2
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

2 Load case/Resulting anchor forces



Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	2,852	2,850	100
2	0	3,025	174	3,020

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	N/A	N/A	N/A	N/A
Bond Strength**	N/A	N/A	N/A	N/A
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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Company:	Ayres - TMR	Page:	3
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	3,025	9,786	31	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	4,345	7,872	56	OK
Concrete edge failure in direction x+**	4,345	18,693	24	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3814

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [lb]
15,055

Results

V_{sa} [lb]	ϕ steel	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
15,055	0.650	9,786	3,025

4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cpg} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ec,N}\,\psi_{ed,N}\,\psi_{c,N}\,\psi_{cp,N}\,N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\,h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\,e'_N}{3\,h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\,\lambda_a\,\sqrt{f'_c}\,h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.500	2.663	2.581	18.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	4.976	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
198.00	110.25	0.663	0.670	1.000	1.000	7,040

Results

V_{cpg} [lb]	ϕ concrete	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
11,246	0.700	7,872	4,345

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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Company:	Ayres - TMR	Page:	4
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-14 Eq. (17.5.2.1b)

$\phi V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e'_v}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\, c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
16.000	18.000	0.000	1.000	24.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
3.500	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
996.00	1,152.00	1.000	0.925	1.000	33,392

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
26,705	0.700	18,693	4,345

5 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

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Company:	Ayres - TMR	Page:	5
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

6 Installation data

Anchor plate, steel: -

Profile: Square HSS (AISC), HSS5X5X.500; (L x W x T) = 5.000 in. x 5.000 in. x 0.500 in.

Hole diameter in the fixture: d_f = 0.813 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-RE 500 V3 + HAS-E-55 (ASTM F1554 Gr.55) 3/4

Installation torque: 1,200.000 in.lb

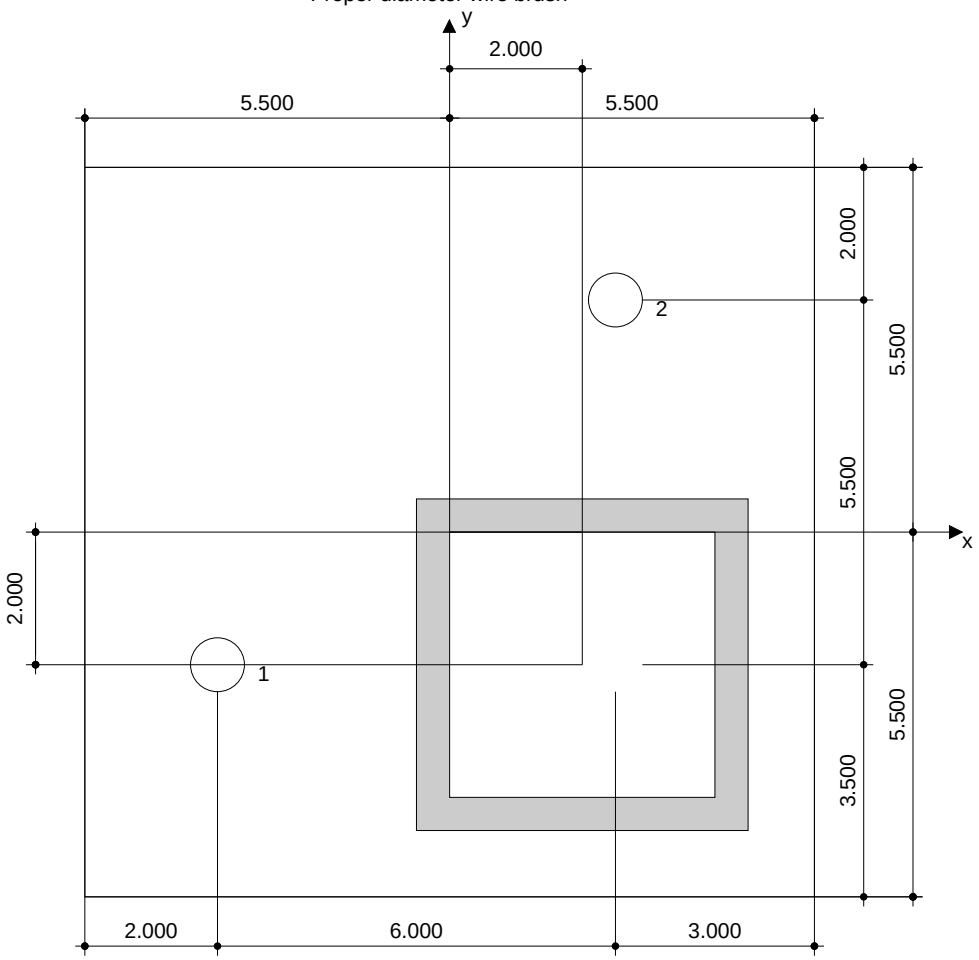
Hole diameter in the base material: 0.875 in.

Hole depth in the base material: 3.500 in.

Minimum thickness of the base material: 5.250 in.

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.500	-2.000	18.000	24.000	18.000	23.500
2	2.500	3.500	24.000	18.000	23.500	18.000

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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Company:	Ayres - TMR	Page:	6
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

7 Remarks; Your Cooperation Duties

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Profis Anchor 2.9.1

Company:	Ayres - TMR	Page:	1
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

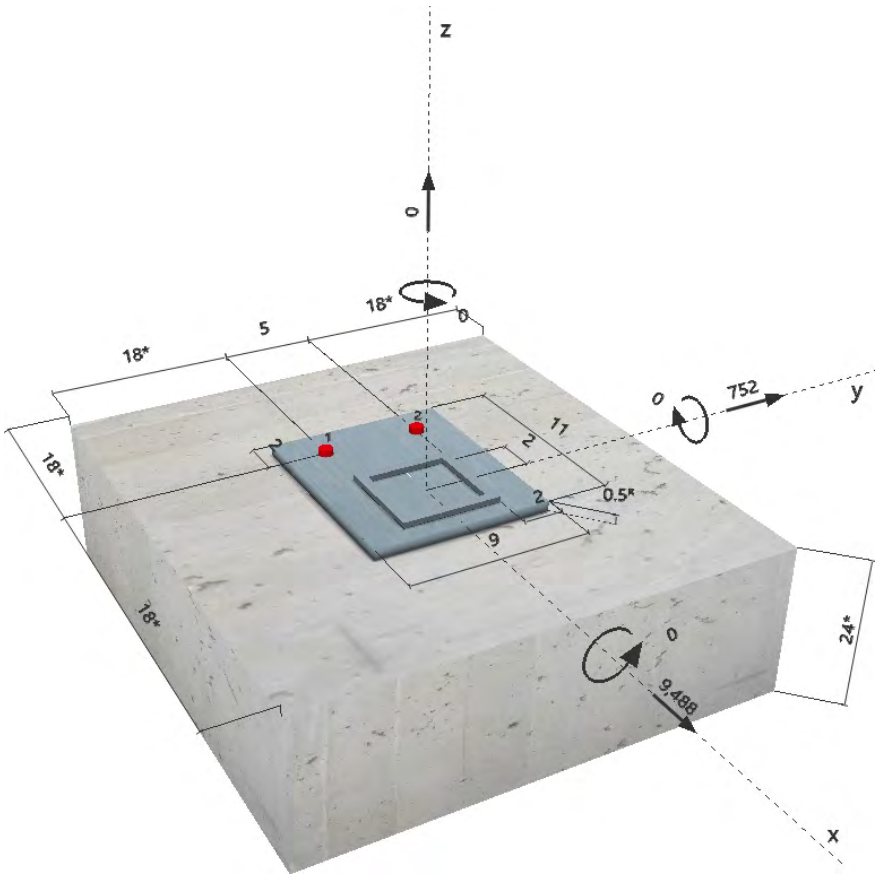
Specifier's comments: Joints 2, 3, 12, 13 (Dec 2020)

1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + HAS-E-55 (ASTM F1554 Gr.55) 3/4**
Effective embedment depth:	$h_{ef,opti}$ = 3.500 in. ($h_{ef,limit}$ = 11.250 in.)
Material:	ASTM A 1554 Grade 55
Evaluation Service Report:	ESR-3814
Issued I Valid:	1/1/2020 | 1/1/2021
Proof:	Design method ACI 318-14 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 11.000 in. x 9.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Square HSS (AISC), HSS5X5X.500; (L x W x T) = 5.000 in. x 5.000 in. x 0.500 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 24.000 in., Temp. short/long: 40/80 °F
Installation:	**hammer drilled hole, Installation condition: Submerged**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

R - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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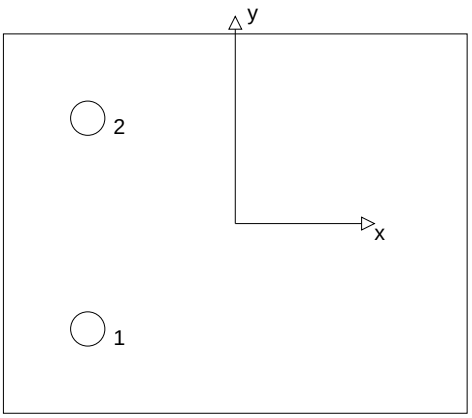
Company:	Ayres - TMR	Page:	2
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	5,584	5,571	376
2	0	3,935	3,917	376

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	N/A	N/A	N/A	N/A
Bond Strength**	N/A	N/A	N/A	N/A
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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Company:	Ayres - TMR	Page:	3
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	5,584	9,786	58	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	5,584	6,644	85	OK
Concrete edge failure in direction y+**	9,518	16,216	59	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3814

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [lb]
15,055

Results

V_{sa} [lb]	ϕ steel	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
15,055	0.650	9,786	5,584

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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Company:	Ayres - TMR	Page:	4
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

4.2 Pryout Strength (Bond Strength controls)

$V_{cp} = k_{cp}\left[\left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na} \psi_{cp,Na} N_{ba}\right]$ ACI 318-14 Eq. (17.5.3.1a)

$\phi V_{cp} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Na} see ACI 318-14, Section 17.4.5.1, Fig. R 17.4.5.1(b)

$A_{Na0} = (2 c_{Na})^2$ ACI 318-14 Eq. (17.4.5.1c)

$c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-14 Eq. (17.4.5.1d)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.5.3)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.5.4b)

$\psi_{cp,Na} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.5.5b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-14 Eq. (17.4.5.2)

Variables

k_{cp}	$\alpha_{overhead}$	$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{k,c}$ [psi]
2	1.000	1,608	0.750	3.500	18.000	901

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a
0.000	0.000	4.976	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
9.028	208.14	326.01	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
0.996	0.954	1.000	7,433

Results

V_{cp} [lb]	$\phi_{concrete}$	ϕV_{cp} [lb]	V_{ua} [lb]
9,492	0.700	6,644	5,584

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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Company:	Ayres - TMR	Page:	5
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-14 Eq. (17.5.2.1b)

$\phi V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e'_v}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\, c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
16.000	18.000	0.000	1.000	24.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
3.500	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
864.00	1,152.00	1.000	0.925	1.000	33,392

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
23,166	0.700	16,216	9,518

5 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

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Company:	Ayres - TMR	Page:	6
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

6 Installation data

Anchor plate, steel: -

Profile: Square HSS (AISC), HSS5X5X.500; (L x W x T) = 5.000 in. x 5.000 in. x 0.500 in.

Hole diameter in the fixture: d_f = 0.813 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-RE 500 V3 + HAS-E-55 (ASTM F1554 Gr.55) 3/4

Installation torque: 1,200.000 in.lb

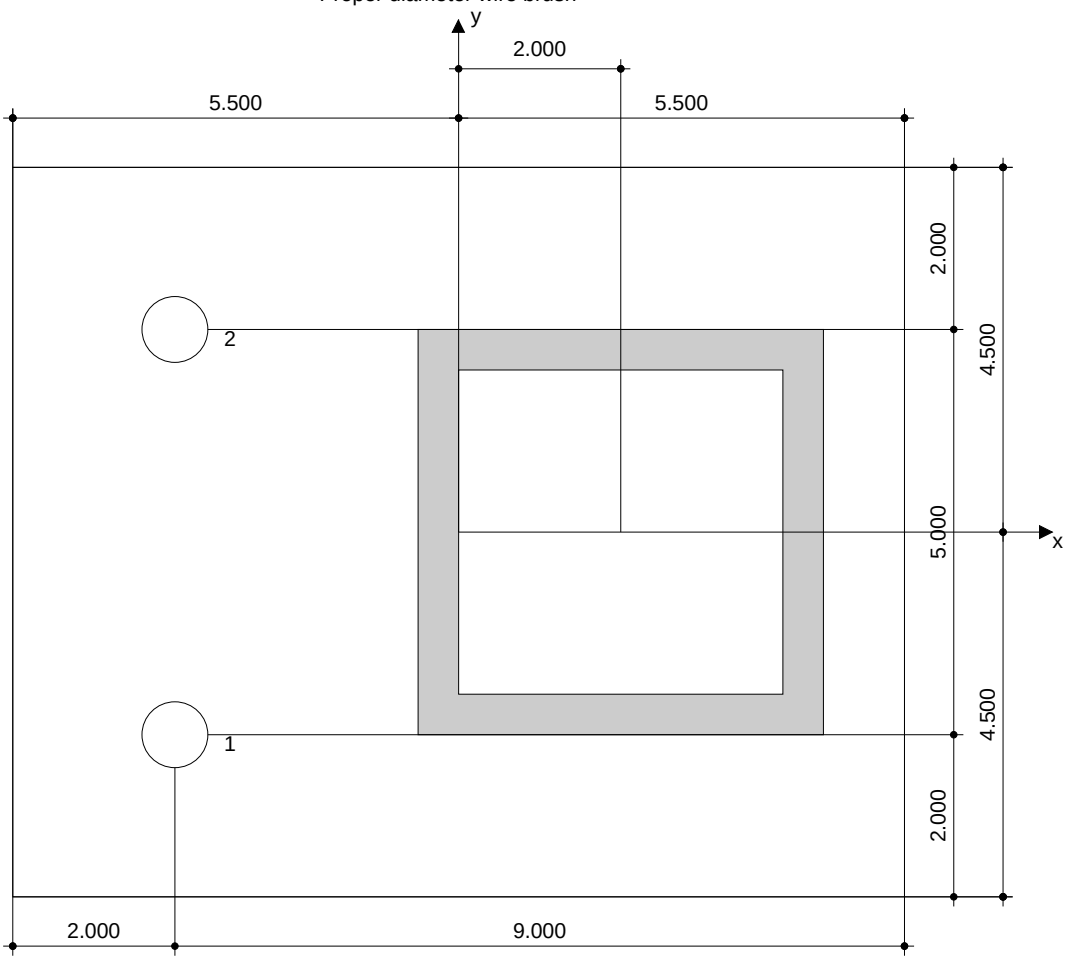
Hole diameter in the base material: 0.875 in.

Hole depth in the base material: 3.500 in.

Minimum thickness of the base material: 5.250 in.

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.500	-2.500	18.000	18.000	18.000	23.000
2	-3.500	2.500	18.000	18.000	23.000	18.000

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Company:	Ayres - TMR	Page:	7
Specifier:		Project:	Hogchute Trash Rack
Address:		Sub-Project I Pos. No.:	Frame Support
Phone I Fax:	|	Date:	12/17/2020
E-Mail:			

7 Remarks; Your Cooperation Duties

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Software licensed to Ayres Associates CONNECTED User: Todd Rudolph	Job No 26-1144.00	Sheet No 1	Rev
	Part		
Job Title HOGCHUTE INTAKE TRASHRACK	Ref TRASHRACK FRAME		
	By AYRES-TMR	Date DEC 2020	Chd AYRES-AEB
Client MESA CO, COLORADO	File Frame.std	Date/Time 18-Jan-2021 07:46	

Job Information

	Engineer	Checked	Approved
Name:	AYRES-TMR	AYRES-AEB	
Date:	DEC 2020	JAN 2021	

Project ID	
Project Name	

Comments

SIMPLE SUPPORTS

Structure Type	SPACE FRAME

Number of Nodes	16	Highest Node	114
Number of Elements	16	Highest Beam	214

Number of Basic Load Cases	3
Number of Combination Load Cases	4

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	SELFWEIGHT
Primary	2	LC1 FULL
Primary	3	LC2 FULL AND HALF
Combination	101	DL + LL
Combination	102	DL + LL
Combination	111	DL + LL
Combination	112	DL + LL

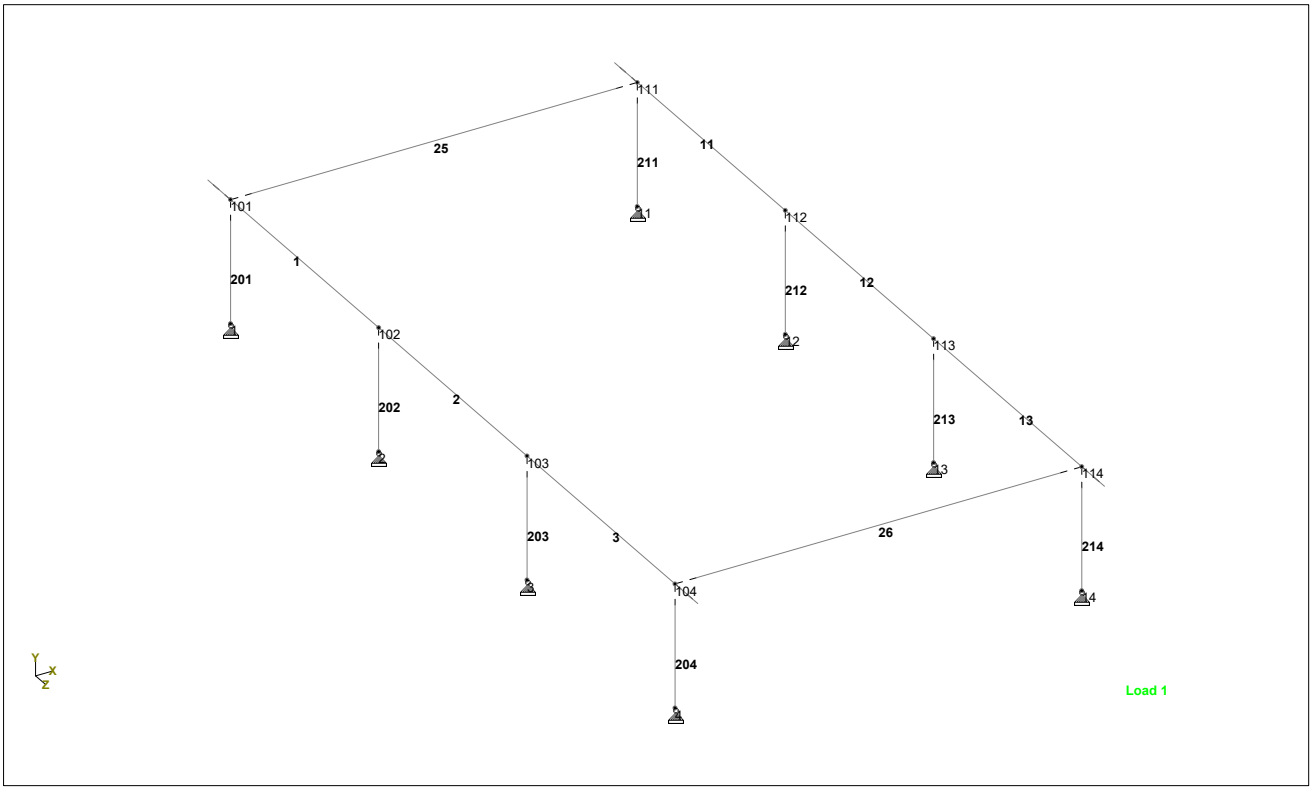
Primary Load Cases

Number	Name	Type
1	SELFWEIGHT	None
2	LC1 FULL	None
3	LC2 FULL AND HALF	None

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
101	DL + LL	1	SELFWEIGHT	1.05
		2	LC1 FULL	1.05
102	DL + LL	1	SELFWEIGHT	1.05
		3	LC2 FULL AND HALF	1.05
111	DL + LL	1	SELFWEIGHT	1.00
		2	LC1 FULL	1.00
112	DL + LL	1	SELFWEIGHT	1.00
		3	LC2 FULL AND HALF	1.00

Software licensed to Ayres Associates CONNECTED User: Todd Rudolph	Job No **26-1144.00**	Sheet No **3**	Rev
	Part		
Job Title HOGCHUTE INTAKE TRASHRACK	Ref TRASHRACK FRAME		
	By AYRES-TMR	Date DEC 2020	Chd AYRES-AEB
Client MESA CO, COLORADO	File Frame.std	Date/Time	18-Jan-2021 07:46



Whole Structure

```
            ************************************************
            *                                              *
            *           STAAD.Pro CONNECT Edition          *
            *           Version  22.05.00.131              *
            *           Proprietary Program of             *
            *           Bentley Systems, Inc.              *
            *           Date=    JAN 18, 2021              *
            *           Time=     7:46:50                  *
            *                                              *
            *  Licensed to: Ayres Associates               *
            ************************************************


     1. STAAD SPACE FRAME
INPUT FILE: D:\STAAD\Frame.STD
     2. START JOB INFORMATION
     3. JOB NAME HOGCHUTE INTAKE TRASHRACK
     4. JOB CLIENT MESA CO, COLORADO
     5. JOB NO 26-1144.00
     6. JOB REF TRASHRACK FRAME
     7. JOB COMMENT SIMPLE SUPPORTS
     8. ENGINEER NAME AYRES-TMR
     9. CHECKER NAME AYRES-AEB
    10. ENGINEER DATE DEC 2020
    11. CHECKER DATE JAN 2021
    12. END JOB INFORMATION
    13. INPUT WIDTH 79
    14. UNIT FEET KIP
    15. *
    16. JOINT COORDINATES
    17. * SUPPORT
    18. 1 0 0 0; 2 0 0 3.53; 3 0 0 7.07; 4 0 0 10.6
    19. 11 5.6 0 0; 12 5.6 0 3.53; 13 5.6 0 7.07; 14 5.6 0 10.6
    20. * FRAME
    21. 101 0 1.71 0; 102 0 1.71 3.53; 103 0 1.71 7.07; 104 0 1.71 10.6
    22. 111 5.6 1.71 0; 112 5.6 1.71 3.53; 113 5.6 1.71 7.07; 114 5.6 1.71 10.6
    23. *
    24. MEMBER INCIDENCES
    25. * FRAME
    26. 1 101 102; 2 102 103; 3 103 104
    27. 11 111 112; 12 112 113; 13 113 114
    28. 25 101 111; 26 104 114
    29. * SUPPORTS
    30. 201 1 101; 202 2 102; 203 3 103; 204 4 104
    31. 211 11 111; 212 12 112; 213 13 113; 214 14 114
    32. *
    33. UNIT INCHES KIP
    34. MEMBER PROPERTY AMERICAN
    35. 1 TO 3 11 TO 13 TABLE ST HSST7X7X0.5
    36. 25 26 201 TO 204 211 TO 214 TABLE ST HSST5X5X0.5
    37. DEFINE MATERIAL START
    38. ISOTROPIC STEEL
```

FRAME -- PAGE NO. 2

```
39. E 29000
40. POISSON 0.3
41. DENSITY 0.000283
42. ALPHA 6.5E-006
43. DAMP 0.03
44. END DEFINE MATERIAL
45. *
46. CONSTANTS
47. MATERIAL STEEL ALL
48. *
49. SUPPORTS
50. 1 2 3 4 11 12 13 14 PINNED
51. MEMBER OFFSET
52. 201 TO 204 211 TO 214 END 0 -3.5 0
53. 1 11 START 0 0 -6.5
54. 3 13 END 0 0 6.5
55. 25 26 START 3.5 0 0
56. 25 26 END -3.5 0 0
57. *
58. UNIT FEET KIP
59. *
60. LOAD 1 SELFWEIGHT
61. SELFWEIGHT Y -1
62. *
63. LOAD 2 LC1 FULL
64. MEMBER LOAD
65. 1 TO 3 11 TO 13 UNI GY -3.848
66. 1 TO 3 UNI GX 0.728
67. 11 TO 13 UNI GX -0.728
68. 25 26 UNI GY -0.520
69. 25 UNI GZ 2.496
70. 25 UMOM X -1.768
71. 26 UNI GZ -2.496
72. 26 UMOM X 1.768
73. *
74. 201 204 UNI GX 2.340
75. 202 203 UNI GX 4.550
76. 201 211 UNI GZ 0.728
77. 211 214 UNI GX -2.340
78. 212 213 UNI GX -4.550
79. 204 214 UNI GZ -0.728
80. *
81. *
82. LOAD 3 LC2 FULL AND HALF
83. MEMBER LOAD
84. * HALF OF LATERAL ON 11 TO 13, 25, 211 TO 214
85. 1 TO 3 11 TO 13 UNI GY -3.848
86. 1 TO 3 UNI GX 0.728
87. 11 TO 13 UNI GX -0.728
88. 25 26 UNI GY -0.520
89. 25 UNI GZ 1.248
90. 25 UMOM X -0.884
91. 26 UNI GZ -2.496
92. 26 UMOM X 1.768
93. *
94. 201 204 UNI GX 2.340
```

FRAME -- PAGE NO. 3

```
 95. 202 203 UNI GX 4.550
 96. 201 UNI GZ 0.728
 97. 211 UNI GZ 0.364
 98. 211 214 UNI GX -1.170
 99. 212 213 UNI GX -2.275
100. 204 UNI GZ -0.728
101. 214 UNI GZ -0.364
102. *
103. * LOAD COMBINATIONS FROM SECTION 2 OF:
104. * ASCE 7-10 MINIMUM DESIGN LOADS FOR
105. * BUILDINGS AND OTHER STRUCTURES
106. * DL + LL
107. * INCREASE LOADS BY 5 PERCENT WHEN
108. * CHECKING CODE AND MEMBER END FORCES.
109. * NO INCREASE FOR DEFLECTION AND REACTIONS FOR ANCHORS.
110. *
111. UNIT INCHES KIP
112. LOAD COMB 101 DL + LL
113. 1 1.05 2 1.05
114. LOAD COMB 102 DL + LL
115. 1 1.05 3 1.05
116. *
117. LOAD COMB 111 DL + LL
118. 1 1.0 2 1.0
119. LOAD COMB 112 DL + LL
120. 1 1.0 3 1.0
121. *
122. PERFORM ANALYSIS


         P R O B L E M   S T A T I S T I C S
         -----------------------------------

   NUMBER OF JOINTS         16  NUMBER OF MEMBERS      16
   NUMBER OF PLATES          0  NUMBER OF SOLIDS        0
   NUMBER OF SURFACES        0  NUMBER OF SUPPORTS      8

         Using 64-bit analysis engine.

         SOLVER USED IS THE IN-CORE ADVANCED MATH SOLVER

  TOTAL      PRIMARY LOAD CASES =     3, TOTAL DEGREES OF FREEDOM =      72
  TOTAL LOAD COMBINATION  CASES =     4  SO FAR.



  123. LOAD LIST 101 102
  124. PRINT SUPPORT REACTION ALL
SUPPORT  REACTION ALL
```

FRAME -- PAGE NO. 4

SUPPORT REACTIONS -UNIT KIP INCH STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	101	0.41	10.39	-1.91	0.00	0.00	0.00
	102	-0.35	9.87	-0.57	0.00	0.00	0.00
2	101	-5.80	14.97	-0.26	0.00	0.00	0.00
	102	-6.23	15.26	0.49	0.00	0.00	0.00
3	101	-5.80	14.97	0.26	0.00	0.00	0.00
	102	-5.91	14.94	1.03	0.00	0.00	0.00
4	101	0.41	10.39	1.91	0.00	0.00	0.00
	102	-0.43	9.36	2.34	0.00	0.00	0.00
11	101	-0.41	10.39	-1.91	0.00	0.00	0.00
	102	-2.07	11.15	-0.29	0.00	0.00	0.00
12	101	5.80	14.97	-0.26	0.00	0.00	0.00
	102	3.51	15.28	0.49	0.00	0.00	0.00
13	101	5.80	14.97	0.26	0.00	0.00	0.00
	102	3.20	14.95	1.03	0.00	0.00	0.00
14	101	-0.41	10.39	1.91	0.00	0.00	0.00
	102	-1.98	10.64	2.05	0.00	0.00	0.00

********** END OF LATEST ANALYSIS RESULT **********

```
125. PARAMETER
126. CODE AISC UNIFIED 2016
127. METHOD ASD
128. * SQUARE HSS IS ASTM A500 GRADE C
129. FYLD 50 MEMB 1 TO 3 11 TO 13 25 26 201 TO 204 211 TO 214
130. FU 62 MEMB 1 TO 3 11 TO 13 25 26 201 TO 204 211 TO 214
131. BEAM 1 ALL
132. TORSION 1 ALL
133. *
134. UNT 120 MEMB 1 2 3 11 12 13
135. UNB 120 MEMB 1 2 3 11 12 13
136. *
137. PRINT MEMBER FORCES LIST 1 TO 3 11 TO 13 25 26 201 TO 204 211 TO 214
MEMBER   FORCES   LIST     1
```

FRAME -- PAGE NO. 5

MEMBER END FORCES STRUCTURE TYPE = SPACE

ALL UNITS ARE -- KIP INCH (LOCAL)

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
1	101	101	5.75	8.91	5.44	37.66	-129.07	87.66
		102	-5.75	7.70	-2.33	-37.66	-60.70	-58.11
	102	101	3.98	9.04	4.85	44.14	-94.85	87.92
		102	-3.98	7.58	-1.73	-44.14	-65.88	-52.18
2	101	102	5.49	7.22	1.35	0.00	60.70	52.78
		103	-5.49	7.22	1.35	-0.00	-60.70	-52.78
	102	102	4.47	7.64	1.19	-2.32	65.88	62.21
		103	-4.47	6.81	1.52	2.32	-58.85	-44.39
3	101	103	5.75	7.70	-2.33	-37.66	60.70	58.11
		104	-5.75	8.91	5.44	37.66	129.07	-87.66
	102	103	5.50	8.09	-2.38	-42.31	58.85	65.59
		104	-5.50	8.53	5.49	42.31	133.46	-76.32
11	101	111	5.75	8.91	-5.44	-37.66	129.07	87.66
		112	-5.75	7.70	2.33	37.66	60.70	-58.11
	102	111	3.36	9.02	-4.18	-29.05	78.23	87.21
		112	-3.36	7.60	1.06	29.05	49.78	-52.43
12	101	112	5.49	7.22	-1.35	-0.00	-60.70	52.78
		113	-5.49	7.22	-1.35	0.00	60.70	-52.78
	102	112	3.85	7.64	-1.19	2.32	-49.78	62.44
		113	-3.85	6.81	-1.52	-2.32	42.75	-44.61
13	101	113	5.75	7.70	2.33	37.66	-60.70	58.11
		114	-5.75	8.91	-5.44	-37.66	-129.07	-87.66
	102	113	4.88	8.11	1.71	27.22	-42.75	65.84
		114	-4.88	8.51	-4.82	-27.22	-116.84	-75.61
25	101	101	9.33	1.44	-6.57	55.88	70.70	7.52
		111	-9.33	1.44	-6.57	55.88	-70.70	-7.52
	102	101	7.98	0.79	-3.47	27.92	51.21	-12.16
		111	-7.98	2.09	-3.10	27.96	-40.22	-26.96
26	101	104	9.33	1.44	6.57	-55.88	-70.70	7.52
		114	-9.33	1.44	6.57	-55.88	70.70	-7.52
	102	104	8.55	0.79	6.76	-55.85	-74.12	-12.00
		114	-8.55	2.09	6.39	-55.90	63.13	-27.11
201	101	1	10.39	-0.41	-1.91	0.00	0.00	-0.00
		101	-10.35	3.89	0.83	0.00	23.27	-36.60

FRAME -- PAGE NO. 6

MEMBER END FORCES STRUCTURE TYPE = SPACE

ALL UNITS ARE -- KIP INCH (LOCAL)

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
	102	1	9.87	0.35	-0.57	0.00	-0.00	-0.00
		101	-9.83	3.14	-0.51	0.00	0.55	-23.76
202	101	2	14.97	5.80	-0.26	0.00	0.00	0.00
		102	-14.93	0.97	0.26	0.00	4.42	41.08
	102	2	15.26	6.23	0.49	0.00	-0.00	-0.00
		102	-15.22	0.55	-0.49	0.00	-8.32	48.37
203	101	3	14.97	5.80	0.26	0.00	0.00	0.00
		103	-14.93	0.97	-0.26	0.00	-4.42	41.08
	102	3	14.94	5.91	1.03	0.00	-0.00	0.00
		103	-14.90	0.86	-1.03	0.00	-17.59	43.00
204	101	4	10.39	-0.41	1.91	0.00	0.00	-0.00
		104	-10.35	3.89	-0.83	0.00	-23.27	-36.60
	102	4	9.36	0.43	2.34	0.00	-0.00	-0.00
		104	-9.32	3.06	-1.25	0.00	-30.58	-22.38
211	101	11	10.39	0.41	-1.91	0.00	0.00	0.00
		111	-10.35	-3.89	0.83	0.00	23.27	36.60
	102	11	11.15	2.07	-0.29	0.00	0.00	0.00
		111	-11.11	-3.81	-0.25	0.00	0.28	49.99
212	101	12	14.97	-5.80	-0.26	0.00	0.00	-0.00
		112	-14.93	-0.97	0.26	0.00	4.42	-41.08
	102	12	15.28	-3.51	0.49	0.00	0.00	-0.00
		112	-15.24	0.12	-0.49	0.00	-8.30	-30.94
213	101	13	14.97	-5.80	0.26	0.00	0.00	-0.00
		113	-14.93	-0.97	-0.26	0.00	-4.42	-41.08
	102	13	14.95	-3.20	1.03	0.00	-0.00	0.00
		113	-14.91	-0.19	-1.03	0.00	-17.61	-25.57
214	101	14	10.39	0.41	1.91	0.00	0.00	0.00
		114	-10.35	-3.89	-0.83	0.00	-23.27	36.60
	102	14	10.64	1.98	2.05	0.00	-0.00	-0.00
		114	-10.60	-3.73	-1.51	0.00	-30.32	48.61

********** END OF LATEST ANALYSIS RESULT **********

138. CHECK CODE ALL

FRAME -- PAGE NO. 7

PARAMETER 1

FRAME -- PAGE NO. 8

STAAD.PRO CODE CHECKING - AISC 360-16 ASD (V1.2)
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted).
***NOTE : AISC 360-16 Design Statement for STAAD.Pro.

*** AXIS CONVENTION ***:
==========

The capacity results and intermediate results in the report follow the notations
and axes labels as defined in the AISC 360-16 code.
The analysis results are reported in STAAD.Pro axis convention and the AISC 360:16
design results are reported in AISC 360-16 code axis convention.

AISC Spec.	STAAD.Pro	Description
X	Z	Axis typically parallel to the sections principal major axis.
Y	Y	Axis typically parallel to the sections principal minor axis.
Z	X	Longitudinal axis perpendicular to the cross section.

SECTION FORCES AXIS MAPPING: -

AISC Spec.	STAAD.Pro	Description
Pz	FX	Axial force.
Vy	FY	Shear force along minor axis.
Vx	FZ	Shear force along major axis.
Tz	MX	Torsional moment.
My	MY	Bending moment about minor axis.
Mx	MZ	Bending moment about major axis.

*** DESIGN MESSAGES ***:
==========

1. Section classification reported is for the cross section and loadcase that
   produced the worst case design ratio for flexure/compression Capacity results.
2. Results for any Capacity/Check that is not relevant for a section/loadcase
   based on the code clause in AISC 360-16 will not be shown in the report.
3. Bending results are reported as being “about” the relevant axis (X/Y), while
   the results for shear are reported as being for shear forces “along” the axis.
   E.g : Mx indicates bending about the X axis, while Vx indicates shear along
   the X axis.

*** ABBREVIATIONS ***:
==========

F-T-B = Flexural-Torsional Buckling
L-T-B = Lateral-Torsional Buckling
F-L-B = Flange Local Buckling
W-L-B = Web Local Buckling
L-L-B = Leg Local Buckling
C-F-Y = Compression Flange Yielding
T-F-Y = Tension Flange Yielding

STAAD.PRO CODE CHECKING - AISC 360-16 ASD (V1.2)
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted).

- Member : 1

Member No:	1	Profile:	ST HSST7X7X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.331	Loadcase:	101		
Location:	0.00	Ref:	DG9:Eq: 4.17				
Pz:	5.748	C	Vy:	8.915	Vx:	5.440	
Tz:	37.66		My:	-129.1	Mx:	87.66	

- Member : 2

Member No:	2	Profile:	ST HSST7X7X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.198	Loadcase:	102		
Location:	0.00	Ref:	DG9:Eq: 4.17				
Pz:	4.468	C	Vy:	7.644	Vx:	1.188	
Tz:	-2.324		My:	65.88	Mx:	62.21	

- Member : 3

Member No:	3	Profile:	ST HSST7X7X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.331	Loadcase:	101		
Location:	48.86	Ref:	DG9:Eq: 4.17				
Pz:	5.748	C	Vy:	-8.915	Vx:	-5.440	
Tz:	-37.66		My:	-129.1	Mx:	87.66	

- Member : 11

Member No:	11	Profile:	ST HSST7X7X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.331	Loadcase:	101		
Location:	0.00	Ref:	DG9:Eq: 4.17				
Pz:	5.748	C	Vy:	8.915	Vx:	-5.440	
Tz:	-37.66		My:	129.1	Mx:	87.66	

- Member : 12

Member No:	12	Profile:	ST HSST7X7X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.180	Loadcase:	101		
Location:	42.48	Ref:	DG9:Eq: 4.17				
Pz:	5.489	C	Vy:	-7.225	Vx:	1.353	
Tz:	-.8860E-06		My:	-60.70	Mx:	52.78	

- Member : 13

Member No:	13	Profile:	ST HSST7X7X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.331	Loadcase:	101		
Location:	48.86	Ref:	DG9:Eq: 4.17				
Pz:	5.748	C	Vy:	-8.915	Vx:	5.440	
Tz:	37.66		My:	129.1	Mx:	87.66	

- Member : 25

Member No:	25	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.290	Loadcase:	101		
Location:	0.00	Ref:	DG9:Eq: 4.17				
Pz:	9.333	C	Vy:	1.440	Vx:	-6.574	
Tz:	55.88		My:	70.70	Mx:	7.523	

STAAD.PRO CODE CHECKING - AISC 360-16 ASD (V1.2)
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted).

- Member : 26

Member No:	26	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.325	Loadcase:	102		
Location:	60.20	Ref:	DG9:Eq: 4.17				
Pz:	8.550 C	Vy:	-2.090	Vx:	-6.391		
Tz:	55.90	My:	-63.13	Mx:	27.11		

- Member : 201

Member No:	201	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.236	Loadcase:	101		
Location:	17.02	Ref:	DG9:Eq: 4.17				
Pz:	10.35 C	Vy:	-3.893	Vx:	-.8254		
Tz:	0.000	My:	-23.27	Mx:	36.60		

- Member : 202

Member No:	202	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.246	Loadcase:	102		
Location:	17.02	Ref:	DG9:Eq: 4.17				
Pz:	15.22 C	Vy:	-.5459	Vx:	0.4887		
Tz:	0.000	My:	8.317	Mx:	-48.37		

- Member : 203

Member No:	203	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.257	Loadcase:	102		
Location:	17.02	Ref:	DG9:Eq: 4.17				
Pz:	14.90 C	Vy:	-.8615	Vx:	1.033		
Tz:	0.000	My:	17.59	Mx:	-43.00		

- Member : 204

Member No:	204	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.236	Loadcase:	101		
Location:	17.02	Ref:	DG9:Eq: 4.17				
Pz:	10.35 C	Vy:	-3.893	Vx:	0.8254		
Tz:	0.000	My:	23.27	Mx:	36.60		

- Member : 211

Member No:	211	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.236	Loadcase:	101		
Location:	17.02	Ref:	DG9:Eq: 4.17				
Pz:	10.35 C	Vy:	3.893	Vx:	-.8254		
Tz:	0.000	My:	-23.27	Mx:	-36.60		

- Member : 212

Member No:	212	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)		
Status:	PASS	Ratio:	0.211	Loadcase:	101		
Location:	15.60	Ref:	DG9:Eq: 4.17				
Pz:	14.93 C	Vy:	0.4100	Vx:	-.2595		
Tz:	0.000	My:	-4.049	Mx:	42.06		

FRAME -- PAGE NO. 11

STAAD.PRO CODE CHECKING - AISC 360-16 ASD (V1.2)
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted).

- Member : 213

Member No:	213	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)
Status:	PASS	Ratio:	0.211	Loadcase:	101
Location:	15.60	Ref:	DG9:Eq: 4.17		
Pz:	14.93 C	Vy:	0.4100	Vx:	0.2595
Tz:	0.000	My:	4.049	Mx:	42.06

- Member : 214

Member No:	214	Profile:	ST HSST5X5X0.5		(AISC SECTIONS)
Status:	PASS	Ratio:	0.298	Loadcase:	102
Location:	17.02	Ref:	DG9:Eq: 4.17		
Pz:	10.60 C	Vy:	3.727	Vx:	1.510
Tz:	0.000	My:	30.32	Mx:	-48.61

```
  139. *
  140. *
  141. LOAD LIST 111 112
  142. PRINT SECTION DISPLACEMENTS NSECTION 12 LIST 1 TO 3 11 TO 13 25 26
 SECTION  DISPLACE NSECTION 12
```

FRAME -- PAGE NO. 12

MEMBER SECTION DISPLACEMENTS

UNITS ARE - INCH

MEMB	LOAD	GLOBAL X,Y,Z DISPL FROM START TO END JOINTS AT 1/12TH PTS					
1	111	-0.0000	-0.0015	0.0011	0.0013	-0.0013	0.0011
		0.0034	-0.0015	0.0010	0.0061	-0.0018	0.0009
		0.0092	-0.0022	0.0009	0.0127	-0.0025	0.0008
		0.0163	-0.0026	0.0007	0.0201	-0.0026	0.0007
		0.0239	-0.0024	0.0006	0.0276	-0.0020	0.0005
		0.0311	-0.0016	0.0005	0.0343	-0.0012	0.0004
		0.0372	-0.0011	0.0003			
	112	0.0043	-0.0013	-0.0017	0.0070	-0.0012	-0.0017
		0.0102	-0.0015	-0.0017	0.0139	-0.0019	-0.0018
		0.0177	-0.0024	-0.0018	0.0218	-0.0027	-0.0019
		0.0259	-0.0029	-0.0019	0.0300	-0.0029	-0.0020
		0.0340	-0.0027	-0.0020	0.0378	-0.0023	-0.0021
		0.0413	-0.0018	-0.0021	0.0445	-0.0014	-0.0022
		0.0473	-0.0011	-0.0022			
MAX LOCAL DISP = 0.00328 AT 16.29 LOAD 111 L/DISP= 14890							
2	111	0.0372	-0.0011	0.0003	0.0392	-0.0011	0.0003
		0.0408	-0.0013	0.0002	0.0421	-0.0016	0.0002
		0.0431	-0.0018	0.0001	0.0437	-0.0020	0.0001
		0.0439	-0.0021	0.0000	0.0437	-0.0020	-0.0001
		0.0431	-0.0018	-0.0001	0.0421	-0.0016	-0.0002
		0.0408	-0.0013	-0.0002	0.0392	-0.0011	-0.0003
		0.0372	-0.0011	-0.0003			
	112	0.0473	-0.0011	-0.0022	0.0492	-0.0010	-0.0022
		0.0507	-0.0012	-0.0023	0.0519	-0.0015	-0.0023
		0.0527	-0.0017	-0.0024	0.0531	-0.0019	-0.0024
		0.0531	-0.0020	-0.0025	0.0527	-0.0020	-0.0025
		0.0519	-0.0019	-0.0026	0.0508	-0.0017	-0.0026
		0.0493	-0.0014	-0.0026	0.0474	-0.0012	-0.0027
		0.0452	-0.0011	-0.0027			
MAX LOCAL DISP = 0.00690 AT 21.24 LOAD 112 L/DISP= 6156							
3	111	0.0372	-0.0011	-0.0003	0.0343	-0.0012	-0.0004
		0.0311	-0.0016	-0.0005	0.0276	-0.0020	-0.0005
		0.0239	-0.0024	-0.0006	0.0201	-0.0026	-0.0007
		0.0163	-0.0026	-0.0007	0.0127	-0.0025	-0.0008
		0.0092	-0.0022	-0.0009	0.0061	-0.0018	-0.0009
		0.0034	-0.0015	-0.0010	0.0013	-0.0013	-0.0011
		-0.0000	-0.0015	-0.0011			

FRAME -- PAGE NO. 13

	112	0.0452	-0.0011	-0.0027	0.0421	-0.0012	-0.0028
		0.0387	-0.0015	-0.0029	0.0350	-0.0019	-0.0029
		0.0312	-0.0023	-0.0030	0.0273	-0.0026	-0.0031
		0.0234	-0.0026	-0.0031	0.0196	-0.0025	-0.0032
		0.0161	-0.0022	-0.0032	0.0129	-0.0018	-0.0033
		0.0102	-0.0013	-0.0034	0.0082	-0.0010	-0.0034
		0.0068	-0.0010	-0.0035			

MAX LOCAL DISP = 0.00371 AT 32.57 LOAD 112 L/DISP= 13165

11	111	0.0000	-0.0015	0.0011	-0.0013	-0.0013	0.0011
		-0.0034	-0.0015	0.0010	-0.0061	-0.0018	0.0009
		-0.0092	-0.0022	0.0009	-0.0127	-0.0025	0.0008
		-0.0163	-0.0026	0.0007	-0.0201	-0.0026	0.0007
		-0.0239	-0.0024	0.0006	-0.0276	-0.0020	0.0005
		-0.0311	-0.0016	0.0005	-0.0343	-0.0012	0.0004
		-0.0372	-0.0011	0.0003			
	112	0.0086	-0.0014	-0.0018	0.0065	-0.0013	-0.0018
		0.0040	-0.0016	-0.0019	0.0012	-0.0020	-0.0019
		-0.0018	-0.0024	-0.0019	-0.0050	-0.0028	-0.0020
		-0.0082	-0.0030	-0.0020	-0.0114	-0.0030	-0.0020
		-0.0144	-0.0027	-0.0021	-0.0174	-0.0023	-0.0021
		-0.0201	-0.0019	-0.0022	-0.0225	-0.0014	-0.0022
		-0.0246	-0.0011	-0.0022			

MAX LOCAL DISP = 0.00328 AT 16.29 LOAD 111 L/DISP= 14890

12	111	-0.0372	-0.0011	0.0003	-0.0392	-0.0011	0.0003
		-0.0408	-0.0013	0.0002	-0.0421	-0.0016	0.0002
		-0.0431	-0.0018	0.0001	-0.0437	-0.0020	0.0001
		-0.0439	-0.0021	0.0000	-0.0437	-0.0020	-0.0001
		-0.0431	-0.0018	-0.0001	-0.0421	-0.0016	-0.0002
		-0.0408	-0.0013	-0.0002	-0.0392	-0.0011	-0.0003
		-0.0372	-0.0011	-0.0003			
	112	-0.0246	-0.0011	-0.0022	-0.0261	-0.0010	-0.0023
		-0.0272	-0.0012	-0.0023	-0.0281	-0.0014	-0.0024
		-0.0287	-0.0017	-0.0024	-0.0290	-0.0019	-0.0024
		-0.0289	-0.0020	-0.0025	-0.0286	-0.0020	-0.0025
		-0.0279	-0.0019	-0.0025	-0.0270	-0.0016	-0.0026
		-0.0258	-0.0014	-0.0026	-0.0243	-0.0012	-0.0027
		-0.0226	-0.0011	-0.0027			

MAX LOCAL DISP = 0.00675 AT 21.24 LOAD 111 L/DISP= 6288

13	111	-0.0372	-0.0011	-0.0003	-0.0343	-0.0012	-0.0004
		-0.0311	-0.0016	-0.0005	-0.0276	-0.0020	-0.0005
		-0.0239	-0.0024	-0.0006	-0.0201	-0.0026	-0.0007
		-0.0163	-0.0026	-0.0007	-0.0127	-0.0025	-0.0008
		-0.0092	-0.0022	-0.0009	-0.0061	-0.0018	-0.0009
		-0.0034	-0.0015	-0.0010	-0.0013	-0.0013	-0.0011
		0.0000	-0.0015	-0.0011			

FRAME -- PAGE NO. 14

```
        112      -0.0226   -0.0011   -0.0027   -0.0201   -0.0012   -0.0028
                 -0.0175   -0.0015   -0.0028   -0.0146   -0.0020   -0.0029
                 -0.0117   -0.0024   -0.0029   -0.0086   -0.0026   -0.0030
                 -0.0057   -0.0027   -0.0030   -0.0028   -0.0026   -0.0031
                 -0.0002   -0.0023   -0.0032    0.0021   -0.0018   -0.0032
                  0.0040   -0.0014   -0.0033    0.0053   -0.0011   -0.0033
                  0.0060   -0.0011   -0.0034

    MAX LOCAL  DISP =    0.00351   AT      32.57  LOAD  112   L/DISP=  13925

     25   111       0.0012   -0.0017    0.0005    0.0010   -0.0030    0.0005
                    0.0008   -0.0045    0.0018    0.0006   -0.0057    0.0036
                    0.0004   -0.0068    0.0053    0.0002   -0.0074    0.0065
                    0.0000   -0.0076    0.0070   -0.0002   -0.0074    0.0065
                   -0.0004   -0.0068    0.0053   -0.0006   -0.0057    0.0036
                   -0.0008   -0.0045    0.0018   -0.0010   -0.0030    0.0005
                   -0.0012   -0.0017    0.0005

        112       0.0079   -0.0025   -0.0036    0.0077   -0.0047   -0.0056
                  0.0075   -0.0064   -0.0066    0.0074   -0.0076   -0.0068
                  0.0072   -0.0082   -0.0066    0.0070   -0.0082   -0.0064
                  0.0069   -0.0077   -0.0061    0.0067   -0.0068   -0.0060
                  0.0065   -0.0055   -0.0059    0.0064   -0.0040   -0.0058
                  0.0062   -0.0026   -0.0056    0.0060   -0.0015   -0.0048
                  0.0059   -0.0009   -0.0032

    MAX LOCAL  DISP =    0.00880   AT      30.10  LOAD  111   L/DISP=   6842

     26   111       0.0012   -0.0017   -0.0005    0.0010   -0.0030   -0.0005
                    0.0008   -0.0045   -0.0018    0.0006   -0.0057   -0.0036
                    0.0004   -0.0068   -0.0053    0.0002   -0.0074   -0.0065
                    0.0000   -0.0076   -0.0070   -0.0002   -0.0074   -0.0065
                   -0.0004   -0.0068   -0.0053   -0.0006   -0.0057   -0.0036
                   -0.0008   -0.0045   -0.0018   -0.0010   -0.0030   -0.0005
                   -0.0012   -0.0017   -0.0005

        112       0.0079   -0.0024   -0.0029    0.0078   -0.0046   -0.0030
                  0.0076   -0.0063   -0.0045    0.0074   -0.0075   -0.0066
                  0.0072   -0.0081   -0.0086    0.0070   -0.0081   -0.0101
                  0.0069   -0.0076   -0.0107    0.0067   -0.0066   -0.0105
                  0.0065   -0.0053   -0.0093    0.0063   -0.0039   -0.0075
                  0.0062   -0.0025   -0.0055    0.0060   -0.0014   -0.0038
                  0.0058   -0.0009   -0.0033

    MAX LOCAL  DISP =    0.00969   AT      30.10  LOAD  112   L/DISP=   6211

        ************ END OF SECT DISPL RESULTS ***********

  143. PRINT SUPPORT REACTION ALL
SUPPORT  REACTION ALL
```

FRAME -- PAGE NO. 15

SUPPORT REACTIONS -UNIT KIP INCH STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	111	0.39	9.90	-1.82	0.00	0.00	0.00
	112	-0.33	9.40	-0.55	0.00	0.00	0.00
2	111	-5.53	14.26	-0.25	0.00	0.00	0.00
	112	-5.93	14.54	0.47	0.00	0.00	0.00
3	111	-5.53	14.26	0.25	0.00	0.00	0.00
	112	-5.63	14.22	0.98	0.00	0.00	0.00
4	111	0.39	9.90	1.82	0.00	0.00	0.00
	112	-0.41	8.91	2.23	0.00	0.00	0.00
11	111	-0.39	9.90	-1.82	0.00	0.00	0.00
	112	-1.97	10.62	-0.27	0.00	0.00	0.00
12	111	5.53	14.26	-0.25	0.00	0.00	0.00
	112	3.34	14.55	0.46	0.00	0.00	0.00
13	111	5.53	14.26	0.25	0.00	0.00	0.00
	112	3.04	14.24	0.99	0.00	0.00	0.00
14	111	-0.39	9.90	1.82	0.00	0.00	0.00
	112	-1.89	10.13	1.95	0.00	0.00	0.00

********** END OF LATEST ANALYSIS RESULT **********

144. STEEL TAKE OFF ALL

FRAME -- PAGE NO. 16

STEEL TAKE OFF

STEEL TAKE-OFF

PROFILE	LENGTH(INCH)	WEIGHT(KIP)
ST HSST7X7X0.5	280.40	0.920
ST HSST5X5X0.5	256.56	0.572
TOTAL =		1.493

********** END OF DATA FROM INTERNAL STORAGE **********

145. FINISH

********** END OF THE STAAD.Pro RUN **********

**** DATE= JAN 18,2021 TIME= 7:46:51 ****

FRAME -- PAGE NO. 17

```
     ************************************************************
     *   For technical assistance on STAAD.Pro, please visit    *
     *   http://www.bentley.com/en/support/                     *
     *                                                          *
     *   Details about additional assistance from               *
     *   Bentley and Partners can be found at program menu      *
     *   Help->Technical Support                                *
     *                                                          *
     *           Copyright (c) Bentley Systems, Inc.            *
     *                http://www.bentley.com                    *
     ************************************************************
```

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Stability.xmcd
Page 1 of 26

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

CONCRETE USBR TYPE VI BASIN
HOG CHUTE DAM - MESA COUNTY, COLORADO

Near Embankment Toe With Exit Pipe - Stability

Ref.:
1) *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2) EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, USACOE, 30 Nov. 2016.
3) EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACOE, 1 December 2005.
4) EM 1110-2-2200, *Gravity Dam Design*, USACOE, 30 June 1995.
5) EM 1110-2-2502, *Retaining and Flood Walls*, USACOE, 29 September 1989.
6) EM 1110-2-6053, *Earthquake Design and Evaluation of Concrete Hydraulic Structures*, USACOE, 1 May 2007.
7) Letter: Geotechnical Design Parameters, Carson Lake, Mesa County, Colorado, from Michael A. Berry, PE, Huddleston-Berry, 27 August 2020 & 15 September 2020 email.
8) Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.
9) HY-8 Energy Dissipation Report output, Ayres, December 2020.

Design

1) Load Cases Per Ref. 3.

LC 1 - Unusual: Construction
LC 2 - Extreme: Construction with OBE
LC 3 - Usual: Normal Pool
LC 4 - Unusual: Normal Pool with Surcharge
LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
LC 6 - Extreme: Coincident Pool = Normal Pool with MDE
LC 7 - Extreme: Maximum Design Flood = IDF

2) Required Sliding Factors of Safety (Ref. 3, Table 3-2) and Resultant Location (Ref. 3, Table 3-5)
Usual = 1.7 w/ 100% Base in Compression
Unusual = 1.3 w/ 75% Base in Compression
Extreme = 1.1 w/ Resultant In Base

3) Assume main structure will not deflect sufficiently to cause active earth pressure conditions to develop so use use at-rest earth pressures. Use active earth pressures for flexible wingwalls.
Ref. 7: Site PGA = 0.236. k_h is defined as 1/3 of the site modified PGA so k_h = 0.08 for this site (OBE).
Ref. 6: Section 7-3, MDE = 2/3 MCE = (2/3)(0.277) = 0.185
Zero vertical earthquake acceleration.

4) Per Ref. 9: Original pipe is 30-inch inside diameter. Lining will reduce to 29-inch inside diameter.
IDF outlet: Q = 146 cfs and V = 34.2 ft/s.
Per Ref. 8: Exit IDF water El. = 9842.29 feet.

5) Per past inspections and site topography:
Since structure will be in similar location, use 1.0 foot of water above top of foundation slab, El. 9838.91 feet.

6) Average fill behind wingwalls is El. 9842.5 feet and behind headwall is El. 9846.0 feet. Wingwall at slab bottom.
Fill slope behind wingwalls is about 6H:1V and slope behind headwall is about 10H:1V. Wingwalls 4.0 feet long.
Toe riprap at top of endsill El. 9839.9 feet for main structure width and half of wingwall length.

7) Elevations in NAVD88.

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Stability.xmcd
Page 2 of 26

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Variables

Variables

Usual case when k_v, δ, and θ are zero.

Concrete Unit Weight
Ref. 4, Section 3-3b
$\gamma_c := 150 \cdot pcf$

Water Unit Weight
Ref. 4, Section 3-3c
$\gamma_w := 62.5 \cdot pcf$

Soil Moist Unit Weight
Ref. 7
$\gamma_m := 125 \cdot pcf$

Soil Saturated Unit Weight
Ref. 7
$\gamma_s := 130 \cdot pcf$

Backfill Friction Angle
Ref. 7
$\phi := 32 \cdot deg$

Foundation Friction Angle
Ref. 7
$\phi_{foundation} := 21 \cdot deg$

Inclination of Soil Surface
(0 if level backfill)
[A at headwall and B at wingwalls]
$\beta_{hA} := 5.7 \cdot deg$ (heel)
$\beta_{hB} := 9.5 \cdot deg$ (heel)
$\beta_t := 0 \cdot deg$ (toe)

Horizontal Acceleration (in g's)
Design Note 3
$k_{h_OBE} := 0.08$
$k_{h_MDE} := 0.185$

Thicknesses
$t_{slab} := 1 \cdot ft + 6 \cdot in$
$t_{baffle} := 1 \cdot ft + 0 \cdot in$
$t_{walls} := 1 \cdot ft + 6 \cdot in$
$t_{endsill} := 1 \cdot ft + 0 \cdot in$ [and wing walls]

Elevations
[min NP = t_{slab}]
$EL_{w_NP} := 9838.91 \cdot ft$
$EL_{wall} := 9847.08 \cdot ft$
$EL_{soilA} := 9846.0 \cdot ft$
$EL_{w_IDF} := 9842.29 \cdot ft$
$EL_{Tslab} := 9837.91 \cdot ft$
$EL_{soilB} := 9842.5 \cdot ft$
$EL_{pipe_invert} := 9839.81 \cdot ft$
$EL_{Bslab} := EL_{Tslab} - t_{slab}$
$EL_{Bslab} = 9836.41\ ft$

Heel Soil Heights
$h_{s1A} := EL_{soilA} - EL_{Bslab}$
$h_{s1A} = 9.59\ ft$
$h_{s1B} := EL_{soilB} - EL_{Bslab}$
$h_{s1B} = 6.09\ ft$

Water Heights
$h_{w1_NP} := EL_{w_NP} - EL_{Bslab}$
$h_{w1_NP} = 2.50\ ft$
$h_{w1_IDF} := EL_{w_IDF} - EL_{Bslab}$
$h_{w1_IDF} = 5.88\ ft$

Toe Soil Height
The toe consists of 2.0 feet of riprap and bedding over the native material placed to top of endsill. Minimum thickness of material is slab thickness. To account for riprap, side slopes, and to ease computations, add extra 1.75 feet using native material.

$h_{s2} := t_{slab} + 1.75 \cdot ft$
$h_{s2} = 3.25\ ft$

Uniform Surcharge
$q_{surcharge} := 200 \cdot psf$

Allowable Bearing Capacity
Ref. 7
$BearingCap_{allow} := 2000 \cdot psf$
$BearingCap_{allow_eq} := 2666 \cdot psf$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

VERTICAL FORCES

-- see Figure 1 (centroid arm taken from downstream structure edge)

Foundation Slab

Base = t_{walls} + (L1 + L2) + $t_{endsill}$ (parallel to flow) *increase L1 by Δ baffle thickness increase of 2"
Width = $2t_{walls}$ + (WB)

$Base_{slab} := t_{walls} + [(7\cdot ft + 0\cdot in) + (9\cdot ft + 2\cdot in)] + t_{endsill}$ $Base_{slab} = 18.67\ ft$

$Width_{slab} := 2\cdot t_{walls} + (12\cdot ft + 0\cdot in)$ $Width_{slab} = 15.00\ ft$

$WT_{slab} := \gamma_c\cdot Base_{slab}\cdot Width_{slab}\cdot t_{slab}$ $WT_{slab} = 63.00\cdot kip$

$arm_{slab} := \frac{Base_{slab}}{2}$ $arm_{slab} = 9.33\ ft$

$armY_{slab} := \frac{t_{slab}}{2}$ $armY_{slab} = 0.75\ ft$

Abutments

$L_{abut} := Base_{slab}$ $L_{abut} = 18.67\ ft$

$WT1_{abut} := 2\cdot\gamma_c\cdot(EL_{wall} - EL_{Tslab})\cdot L_{abut}\cdot t_{walls}$ $WT1_{abut} = 77.03\cdot kip$

$arm1_{abut} := \frac{L_{abut}}{2}$ $arm1_{abut} = 9.33\ ft$

$armY1_{abut} := \frac{EL_{wall} - EL_{Tslab}}{2}$ $armY1_{abut} = 4.59\ ft$

L2

$WT2_{abut} := 2\cdot\frac{1}{2}\cdot\gamma_c\cdot(9\cdot ft + 2\cdot in)\cdot(EL_{wall} - 9843.91\cdot ft)\cdot t_{walls}$ $WT2_{abut} = 6.54\cdot kip$

$arm2_{abut} := \frac{9\cdot ft + 2\cdot in}{3}$ $arm2_{abut} = 3.06\ ft$

$armY2_{abut} := (EL_{wall} - EL_{Tslab}) - \frac{EL_{wall} - 9843.91\cdot ft}{3}$ $armY2_{abut} = 8.11\ ft$

Stability

$WT_{abut} := WT1_{abut} - WT2_{abut}$ $WT_{abut} = 70.49 \cdot kip$

$arm_{abut} := \frac{WT1_{abut} \cdot arm1_{abut} - WT2_{abut} \cdot arm2_{abut}}{WT_{abut}}$ $arm_{abut} = 9.92\ ft$

$armY_{abut} := \frac{WT1_{abut} \cdot armY1_{abut} - WT2_{abut} \cdot armY2_{abut}}{WT_{abut}}$ $armY_{abut} = 4.26\ ft$

Headwall (ignore opening)

$L_{headwall} := Width_{slab} - 2 \cdot t_{walls}$ $L_{headwall} = 12.00\ ft$

$WT_{headwall} := \gamma_c \cdot (EL_{wall} - EL_{Tslab}) \cdot L_{headwall} \cdot t_{walls}$ $WT_{headwall} = 24.76 \cdot kip$

$arm_{headwall} := Base_{slab} - \frac{t_{walls}}{2}$ $arm_{headwall} = 17.92\ ft$

$armY_{headwall} := \frac{EL_{wall} - EL_{Tslab}}{2}$ $armY_{headwall} = 4.59\ ft$

Baffle

$L_{baffle} := L_{headwall}$ $L_{baffle} = 12.00\ ft$

$H_{baffle} := (4 \cdot ft + 6 \cdot in) + (8 \cdot in)$ $H_{baffle} = 5.17\ ft$

$A_{baffle} := L_{headwall} \cdot (H_{baffle}) - 2 \cdot [(2 \cdot 12 \cdot in) \cdot (18 \cdot in)]$ $A_{baffle} = 56.00 \cdot ft^2$

$WT1_{baffle} := \gamma_c \cdot A_{baffle} \cdot t_{baffle}$ $WT1_{baffle} = 8.40 \cdot kip$

$arm1_{baffle} := \left[t_{endsill} + (9 \cdot ft + 2 \cdot in)\right] + \frac{t_{baffle}}{2}$ $arm1_{baffle} = 10.67\ ft$

$armY1_{baffle} := (2 \cdot ft + 0 \cdot in) + \frac{H_{baffle}}{2}$ Approximate. $armY1_{baffle} = 4.58\ ft$

H3

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

$WT2_{baffle} := \gamma_c \cdot (8 \cdot in) \cdot (2 \cdot ft + 0 \cdot in) \cdot L_{baffle}$ $WT2_{baffle} = 2.40 \cdot kip$

$arm2_{baffle} := arm1_{baffle} + \frac{t_{baffle}}{2} + \frac{2 \cdot ft + 0 \cdot in}{2}$ $arm2_{baffle} = 12.17\ ft$

$armY2_{baffle} := (2 \cdot ft + 0 \cdot in) + H_{baffle} + \frac{8 \cdot in}{2}$ $armY2_{baffle} = 7.50\ ft$

H3

$WT_{baffle} := WT1_{baffle} + WT2_{baffle}$ $WT_{baffle} = 10.80 \cdot kip$

$arm_{baffle} := \frac{WT1_{baffle} \cdot arm1_{baffle} + WT2_{baffle} \cdot arm2_{baffle}}{WT_{baffle}}$ $arm_{baffle} = 11.00\ ft$

$armY_{baffle} := \frac{WT1_{baffle} \cdot armY1_{baffle} + WT2_{baffle} \cdot armY2_{baffle}}{WT_{baffle}}$ $armY_{baffle} = 5.23\ ft$

Endsill

$L_{endsill} := L_{headwall}$ $L_{endsill} = 12.00\ ft$

$WT_{endsill} := \gamma_c \cdot (2 \cdot ft + 0 \cdot in) \cdot L_{endsill} \cdot t_{endsill}$ $WT_{endsill} = 3.60 \cdot kip$

$arm_{endsill} := \frac{t_{endsill}}{2}$ $arm_{endsill} = 0.50\ ft$

$armY_{endsill} := \frac{2 \cdot ft + 0 \cdot in}{2}$ $armY_{endsill} = 1.00\ ft$

Other

Ignore fillets as pipe opening was ignored.

Define Check Function: $Check(x, y) := if(x \geq y, "OK", "NG")$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

Wingwalls

$L_{wing} := 4 \cdot ft + 0 \cdot in$ (each)

$WT_{wing} := (2) \cdot \gamma_c \cdot (9843.91 \cdot ft - EL_{Bslab}) \cdot L_{wing} \cdot t_{endsill}$ $WT_{wing} = 9.00 \cdot kip$

$arm_{wing} := \frac{t_{endsill}}{2}$ $arm_{wing} = 0.50\ ft$ $armY_{wing} := \frac{(9843.91 \cdot ft - EL_{Bslab})}{2}$ $armY_{wing} = 3.75\ ft$

Total Concrete:

$WT_{concrete} := WT_{slab} + WT_{abut} + WT_{headwall} + WT_{baffle} + WT_{endsill} + WT_{wing}$ $WT_{concrete} = 181.65 \cdot kip$

$Mw_{concrete} := WT_{slab} \cdot arm_{slab} + WT_{abut} \cdot arm_{abut} + WT_{headwall} \cdot arm_{headwall} \ \cdots$
$+ WT_{baffle} \cdot arm_{baffle} + (WT_{endsill} \cdot arm_{endsill} + WT_{wing} \cdot arm_{wing})$ $Mw_{concrete} = 1855.6 \cdot kip \cdot ft$

$armX_{concrete} := \frac{Mw_{concrete}}{WT_{concrete}}$ $armX_{concrete} = 10.22\ ft$

$armY_{concrete} := \left(\frac{1}{WT_{concrete}}\right) \cdot \begin{pmatrix} WT_{slab} \cdot armY_{slab} + WT_{abut} \cdot armY_{abut} + WT_{headwall} \cdot armY_{headwall} \ \cdots \\ + WT_{baffle} \cdot armY_{baffle} + WT_{endsill} \cdot armY_{endsill} + WT_{wing} \cdot armY_{wing} \end{pmatrix}$ $armY_{concrete} = 3.05\ ft$

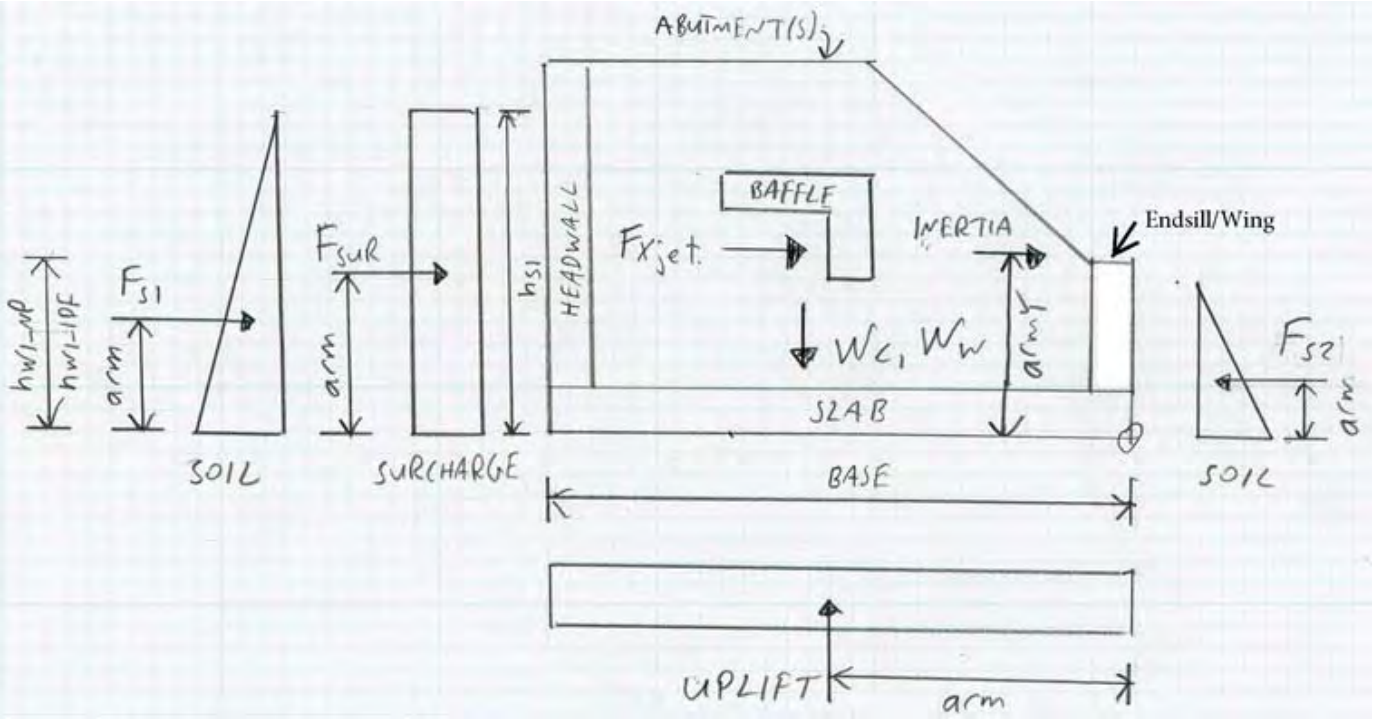


Figure 1 - Loads

Ayres Associates Inc
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Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

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Stability

Inside Water

- Normal -> ignore

- IDF -> entire inside full from headwall to endsill with IDF El.
(ignore on endsill so do not have to remove baffle area)

$A_{water_IDF} := L_{headwall} \cdot (Base_{slab} - t_{walls} - t_{endsill})$ $A_{water_IDF} = 194.00 \cdot ft^2$

$WT_{water_IDF} := \gamma_w \cdot (EL_{w_IDF} - EL_{Tslab}) \cdot A_{water_IDF}$ $WT_{water_IDF} = 53.11 \cdot kip$

$arm_{water_IDF} := t_{endsill} + \frac{(Base_{slab} - t_{walls} - t_{endsill})}{2}$ $arm_{water_IDF} = 9.08\,ft$

$Mw_{water_IDF} := WT_{water_IDF} \cdot arm_{water_IDF}$ $Mw_{water_IDF} = 482.4 \cdot kip \cdot ft$

Uniform Uplift - Normal

$p_{w_NP} := \gamma_w \cdot (EL_{w_NP} - EL_{Bslab})$ $p_{w_NP} = 0.156 \cdot ksf$

$V_UP := p_{w_NP} \cdot (Base_{slab} \cdot Width_{slab} + 2 \cdot L_{wing} \cdot t_{endsill})$ $V_UP = 45.00 \cdot kip$

$armUP := \frac{Base_{slab}}{2}$ [ignore minor wingwall area] $armUP = 9.33\,ft$

$MV_UP := V_UP \cdot armUP$ $MV_UP = 420.0 \cdot kip \cdot ft$

Uniform Uplift - IDF

$p_{w_IDF} := \gamma_w \cdot (EL_{w_IDF} - EL_{Bslab})$ $p_{w_IDF} = 0.368 \cdot ksf$

$V_UPf := p_{w_IDF} \cdot (Base_{slab} \cdot Width_{slab} + 2 \cdot L_{wing} \cdot t_{endsill})$ $V_UPf = 105.84 \cdot kip$

$armUPf := \frac{Base_{slab}}{2}$ [ignore minor wingwall area] $armUPf = 9.33\,ft$

$MV_UPf := V_UPf \cdot armUPf$ $MV_UPf = 987.8 \cdot kip \cdot ft$

Flotation

Flotation - Normal and IDF

Ref. 3, Section 3-8.

Note: Include water in W_c as it is maintained by downstream channel conditions.

$W_s := WT_{concrete}$ $\quad W_s = 181.65 \cdot kip$

$W_{c_NP} := 0 \cdot kip$ $\quad W_{c_NP} = 0 \cdot kip$

$W_{c_IDF} := WT_{water_IDF}$ $\quad W_{c_IDF} = 53.11 \cdot kip$

$S := 0 \cdot kip$ $\quad S = 0 \cdot kip$

$U_{NP} := V_UP$ $\quad U_{NP} = 45.00 \cdot kip$

$U_{IDF} := V_UPf$ $\quad U_{IDF} = 105.84 \cdot kip$

$W_g := 0 \cdot kip$ $\quad W_g = 0.00 \cdot kip$

$$SF_{flotation_NP} := \frac{W_s + W_{c_NP} + S}{U_{NP} - W_g}$$

$SF_{flotation_NP} = 4.04$

$Check(SF_{flotation_NP}, 1.3) = "OK"$

$$SF_{flotation_IDF} := \frac{W_s + W_{c_IDF} + S}{U_{IDF} - W_g}$$

$SF_{flotation_IDF} = 2.22$

$Check(SF_{flotation_IDF}, 1.1) = "OK"$

*** Structure meets flotation requirements. ***

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Stability.xmcd
Page 9 of 26

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

HORIZONTAL FORCES

-- see Figure 1 (arms from base)

$Base := Base_{slab} = 18.67\ ft$

$Width := Width_{slab} = 15.00\ ft$

Soil At-Rest and Active Earth Pressure Coefficients:

$K_{o_hA} := (1 - \sin(\phi)) \cdot (1 + \sin(\beta_{hA}))$ $K_{o_hA} = 0.517$

$K_{o_t} := (1 - \sin(\phi)) \cdot (1 + \sin(\beta_t))$ $K_{o_t} = 0.470$

$$K_{a_hB} := \frac{\cos(\phi)^2}{\left(1 + \sqrt{\frac{\sin(\phi) \cdot \sin(\phi - \beta_{hB})}{\cos(\beta_{hB})}}\right)^2} \qquad K_{a_hB} = 0.340$$

Moist Soil

$F_{s1Am_full} := \frac{1}{2} \cdot Width \cdot K_{o_hA} \cdot \gamma_m \cdot h_{s1A}^2$ $F_{s1Am_full} = 44.56 \cdot kip$

$armF_{s1Am_full} := \frac{h_{s1A}}{3}$ $armF_{s1Am_full} = 3.20\ ft$

Pressure:

$F_{s1Bm_full} := \frac{1}{2} \cdot (2 \cdot L_{wing}) \cdot K_{a_hB} \cdot \gamma_m \cdot h_{s1B}^2$ $F_{s1Bm_full} = 6.31 \cdot kip$

$\sigma := \gamma_m \cdot h_{s1B}$ $\sigma = 0.761 \cdot ksf$

$armF_{s1Bm_full} := \frac{h_{s1B}}{3}$ $armF_{s1Bm_full} = 2.03\ ft$

-> Remove Force at 30" Inside Diameter Pipe
- There is a 6" concrete topping around entire pipe. Use 42-inch diameter and ignore weight for stress.
- Use average centerline pressure to remove uniform section

$EL_{pipe_cL} := EL_{pipe_invert} + \frac{30 \cdot in}{2}$ $EL_{pipe_cL} = 9841.06\ ft$

$\sigma_{s1m_pipe} := \gamma_m \cdot (EL_{soilA} - EL_{pipe_cL})$ $\sigma_{s1m_pipe} = 0.618 \cdot ksf$

$F_{s1m_pipe} := K_{o_hA} \cdot \left[\left(\frac{\pi}{4}\right) \cdot (42 \cdot in)^2\right] \cdot \sigma_{s1m_pipe}$ $F_{s1m_pipe} = 3.07 \cdot kip$

$armF_{s1m_pipe} := EL_{pipe_cL} - EL_{Bslab}$ $armF_{s1m_pipe} = 4.65\ ft$

$F_{s1m} := F_{s1Am_full} - F_{s1m_pipe} + F_{s1Bm_full}$ $F_{s1m} = 47.80 \cdot kip$

$M_{s1m} := F_{s1Am_full} \cdot armF_{s1Am_full} - F_{s1m_pipe} \cdot armF_{s1m_pipe} + F_{s1Bm_full} \cdot armF_{s1Bm_full}$ $M_{s1m} = 141.0 \cdot kip \cdot ft$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

$arm_{s1m} := \frac{M_{s1m}}{F_{s1m}}$ $arm_{s1m} = 2.95\ ft$

$F_{s2m} := \frac{1}{2} \cdot Width \cdot K_{o_t} \cdot \gamma_m \cdot h_{s2}^2$ $F_{s2m} = 4.65 \cdot kip$ $armF_{s2m} := \frac{h_{s2}}{3}$ $armF_{s2m} = 1.08\ ft$

$M_{s2m} := F_{s2m} \cdot armF_{s2m}$ $M_{s2m} = 5.0 \cdot kip \cdot ft$

Saturated Soil

Pressures:

$\sigma_{1A_NP} := \gamma_m \cdot (h_{s1A} - h_{w1_NP})$ $\sigma_{1A_NP} = 0.886 \cdot ksf$

$\sigma_{2A_NP} := \sigma_{1A_NP} + (\gamma_s - \gamma_w) \cdot h_{w1_NP}$ $\sigma_{2A_NP} = 1.055 \cdot ksf$

$\sigma_{1A_IDF} := \gamma_m \cdot (h_{s1A} - h_{w1_IDF})$ $\sigma_{1A_IDF} = 0.464 \cdot ksf$

$\sigma_{2A_IDF} := \sigma_{1A_IDF} + (\gamma_s - \gamma_w) \cdot h_{w1_IDF}$ $\sigma_{2A_IDF} = 0.861 \cdot ksf$

$F_{s1A_NPa} := \frac{1}{2} \cdot Width \cdot K_{o_hA} \cdot (\sigma_{1A_NP}) \cdot (h_{s1A} - h_{w1_NP})$ $armF_{s1A_NPa} := h_{w1_NP} + \frac{h_{s1A} - h_{w1_NP}}{3}$

$F_{s1A_NPa} = 24.35 \cdot kip$ $armF_{s1A_NPa} = 4.86\ ft$

$F_{s1A_NPb} := Width \cdot K_{o_hA} \cdot (\sigma_{1A_NP}) \cdot h_{w1_NP}$ $armF_{s1A_NPb} := \frac{h_{w1_NP}}{2}$

$F_{s1A_NPb} = 17.17 \cdot kip$ $armF_{s1A_NPb} = 1.25\ ft$

$F_{s1A_NPc} := \frac{1}{2} \cdot Width \cdot K_{o_hA} \cdot (\sigma_{2A_NP} - \sigma_{1A_NP}) \cdot h_{w1_NP}$ $armF_{s1A_NPc} := \frac{h_{w1_NP}}{3}$

$F_{s1A_NPc} = 1.64 \cdot kip$ $armF_{s1A_NPc} = 0.83\ ft$

-> Remove Force at Pipe

$\sigma_{s1_NPpipe} := if\left[EL_{w_NP} > EL_{pipe_cL}, (\gamma_s - \gamma_w) \cdot (EL_{soilA} - EL_{pipe_cL}), \gamma_m \cdot (EL_{soilA} - EL_{pipe_cL})\right]$ $\sigma_{s1_NPpipe} = 0.618 \cdot ksf$

$F_{s1_NPpipe} := K_{o_hA} \cdot \left[\left(\frac{\pi}{4}\right) \cdot (42 \cdot in)^2\right] \cdot \sigma_{s1_NPpipe}$ $F_{s1_NPpipe} = 3.07 \cdot kip$

$armF_{s1_NPpipe} := armF_{s1m_pipe}$ $armF_{s1_NPpipe} = 4.65\ ft$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Stability.xmcd
Page 11 of 26

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

Pressures:

$\sigma_{1B_NP} := \gamma_m \cdot (h_{s1B} - h_{w1_NP})$ $\quad \sigma_{1B_NP} = 0.449 \cdot ksf$

$\sigma_{2B_NP} := \sigma_{1B_NP} + (\gamma_s - \gamma_w) \cdot h_{w1_NP}$ $\quad \sigma_{2B_NP} = 0.618 \cdot ksf$

$\sigma_{1B_IDF} := \gamma_m \cdot (h_{s1B} - h_{w1_IDF})$ $\quad \sigma_{1B_IDF} = 0.026 \cdot ksf$

$\sigma_{2B_IDF} := \sigma_{1B_IDF} + (\gamma_s - \gamma_w) \cdot h_{w1_IDF}$ $\quad \sigma_{2B_IDF} = 0.423 \cdot ksf$

$F_{s1B_NPa} := \frac{1}{2} \cdot (2 \cdot L_{wing}) \cdot K_{a_hB} \cdot (\sigma_{1B_NP}) \cdot (h_{s1B} - h_{w1_NP})$ $\quad armF_{s1B_NPa} := h_{w1_NP} + \frac{h_{s1B} - h_{w1_NP}}{3}$

$F_{s1B_NPa} = 2.19 \cdot kip$ $\quad armF_{s1B_NPa} = 3.70\ ft$

$F_{s1B_NPb} := (2 \cdot L_{wing}) \cdot K_{a_hB} \cdot (\sigma_{1B_NP}) \cdot h_{w1_NP}$ $\quad armF_{s1B_NPb} := \frac{h_{w1_NP}}{2}$

$F_{s1B_NPb} = 3.06 \cdot kip$ $\quad armF_{s1B_NPb} = 1.25\ ft$

$F_{s1B_NPc} := \frac{1}{2} \cdot (2 \cdot L_{wing}) \cdot K_{a_hB} \cdot (\sigma_{2B_NP} - \sigma_{1B_NP}) \cdot h_{w1_NP}$ $\quad armF_{s1B_NPc} := \frac{h_{w1_NP}}{3}$

$F_{s1B_NPc} = 0.57 \cdot kip$ $\quad armF_{s1B_NPc} = 0.83\ ft$

$F_{s1_NP} := (F_{s1A_NPa} + F_{s1A_NPb} + F_{s1A_NPc}) - F_{s1_NPpipe} + (F_{s1B_NPa} + F_{s1B_NPb} + F_{s1B_NPc})$ $\quad F_{s1_NP} = 45.92 \cdot kip$

$M_{s1_NP} := (F_{s1A_NPa} \cdot armF_{s1A_NPa} + F_{s1A_NPb} \cdot armF_{s1A_NPb} + F_{s1A_NPc} \cdot armF_{s1A_NPc}) - F_{s1_NPpipe} \cdot armF_{s1_NPpipe}$ ...
$+ (F_{s1B_NPa} \cdot armF_{s1B_NPa} + F_{s1B_NPb} \cdot armF_{s1B_NPb} + F_{s1B_NPc} \cdot armF_{s1B_NPc})$

$M_{s1_NP} = 139.4 \cdot kip \cdot ft$

$arm_{s1_NP} := \frac{M_{s1_NP}}{F_{s1_NP}}$ $\quad arm_{s1_NP} = 3.04\ ft$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Stability.xmcd
Page 12 of 26

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

$F_{s1A_IDFa} := \frac{1}{2}\cdot Width\cdot K_{o_hA}\cdot(\sigma_{1A_IDF})\cdot(h_{s1A} - h_{w1_IDF})$

$F_{s1A_IDFa} = 6.67\cdot kip$

$armF_{s1A_IDFa} := h_{w1_IDF} + \frac{h_{s1A} - h_{w1_IDF}}{3}$

$armF_{s1A_IDFa} = 7.12\ ft$

$F_{s1A_IDFb} := Width\cdot K_{o_hA}\cdot(\sigma_{1A_IDF})\cdot h_{w1_IDF}$

$F_{s1A_IDFb} = 21.14\cdot kip$

$armF_{s1A_IDFb} := \frac{h_{w1_IDF}}{2}$

$armF_{s1A_IDFb} = 2.94\ ft$

$F_{s1A_IDFc} := \frac{1}{2}\cdot Width\cdot K_{o_hA}\cdot(\sigma_{2A_IDF} - \sigma_{1A_IDF})\cdot h_{w1_IDF}$

$F_{s1A_IDFc} = 9.05\cdot kip$

$armF_{s1A_IDFc} := \frac{h_{w1_IDF}}{3}$

$armF_{s1A_IDFc} = 1.96\ ft$

-> Remove Force at Pipe

$\sigma_{s1_IDFpipe} := if\left[EL_{w_IDF} > EL_{pipe_cL}, (\gamma_s - \gamma_w)\cdot(EL_{soilA} - EL_{pipe_cL}), \gamma_m\cdot(EL_{soilA} - EL_{pipe_cL})\right]$

$\sigma_{s1_IDFpipe} = 0.333\cdot ksf$

$F_{s1_IDFpipe} := K_{o_hA}\cdot\left[\left(\frac{\pi}{4}\right)\cdot(42\cdot in)^2\right]\cdot\sigma_{s1_IDFpipe}$

$F_{s1_IDFpipe} = 1.66\cdot kip$

$armF_{s1_IDFpipe} := armF_{s1m_pipe}$

$armF_{s1_IDFpipe} = 4.65\ ft$

$F_{s1B_IDFa} := \frac{1}{2}\cdot(2\cdot L_{wing})\cdot K_{a_hB}\cdot(\sigma_{1B_IDF})\cdot(h_{s1B} - h_{w1_IDF})$

$F_{s1B_IDFa} = 0.01\cdot kip$

$armF_{s1B_IDFa} := h_{w1_IDF} + \frac{h_{s1B} - h_{w1_IDF}}{3}$

$armF_{s1B_IDFa} = 5.95\ ft$

$F_{s1B_IDFb} := (2\cdot L_{wing})\cdot K_{a_hB}\cdot(\sigma_{1B_IDF})\cdot h_{w1_IDF}$

$F_{s1B_IDFb} = 0.42\cdot kip$

$armF_{s1B_IDFb} := \frac{h_{w1_IDF}}{2}$

$armF_{s1B_IDFb} = 2.94\ ft$

$F_{s1B_IDFc} := \frac{1}{2}\cdot(2\cdot L_{wing})\cdot K_{a_hB}\cdot(\sigma_{2B_IDF} - \sigma_{1B_IDF})\cdot h_{w1_IDF}$

$F_{s1B_IDFc} = 3.18\cdot kip$

$armF_{s1B_IDFc} := \frac{h_{w1_IDF}}{3}$

$armF_{s1B_IDFc} = 1.96\ ft$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

$F_{s1_IDF} := \left(F_{s1A_IDFa} + F_{s1A_IDFb} + F_{s1A_IDFc}\right) - F_{s1_IDFpipe} + \left(F_{s1B_IDFa} + F_{s1B_IDFb} + F_{s1B_IDFc}\right)$ $F_{s1_IDF} = 38.80 \cdot kip$

$M_{s1_IDF} := \left(F_{s1A_IDFa} \cdot armF_{s1A_IDFa} + F_{s1A_IDFb} \cdot armF_{s1A_IDFb} + F_{s1A_IDFc} \cdot armF_{s1A_IDFc}\right) - F_{s1_IDFpipe} \cdot armF_{s1_IDFpipe} \cdots$
$+ \left(F_{s1B_IDFa} \cdot armF_{s1B_IDFa} + F_{s1B_IDFb} \cdot armF_{s1B_IDFb} + F_{s1B_IDFc} \cdot armF_{s1B_IDFc}\right)$

$M_{s1_IDF} = 127.1 \cdot kip \cdot ft$

$arm_{s1_IDF} := \frac{M_{s1_IDF}}{F_{s1_IDF}}$ $arm_{s1_IDF} = 3.28\ ft$

Toe Soil

$F_{s2} := \frac{1}{2} \cdot Width \cdot K_{o_t} \cdot \left(\gamma_s - \gamma_w\right) \cdot h_{s2}^{\ 2}$ $F_{s2} = 2.51 \cdot kip$ $armF_{s2} := \frac{h_{s2}}{3}$ $armF_{s2} = 1.08\ ft$

$M_{s2} := F_{s2} \cdot armF_{s2}$ $M_{s2} = 2.7 \cdot kip \cdot ft$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

Water Equal on both sides so do not compute since cancel each other out.
(Construction does not have water present).

Surcharge (Only On Headwall)

$F_{surcharge_full} := K_{o_hA} \cdot q_{surcharge} \cdot Width \cdot h_{s1A}$ $F_{surcharge_full} = 14.87 \cdot kip$

$armF_{surcharge_full} := \frac{h_{s1A}}{2}$ $armF_{surcharge_full} = 4.80\ ft$

-> Remove Force at Pipe

$F_{sur_pipe} := K_{o_hA} \cdot \left[\left(\frac{\pi}{4}\right) \cdot (42 \cdot in)^2\right] \cdot q_{surcharge}$ $F_{sur_pipe} = 0.99 \cdot kip$

$armF_{sur_pipe} := armF_{s1m_pipe}$ $armF_{sur_pipe} = 4.65\ ft$

$F_{surcharge} := F_{surcharge_full} - F_{sur_pipe}$ $F_{surcharge} = 13.87 \cdot kip$

$MH_Sur := F_{surcharge_full} \cdot armF_{surcharge_full} - F_{sur_pipe} \cdot armF_{sur_pipe}$ $MH_Sur = 66.7 \cdot kip \cdot ft$

$armF_{surcharge} := \frac{MH_Sur}{F_{surcharge}}$ $armF_{surcharge} = 4.81\ ft$

Force from Pipe Water Jet IDF only

$F = \rho A v^2$ where $\rho_{water} = \gamma_w / g = 1.94$ slug/cf and $g = 32.17 \cdot \frac{ft}{s^2}$

Pipe Diameter: $D_{pipe} := 29 \cdot in$

Velocity: $v := 34.2 \frac{ft}{s}$

Forces: $Fx_{jet} := \frac{\gamma_w}{g} \cdot \left[\left(\frac{\pi}{4}\right) \cdot D_{pipe}^2\right] \cdot v^2$ $Fx_{jet} = 10.42 \cdot kip$

$armFx_{jet} := \left(EL_{pipe_invert} - EL_{Bslab}\right) + \frac{30 \cdot in}{2}$ $armFx_{jet} = 4.65\ ft$

$MH_Fx_{jet} := Fx_{jet} \cdot armFx_{jet}$ $MH_Fx_{jet} = 48.5 \cdot kip \cdot ft$

Stability

Earthquake for Load Cases 2, 5 and 6

The earthquake load cases again are:

LC 2 - Extreme: Construction with OBE
LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
LC 6 - Extreme: Coincident Pool = Normal Pool with MDE

Mononobe-Okabe Analysis (Ref. 3, Appendix G)

Usual case when k_v, δ, and θ are zero.

N/A since using at-rest earth pressures for main structure. Use alternate method.

Seismic Coefficient Method (Ref. 6, Section 7-3a)

Use the provided coefficient to compute the structure inertial force.

- Horizontal Inertia from vertical weight.

$WT_{concrete} = 181.65 \cdot kip$

$armY_{concrete} = 3.05\ ft$

$Inertia_{str_OBE} := k_{h_OBE} \cdot WT_{concrete}$ $\quad Inertia_{str_OBE} = 14.53 \cdot kip$

$M_Inertia_{str_OBE} := Inertia_{str_OBE} \cdot armY_{concrete}$ $\quad M_Inertia_{str_OBE} = 44.4 \cdot kip \cdot ft$

$Inertia_{str_MDE} := k_{h_MDE} \cdot WT_{concrete}$ $\quad Inertia_{str_MDE} = 33.61 \cdot kip$

$M_Inertia_{str_MDE} := Inertia_{str_MDE} \cdot armY_{concrete}$ $\quad M_Inertia_{str_MDE} = 102.6 \cdot kip \cdot ft$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

SUM OF VERTICAL FORCES

LC1 & LC2 $sumV_{1_2} := WT_{concrete}$ $sumV_{1_2} = 181.65 \cdot kip$

LC3 & LC4 & LC5 & LC6 $sumV_{3_4_5_6} := WT_{concrete} - V_UP$ $sumV_{3_4_5_6} = 136.65 \cdot kip$

LC7 $sumV_7 := WT_{concrete} + WT_{water_IDF} - V_UPf$ $sumV_7 = 128.92 \cdot kip$

SUM OF HORIZONTAL FORCES w/o Key

LC1 $sumH_1 := F_{s1m} + F_{surcharge} - F_{s2m}$ $sumH_1 = 57.02 \cdot kip$

LC2 $sumH_2 := F_{s1m} + Inertia_{str_OBE} - F_{s2m}$ $sumH_2 = 57.68 \cdot kip$

LC3 $sumH_3 := F_{s1_NP} - F_{s2}$ $sumH_3 = 43.40 \cdot kip$

LC4 $sumH_4 := F_{s1_NP} + F_{surcharge} - F_{s2}$ $sumH_4 = 57.28 \cdot kip$

LC5 $sumH_5 := F_{s1_NP} + Inertia_{str_OBE} - F_{s2}$ $sumH_5 = 57.93 \cdot kip$

LC6 $sumH_6 := F_{s1_NP} + Inertia_{str_MDE} - F_{s2}$ $sumH_6 = 77.01 \cdot kip$

LC7 $sumH_7 := F_{s1_IDF} + Fx_{jet} - F_{s2}$ $sumH_7 = 46.71 \cdot kip$

RESISTING MOMENT (moments about point A)

$M_{R_conc} := Mw_{concrete}$ $M_{R_conc} = 1855.6 \cdot kip \cdot ft$

$M_{R_water_IDF} := Mw_{water_IDF}$ $M_{R_water_IDF} = 482.4 \cdot kip \cdot ft$

$M_{R1_2} := M_{R_conc} + M_{s2m}$ $M_{R1_2} = 1860.7 \cdot kip \cdot ft$

$M_{R3_4_5_6} := M_{R_conc} + M_{s2}$ $M_{R3_4_5_6} = 1858.4 \cdot kip \cdot ft$

$M_{R7} := M_{R_conc} + M_{R_water_IDF} + M_{s2}$ $M_{R7} = 2340.8 \cdot kip \cdot ft$

Stability

OVERTURNING MOMENT (moments about point A) w/o Uplift

LC1 $M_{O_1} := M_{s1m} + MH_Sur$ $M_{O_1} = 207.6 \cdot kip \cdot ft$

LC2 $M_{O_2} := M_{s1m} + M_Inertia_{str_OBE}$ $M_{O_2} = 185.3 \cdot kip \cdot ft$

LC3 $M_{O_3} := M_{s1_NP}$ $M_{O_3} = 139.4 \cdot kip \cdot ft$

LC4 $M_{O_4} := M_{s1_NP} + MH_Sur$ $M_{O_4} = 206.1 \cdot kip \cdot ft$

LC5 $M_{O_5} := M_{s1_NP} + M_Inertia_{str_OBE}$ $M_{O_5} = 183.8 \cdot kip \cdot ft$

LC6 $M_{O_6} := M_{s1_NP} + M_Inertia_{str_MDE}$ $M_{O_6} = 242.0 \cdot kip \cdot ft$

LC7 $M_{O_7} := M_{s1_IDF} + MH_Fx_{jet}$ $M_{O_7} = 175.6 \cdot kip \cdot ft$

SLIDING SAFETY FACTORS --> Use shear key if required. See details after bearing capacity.

$FS1_{sliding} := \frac{(sumV_{1_2}) \cdot \tan(\phi_{foundation})}{sumH_1}$ $FS1_{sliding} = 1.22$ $Check(FS1_{sliding}, 1.3) = "NG"$

$FS2_{sliding} := \frac{(sumV_{1_2}) \cdot \tan(\phi_{foundation})}{sumH_2}$ $FS2_{sliding} = 1.21$ $Check(FS2_{sliding}, 1.1) = "OK"$

$FS3_{sliding} := \frac{(sumV_{3_4_5_6}) \cdot \tan(\phi_{foundation})}{sumH_3}$ $FS3_{sliding} = 1.21$ $Check(FS3_{sliding}, 1.7) = "NG"$

$FS4_{sliding} := \frac{(sumV_{3_4_5_6}) \cdot \tan(\phi_{foundation})}{sumH_4}$ $FS4_{sliding} = 0.92$ $Check(FS4_{sliding}, 1.3) = "NG"$

$FS5_{sliding} := \frac{(sumV_{3_4_5_6}) \cdot \tan(\phi_{foundation})}{sumH_5}$ $FS5_{sliding} = 0.91$ $Check(FS5_{sliding}, 1.3) = "NG"$

$FS6_{sliding} := \frac{(sumV_{3_4_5_6}) \cdot \tan(\phi_{foundation})}{sumH_6}$ $FS6_{sliding} = 0.68$ $Check(FS6_{sliding}, 1.1) = "NG"$

$FS7_{sliding} := \frac{(sumV_7) \cdot \tan(\phi_{foundation})}{sumH_7}$ $FS7_{sliding} = 1.06$ $Check(FS7_{sliding}, 1.1) = "NG"$

Ayres Associates Inc
Eau Claire, WI

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

BEARING CAPACITY

Ref. 5

R = Resultant from toe & e = eccentricity from base centroid [ignore minor wingwall area]

$$R1 := \frac{M_{R1_2} - (M_{O_1})}{sumV_{1_2}} \qquad R1 = 9.100\ ft$$

$$R2 := \frac{M_{R1_2} - (M_{O_2})}{sumV_{1_2}} \qquad R2 = 9.223\ ft$$

$$e1 := \frac{Base}{2} - R1 \qquad e1 = 0.233\ ft$$

$$e2 := \frac{Base}{2} - R2 \qquad e2 = 0.110\ ft$$

$$R3 := \frac{M_{R3_4_5_6} - (M_{O_3} + MV_UP)}{sumV_{3_4_5_6}} \qquad R3 = 9.506\ ft$$

$$R4 := \frac{M_{R3_4_5_6} - (M_{O_4} + MV_UP)}{sumV_{3_4_5_6}} \qquad R4 = 9.018\ ft$$

$$e3 := \frac{Base}{2} - R3 \qquad e3 = -0.173\ ft$$

$$e4 := \frac{Base}{2} - R4 \qquad e4 = 0.315\ ft$$

$$R5 := \frac{M_{R3_4_5_6} - (M_{O_5} + MV_UP)}{sumV_{3_4_5_6}} \qquad R5 = 9.181\ ft$$

$$R6 := \frac{M_{R3_4_5_6} - (M_{O_6} + MV_UP)}{sumV_{3_4_5_6}} \qquad R6 = 8.755\ ft$$

$$e5 := \frac{Base}{2} - R5 \qquad e5 = 0.152\ ft$$

$$e6 := \frac{Base}{2} - R6 \qquad e6 = 0.578\ ft$$

$$R7 := \frac{M_{R7} - (M_{O_7} + MV_UPf)}{sumV_7} \qquad R7 = 9.133\ ft$$

$$e7 := \frac{Base}{2} - R7 \qquad e7 = 0.201\ ft$$

where $\frac{Base}{3} = 6.222\ ft$ and $\frac{2 \cdot Base}{3} = 12.444\ ft$

Stability

$$For1 := if\left[R1 > \left(\frac{1}{3}\cdot Base\right), if\left[R1 > \left(\frac{2}{3}\cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right] \qquad For1 = \text{"In Kern"}$$

$$For2 := if\left[R2 > \left(\frac{1}{3}\cdot Base\right), if\left[R2 > \left(\frac{2}{3}\cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right] \qquad For2 = \text{"In Kern"}$$

$$For3 := if\left[R3 > \left(\frac{1}{3}\cdot Base\right), if\left[R3 > \left(\frac{2}{3}\cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right] \qquad For3 = \text{"In Kern"}$$

$$For4 := if\left[R4 > \left(\frac{1}{3}\cdot Base\right), if\left[R4 > \left(\frac{2}{3}\cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right] \qquad For4 = \text{"In Kern"}$$

$$For5 := if\left[R5 > \left(\frac{1}{3}\cdot Base\right), if\left[R5 > \left(\frac{2}{3}\cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right] \qquad For5 = \text{"In Kern"}$$

$$For6 := if\left[R6 > \left(\frac{1}{3}\cdot Base\right), if\left[R6 > \left(\frac{2}{3}\cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right] \qquad For6 = \text{"In Kern"}$$

$$For7 := if\left[R7 > \left(\frac{1}{3}\cdot Base\right), if\left[R7 > \left(\frac{2}{3}\cdot Base\right), \text{"Not In Kern"}, \text{"In Kern"}\right], \text{"Not In Kern"}\right] \qquad For7 = \text{"In Kern"}$$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

Percent Base In Compression

Table 4-1, Page 4-5.

$$\%Base_Comp1 := if\left[For1 = "In\ Kern",\ 1,\ \left(\frac{3\cdot R1}{Base}\right)\right] \qquad \%Base_Comp1 = 100.0\cdot\% \qquad Check(\%Base_Comp1, 0.75) = "OK"$$

$$\%Base_Comp2 := if\left[For2 = "In\ Kern",\ 1,\ \left(\frac{3\cdot R2}{Base}\right)\right] \qquad \%Base_Comp2 = 100.0\cdot\% \qquad Check(\%Base_Comp2, 0.01) = "OK"$$

$$\%Base_Comp3 := if\left[For3 = "In\ Kern",\ 1,\ \left(\frac{3\cdot R3}{Base}\right)\right] \qquad \%Base_Comp3 = 100.0\cdot\% \qquad Check(\%Base_Comp3, 1.0) = "OK"$$

$$\%Base_Comp4 := if\left[For4 = "In\ Kern",\ 1,\ \left(\frac{3\cdot R4}{Base}\right)\right] \qquad \%Base_Comp4 = 100.0\cdot\% \qquad Check(\%Base_Comp4, 0.75) = "OK"$$

$$\%Base_Comp5 := if\left[For5 = "In\ Kern",\ 1,\ \left(\frac{3\cdot R5}{Base}\right)\right] \qquad \%Base_Comp5 = 100.0\cdot\% \qquad Check(\%Base_Comp5, 0.75) = "OK"$$

$$\%Base_Comp6 := if\left[For6 = "In\ Kern",\ 1,\ \left(\frac{3\cdot R6}{Base}\right)\right] \qquad \%Base_Comp6 = 100.0\cdot\% \qquad Check(\%Base_Comp6, 0.01) = "OK"$$

$$\%Base_Comp7 := if\left[For7 = "In\ Kern",\ 1,\ \left(\frac{3\cdot R7}{Base}\right)\right] \qquad \%Base_Comp7 = 100.0\cdot\% \qquad Check(\%Base_Comp7, 0.01) = "OK"$$

Heel Pressure (Equ. 3-1)

$$Press1_{Heel} := if\left[For1 = "In\ Kern",\ \left(\frac{sumV_{1_2}}{Base\cdot Width}\right)\cdot\left(1-\frac{6\cdot e1}{Base}\right),\ 0\cdot psf\right] \qquad Press1_{Heel} = 600\cdot psf$$

$$Press2_{Heel} := if\left[For2 = "In\ Kern",\ \left(\frac{sumV_{1_2}}{Base\cdot Width}\right)\cdot\left(1-\frac{6\cdot e2}{Base}\right),\ 0\cdot psf\right] \qquad Press2_{Heel} = 626\cdot psf$$

$$Press3_{Heel} := if\left[For3 = "In\ Kern",\ \left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1-\frac{6\cdot e3}{Base}\right),\ 0\cdot psf\right] \qquad Press3_{Heel} = 515\cdot psf$$

$$Press4_{Heel} := if\left[For4 = "In\ Kern",\ \left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1-\frac{6\cdot e4}{Base}\right),\ 0\cdot psf\right] \qquad Press4_{Heel} = 439\cdot psf$$

$$Press5_{Heel} := if\left[For5 = "In\ Kern",\ \left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1-\frac{6\cdot e5}{Base}\right),\ 0\cdot psf\right] \qquad Press5_{Heel} = 464\cdot psf$$

$$Press6_{Heel} := if\left[For6 = "In\ Kern",\ \left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1-\frac{6\cdot e6}{Base}\right),\ 0\cdot psf\right] \qquad Press6_{Heel} = 397\cdot psf$$

$$Press7_{Heel} := if\left[For7 = "In\ Kern",\ \left(\frac{sumV_{7}}{Base\cdot Width}\right)\cdot\left(1-\frac{6\cdot e7}{Base}\right),\ 0\cdot psf\right] \qquad Press7_{Heel} = 431\cdot psf$$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Stability.xmcd
Page 21 of 26

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

Toe Pressure (Equ. 3-1 and 3-2)

$$Press1_{Toe} := if\left[For1 = "In\ Kern",\left(\frac{sumV_{1_2}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e1}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{1_2}}{(Base-2\cdot e1)\cdot Width}\right]\right] \quad Press1_{Toe} = 697\cdot psf$$

$$Press2_{Toe} := if\left[For1 = "In\ Kern",\left(\frac{sumV_{1_2}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e2}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{1_2}}{(Base-2\cdot e2)\cdot Width}\right]\right] \quad Press2_{Toe} = 672\cdot psf$$

$$Press3_{Toe} := if\left[For3 = "In\ Kern",\left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e3}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{3_4_5_6}}{(Base-2\cdot e3)\cdot Width}\right]\right] \quad Press3_{Toe} = 461\cdot psf$$

$$Press4_{Toe} := if\left[For4 = "In\ Kern",\left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e4}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{3_4_5_6}}{(Base-2\cdot e4)\cdot Width}\right]\right] \quad Press4_{Toe} = 537\cdot psf$$

$$Press5_{Toe} := if\left[For5 = "In\ Kern",\left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e5}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{3_4_5_6}}{(Base-2\cdot e5)\cdot Width}\right]\right] \quad Press5_{Toe} = 512\cdot psf$$

$$Press6_{Toe} := if\left[For6 = "In\ Kern",\left(\frac{sumV_{3_4_5_6}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e6}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{3_4_5_6}}{(Base-2\cdot e6)\cdot Width}\right]\right] \quad Press6_{Toe} = 579\cdot psf$$

$$Press7_{Toe} := if\left[For7 = "In\ Kern",\left(\frac{sumV_{7}}{Base\cdot Width}\right)\cdot\left(1+\frac{6\cdot e7}{Base}\right),\left(\frac{4}{3}\right)\cdot\left[\frac{sumV_{7}}{(Base-2\cdot e7)\cdot Width}\right]\right] \quad Press7_{Toe} = 490\cdot psf$$

Base Pressure Length (Equ. 3-3)

$$Press1_{Length} := if\left[For1 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e1)\right] \quad Press1_{Length} = 18.667\ ft$$

$$Press2_{Length} := if\left[For2 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e2)\right] \quad Press2_{Length} = 18.667\ ft$$

$$Press3_{Length} := if\left[For3 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e3)\right] \quad Press3_{Length} = 18.667\ ft$$

$$Press4_{Length} := if\left[For4 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e4)\right] \quad Press4_{Length} = 18.667\ ft$$

$$Press5_{Length} := if\left[For5 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e5)\right] \quad Press5_{Length} = 18.667\ ft$$

$$Press6_{Length} := if\left[For6 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e6)\right] \quad Press6_{Length} = 18.667\ ft$$

$$Press7_{Length} := if\left[For7 = "In\ Kern", Base, \left(\frac{3}{2}\right)\cdot(Base-2\cdot e7)\right] \quad Press7_{Length} = 18.667\ ft$$

Ayres Associates Inc
Eau Claire, WI

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

Summary

$BasePressure1 := \max(Press1_{Heel}, Press1_{Toe})$ $BasePressure1 = 697 \cdot psf$

$Check(BearingCap_{allow}, BasePressure1) = "OK"$

$BasePressure2 := \max(Press2_{Heel}, Press2_{Toe})$ $BasePressure2 = 672 \cdot psf$

$Check(BearingCap_{allow_eq}, BasePressure2) = "OK"$

$BasePressure3 := \max(Press3_{Heel}, Press3_{Toe})$ $BasePressure3 = 515 \cdot psf$

$Check(BearingCap_{allow}, BasePressure3) = "OK"$

$BasePressure4 := \max(Press4_{Heel}, Press4_{Toe})$ $BasePressure4 = 537 \cdot psf$

$Check(BearingCap_{allow}, BasePressure4) = "OK"$

$BasePressure5 := \max(Press5_{Heel}, Press5_{Toe})$ $BasePressure5 = 512 \cdot psf$

$Check(BearingCap_{allow_eq}, BasePressure5) = "OK"$

$BasePressure6 := \max(Press6_{Heel}, Press6_{Toe})$ $BasePressure6 = 579 \cdot psf$

$Check(BearingCap_{allow_eq}, BasePressure6) = "OK"$

$BasePressure7 := \max(Press7_{Heel}, Press7_{Toe})$ $BasePressure7 = 490 \cdot psf$

$Check(BearingCap_{allow}, BasePressure7) = "OK"$

Stability

SHEAR KEY

Key will be excavated into native ground, therefore, no additional driving force.
Use minimum heel pressure and key soil height.
Since factors of safety are greater than or near to 1.0, use passive earth pressure as purpose of key is to resist movement so passive will develop if movement occurs. Conservatively use 75% of passive pressure.
Not used in determining flotation or base pressure.

Key Properties $H_{key} := 1\cdot ft + 9\cdot in$ [if no key, enter 0 ft + 0 in] Value is that required for controlling load case.

PassiveEarth Pressure Coefficient: $K_{p_key} := \tan\left(45\cdot deg + \frac{\phi}{2}\right)^2$ $K_{p_key} = 3.255$

Pressures

$p_{key_Top_m} := \min(Press1_{Heel}, Press2_{Heel})$ $p_{key_Top_m} = 0.600\cdot ksf$

$p_{key_Top} := \min(Press3_{Heel}, Press4_{Heel}, Press5_{Heel}, Press6_{Heel}, Press7_{Heel})$ $p_{key_Top} = 0.397\cdot ksf$

$p_{key_Bottom_m} := p_{key_Top_m} + H_{key}\cdot\gamma_m$ $p_{key_Bottom_m} = 0.819\cdot ksf$

$p_{key_Bottom} := p_{key_Top} + H_{key}\cdot(\gamma_s - \gamma_w)$ $p_{key_Bottom} = 0.515\cdot ksf$

Resisting Native Material Forces

$Fs_{key_m} := \frac{1}{2}\cdot(0.75\cdot K_{p_key})\cdot Width\cdot(p_{key_Top_m} + p_{key_Bottom_m})\cdot H_{key}$ $Fs_{key_m} = 45.46\cdot kip$

$armFs_{key_m} := \frac{H_{key}\cdot(p_{key_Top_m} + 2\cdot p_{key_Bottom_m})}{3\cdot(p_{key_Top_m} + p_{key_Bottom_m})}$ $armFs_{key_m} = 0.92\ ft$

$Fs_{key} := \frac{1}{2}\cdot(0.75\cdot K_{p_key})\cdot Width\cdot(p_{key_Top} + p_{key_Bottom})\cdot H_{key}$

$Fs_{key} = 29.24\cdot kip$

$armFs_{key} := \frac{H_{key}\cdot(p_{key_Top} + 2\cdot p_{key_Bottom})}{3\cdot(p_{key_Top} + p_{key_Bottom})}$

$armFs_{key} = 0.91\ ft$

Weight

$WT_{key} := \gamma_c\cdot Width\cdot H_{key}\cdot t_{walls}$

$WT_{key} = 5.91\cdot kip$

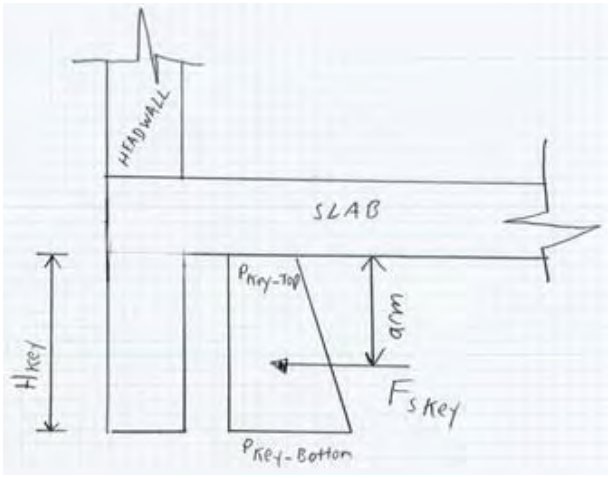

Figure 2 - Shear Key

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

Key: Force/Shear Per Foot of Width

$Fkey1_{resist} := \left(\frac{1\cdot ft}{Width_{slab}}\right)\cdot\left(sumH_1 - Fs_{key_m}\right)$ $Fkey1_{resist} = 0.77\cdot kip$

$Fkey2_{resist} := \left(\frac{1\cdot ft}{Width_{slab}}\right)\cdot\left(sumH_2 - Fs_{key_m}\right)$ $Fkey2_{resist} = 0.81\cdot kip$

$Fkey3_{resist} := \left(\frac{1\cdot ft}{Width_{slab}}\right)\cdot\left(sumH_3 - Fs_{key}\right)$ $Fkey3_{resist} = 0.94\cdot kip$

$Fkey4_{resist} := \left(\frac{1\cdot ft}{Width_{slab}}\right)\cdot\left(sumH_4 - Fs_{key}\right)$ $Fkey4_{resist} = 1.87\cdot kip$

$Fkey5_{resist} := \left(\frac{1\cdot ft}{Width_{slab}}\right)\cdot\left(sumH_5 - Fs_{key}\right)$ $Fkey5_{resist} = 1.91\cdot kip$

$Fkey6_{resist} := \left(\frac{1\cdot ft}{Width_{slab}}\right)\cdot\left(sumH_6 - Fs_{key}\right)$ $Fkey6_{resist} = 3.18\cdot kip$

$Fkey7_{resist} := \left(\frac{1\cdot ft}{Width_{slab}}\right)\cdot\left(sumH_7 - Fs_{key}\right)$ $Fkey7_{resist} = 1.16\cdot kip$

Key: Moment Per Foot of Width

$Mkey1_{resist} := Fkey1_{resist}\cdot armFs_{key_m}$ $Mkey1_{resist} = 0.71\cdot kip\cdot ft$

$Mkey2_{resist} := Fkey2_{resist}\cdot armFs_{key_m}$ $Mkey2_{resist} = \blacksquare\cdot kip\cdot ft$

$Mkey3_{resist} := Fkey3_{resist}\cdot armFs_{key}$ $Mkey3_{resist} = 0.86\cdot kip\cdot ft$

$Mkey4_{resist} := Fkey4_{resist}\cdot armFs_{key}$ $Mkey4_{resist} = 1.71\cdot kip\cdot ft$

$Mkey5_{resist} := Fkey5_{resist}\cdot armFs_{key}$ $Mkey5_{resist} = 1.75\cdot kip\cdot ft$

$Mkey6_{resist} := Fkey6_{resist}\cdot armFs_{key}$ $Mkey6_{resist} = 2.91\cdot kip\cdot ft$

$Mkey7_{resist} := Fkey7_{resist}\cdot armFs_{key}$ $Mkey7_{resist} = 1.06\cdot kip\cdot ft$

- Horizontal Inertia from key weight.

$Inertia_{str_OBE_key} := k_{h_OBE}\cdot\left(WT_{key}\right)$ $Inertia_{str_OBE_key} = 0.47\cdot kip$

$Inertia_{str_MDE_key} := k_{h_MDE}\cdot\left(WT_{key}\right)$ $Inertia_{str_MDE_key} = 1.09\cdot kip$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability

SLIDING SAFETY FACTORS w/ KEY

$$FS1_{sliding} := \frac{(sumV_{1_2} + WT_{key})\cdot\tan(\phi_{foundation})}{sumH_1 - Fs_{key_m}} \qquad FS1_{sliding} = 6.23 \qquad Check(FS1_{sliding}, 1.3) = \text{"OK"}$$

$$FS2_{sliding} := \frac{(sumV_{1_2} + WT_{key})\cdot\tan(\phi_{foundation})}{Inertia_{str_OBE_key} + sumH_2 - Fs_{key_m}} \qquad FS2_{sliding} = 5.68 \qquad Check(FS2_{sliding}, 1.1) = \text{"OK"}$$

$$FS3_{sliding} := \frac{(sumV_{3_4_5_6} + WT_{key})\cdot\tan(\phi_{foundation})}{sumH_3 - Fs_{key}} \qquad FS3_{sliding} = 3.86 \qquad Check(FS3_{sliding}, 1.7) = \text{"OK"}$$

$$FS4_{sliding} := \frac{(sumV_{3_4_5_6} + WT_{key})\cdot\tan(\phi_{foundation})}{sumH_4 - Fs_{key}} \qquad FS4_{sliding} = 1.95 \qquad Check(FS4_{sliding}, 1.3) = \text{"OK"}$$

$$FS5_{sliding} := \frac{(sumV_{3_4_5_6} + WT_{key})\cdot\tan(\phi_{foundation})}{Inertia_{str_OBE_key} + sumH_5 - Fs_{key}} \qquad FS5_{sliding} = 1.88 \qquad Check(FS5_{sliding}, 1.3) = \text{"OK"}$$

$$FS6_{sliding} := \frac{(sumV_{3_4_5_6} + WT_{key})\cdot\tan(\phi_{foundation})}{Inertia_{str_MDE_key} + sumH_6 - Fs_{key}} \qquad FS6_{sliding} = 1.12 \qquad Check(FS6_{sliding}, 1.1) = \text{"OK"}$$

$$FS7_{sliding} := \frac{(sumV_7 + WT_{key})\cdot\tan(\phi_{foundation})}{sumH_7 - Fs_{key}} \qquad FS7_{sliding} = 2.96 \qquad Check(FS7_{sliding}, 1.1) = \text{"OK"}$$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Stability Summary

SUMMARY

LC 1 - Unusual: Construction
LC 2 - Extreme: Construction with OBE
LC 3 - Usual: Normal Pool
LC 4 - Unusual: Normal Pool with Surcharge
LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
LC 6 - Extreme: Coincident Pool = Normal Pool with MDE
LC 7 - Extreme: Maximum Design Flood = IDF

SLIDING SAFETY FACTORS

$FS1_{sliding} = 6.23$ Check$(FS1_{sliding}, 1.3) = "OK"$

$FS2_{sliding} = 5.68$ Check$(FS2_{sliding}, 1.1) = "OK"$

$FS3_{sliding} = 3.86$ Check$(FS3_{sliding}, 1.7) = "OK"$

$FS4_{sliding} = 1.95$ Check$(FS4_{sliding}, 1.3) = "OK"$

$FS5_{sliding} = 1.88$ Check$(FS5_{sliding}, 1.3) = "OK"$

$FS6_{sliding} = 1.12$ Check$(FS6_{sliding}, 1.1) = "OK"$

$FS7_{sliding} = 2.96$ Check$(FS7_{sliding}, 1.1) = "OK"$

BEARING CAPACITY

$\%Base_Comp1 = 100.0 \cdot \%$ Check$(\%Base_Comp1, 0.75) = "OK"$

$\%Base_Comp2 = 100.0 \cdot \%$ Check$(\%Base_Comp2, 0.01) = "OK"$

$\%Base_Comp3 = 100.0 \cdot \%$ Check$(\%Base_Comp3, 1.0) = "OK"$

$\%Base_Comp4 = 100.0 \cdot \%$ Check$(\%Base_Comp4, 0.75) = "OK"$

$\%Base_Comp5 = 100.0 \cdot \%$ Check$(\%Base_Comp5, 0.75) = "OK"$

$\%Base_Comp6 = 100.0 \cdot \%$ Check$(\%Base_Comp6, 0.01) = "OK"$

$\%Base_Comp7 = 100.0 \cdot \%$ Check$(\%Base_Comp7, 0.01) = "OK"$

$BasePressure1 = 697 \cdot psf$ Check$(BearingCap_{allow}, BasePressure1) = "OK"$

$BasePressure2 = 672 \cdot psf$ Check$(BearingCap_{allow_eq}, BasePressure2) = "OK"$

$BasePressure3 = 515 \cdot psf$ Check$(BearingCap_{allow_eq}, BasePressure3) = "OK"$

$BasePressure4 = 537 \cdot psf$ Check$(BearingCap_{allow}, BasePressure4) = "OK"$

$BasePressure5 = 512 \cdot psf$ Check$(BearingCap_{allow_eq}, BasePressure5) = "OK"$

$BasePressure6 = 579 \cdot psf$ Check$(BearingCap_{allow_eq}, BasePressure6) = "OK"$

$BasePressure7 = 490 \cdot psf$ Check$(BearingCap_{allow}, BasePressure7) = "OK"$

FLOTATION SAFETY FACTORS

$SF_{flotation_NP} = 4.04$ Check$(SF_{flotation_NP}, 1.3) = "OK"$

$SF_{flotation_IDF} = 2.22$ Check$(SF_{flotation_IDF}, 1.1) = "OK"$

CONCRETE USBR TYPE VI BASIN
HOG CHUTE DAM - MESA COUNTY, COLORADO

Near Embankment Toe With Exit Pipe - Design

Ref.:
1) *Rules and Regulations for Dam Safety and Dam Construction*, Colorado Dam Safety, 1 January 2020 (2-CCR 402-1).
2) *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14).*
3) EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, USACOE, 30 Nov. 2016.
4) EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACOE, 1 December 2005.
5) EM 1110-2-2200, *Gravity Dam Design*, USACOE, 30 June 1995.
6) EM 1110-2-6053, *Earthquake Design and Evaluation of Concrete Hydraulic Structures*, USACOE, 1 May 2007.
7) Letter: Geotechnical Design Parameters, Carson Lake, Mesa County, Colorado, from Michael A. Berry, PE, Huddleston-Berry, 27 August 2020 & 15 September 2020 email.
8) Memorandum, Inflow Design Flood (IDF) for Carson Dam, Mesa County, Colorado, Ayres, 16 June 2020.
9) HY-8 Energy Dissipation Report output, Ayres, December 2020.
10) Stability Analysis, Ayres, January 2021.

Design

1) Load Cases Per Ref. 4 for STAAD.Pro model.

LC 1 - Unusual: Construction
LC 2 - Extreme: Construction with OBE
LC 3 - Usual: Normal Pool
LC 4 - Unusual: Normal Pool + Surcharge
LC 5 - Unusual: Coincident Pool = Normal Pool with OBE
LC 6 - Extreme: Coincident Pool = Normal Pool with MDE
LC 7 - Extreme: Maximum Design Flood = IDF

2) Stability per Ref. 10.

3) Assume structure will not deflect sufficiently to cause active earth pressure conditions to develop so use use at-rest earth pressures.
Ref. 7: Site PGA = 0.236. k_h is defined as 1/3 of the site modified PGA so k_h = 0.08 for this site (OBE).
Ref. 6: Section 7-3, MDE = 2/3 MCE = (2/3)(0.277) = 0.185
Zero vertical earthquake acceleration.

4) Per Ref. 9: Original pipe is 30-inch inside diameter. Lining will reduce to 29-inch inside diameter.
IDF outlet: Q = 146 cfs and V = 34.2 ft/s.
Per Ref. 8: Exit IDF water El. = 9842.29 feet.

5) Per past inspections and site topography:
Since structure will be in similar location, use 1.0 foot of water above top of foundation slab, El. 9838.91 feet.

6) Elevations in NAVD88.

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Design.xmcd
Page 2 of 21

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Variables / Model

Variables

Concrete Unit Weight
Ref. 5, Section 3-3b
$\gamma_c := 150 \cdot pcf$

Water Unit Weight
Ref. 5, Section 3-3c
$\gamma_w := 62.5 \cdot pcf$

Soil Moist Unit Weight
Ref. 7
$\gamma_m := 125 \cdot pcf$

Inclination of Soil Surface
(0 if level backfill)
$\beta_h := 5.7 \cdot deg$ (headwall)
$\beta := 0 \cdot deg$ (else)

Soil Saturated Unit Weight
Ref. 7
$\gamma_s := 130 \cdot pcf$

Horizontal
Acceleration (in g's)
Design Note 3
$k_{h_OBE} := 0.08$
$k_{h_MDE} := 0.185$

Backfill Friction Angle
Ref. 7
$\phi := 32 \cdot deg$

Concrete Compressive Strength:
$f'_c := 4000 \cdot psi$

Uniform Surcharge
$q_{surcharge} := 200 \cdot psf$

Concrete Modulus:
Ref. 2, 19.2.2
$E_c := 57000 \cdot \sqrt{f'_c \cdot psi}$
$E_c = 519120 \cdot ksf$

Thicknesses
$t_{slab} := 1 \cdot ft + 6 \cdot in$
$t_{baffle} := 1 \cdot ft + 0 \cdot in$
$t_{walls} := 1 \cdot ft + 6 \cdot in$
$t_{endsill} := 1 \cdot ft + 6 \cdot in$

Elevations
$EL_{w_NP} := 9838.91 \cdot ft$
$EL_{Tslab} := 9837.91 \cdot ft$
$EL_{w_IDF} := 9842.29 \cdot ft$
$EL_{Bslab} := EL_{Tslab} - t_{slab}$ $EL_{Bslab} = 9836.41\ ft$

VERTICAL FORCES

-- Structure plates are 1.0' square.
-- Concrete: Let program compute based on 150 pcf.
-- Normal inside water: N/A
-- IDF inside water: Place water on entire inside.
-- Uplift: Place on entire bottom for normal and IDF.

IDF Inside Water

$p_{water_IDF} := \gamma_w \cdot (EL_{w_IDF} - EL_{Tslab})$ $p_{water_IDF} = 0.274 \cdot ksf$

Uniform Uplift - Normal

$p_{uplift_NP} := \gamma_w \cdot (EL_{w_NP} - EL_{Bslab})$ $p_{uplift_NP} = 0.156 \cdot ksf$

Uniform Uplift - IDF

$p_{uplift_IDF} := \gamma_w \cdot (EL_{w_IDF} - EL_{Bslab})$ $p_{uplift_IDF} = 0.368 \cdot ksf$

Ayres Associates Inc
Eau Claire, WI

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Model

HORIZONTAL FORCES

-- Structure plates are 1.0' square
-- Soil and surcharge: Place to top of walls (conservative).
-- Normal water: Do not include on inside.
-- IDF water: Do not include since on all sides so cancel each other out.
-- IDF water jet: Pipe is 2.5' diameter and baffle is 6.0' away. Assume full load acts over the modeled 4.0' effective baffle height and over a reduced width of 8.0' (about a pipe diameter on each side).
-- Earthquake: For walls, compute inertia based on wall concrete thickness.

Soil At-Rest Earth Pressure Coefficient:

$K_{o_h} := (1 - \sin(\phi)) \cdot (1 + \sin(\beta_h))$ $K_{o_h} = 0.517$

$K_o := (1 - \sin(\phi)) \cdot (1 + \sin(\beta))$ $K_o = 0.470$

Moist Soil

Pressures:

$\sigma_{1m_h} := K_{o_h} \cdot \gamma_m \cdot (1 \cdot ft)$	$\sigma_{1m_h} = 0.065 \cdot ksf$	$\sigma_{1m} := K_o \cdot \gamma_m \cdot (1 \cdot ft)$	$\sigma_{1m} = 0.059 \cdot ksf$
$\sigma_{2m_h} := K_{o_h} \cdot \gamma_m \cdot (2 \cdot ft)$	$\sigma_{2m_h} = 0.129 \cdot ksf$	$\sigma_{2m} := K_o \cdot \gamma_m \cdot (2 \cdot ft)$	$\sigma_{2m} = 0.118 \cdot ksf$
$\sigma_{3m_h} := K_{o_h} \cdot \gamma_m \cdot (3 \cdot ft)$	$\sigma_{3m_h} = 0.194 \cdot ksf$	$\sigma_{3m} := K_o \cdot \gamma_m \cdot (3 \cdot ft)$	$\sigma_{3m} = 0.176 \cdot ksf$
$\sigma_{4m_h} := K_{o_h} \cdot \gamma_m \cdot (4 \cdot ft)$	$\sigma_{4m_h} = 0.258 \cdot ksf$	$\sigma_{4m} := K_o \cdot \gamma_m \cdot (4 \cdot ft)$	$\sigma_{4m} = 0.235 \cdot ksf$
$\sigma_{5m_h} := K_{o_h} \cdot \gamma_m \cdot (5 \cdot ft)$	$\sigma_{5m_h} = 0.323 \cdot ksf$	$\sigma_{5m} := K_o \cdot \gamma_m \cdot (5 \cdot ft)$	$\sigma_{5m} = 0.294 \cdot ksf$
$\sigma_{6m_h} := K_{o_h} \cdot \gamma_m \cdot (6 \cdot ft)$	$\sigma_{6m_h} = 0.388 \cdot ksf$	$\sigma_{6m} := K_o \cdot \gamma_m \cdot (6 \cdot ft)$	$\sigma_{6m} = 0.353 \cdot ksf$
$\sigma_{7m_h} := K_{o_h} \cdot \gamma_m \cdot (7 \cdot ft)$	$\sigma_{7m_h} = 0.452 \cdot ksf$	$\sigma_{7m} := K_o \cdot \gamma_m \cdot (7 \cdot ft)$	$\sigma_{7m} = 0.411 \cdot ksf$
$\sigma_{8m_h} := K_{o_h} \cdot \gamma_m \cdot (8 \cdot ft)$	$\sigma_{8m_h} = 0.517 \cdot ksf$	$\sigma_{8m} := K_o \cdot \gamma_m \cdot (8 \cdot ft)$	$\sigma_{8m} = 0.470 \cdot ksf$
$\sigma_{9m_h} := K_{o_h} \cdot \gamma_m \cdot (9 \cdot ft)$	$\sigma_{9m_h} = 0.581 \cdot ksf$	$\sigma_{9m} := K_o \cdot \gamma_m \cdot (9 \cdot ft)$	$\sigma_{9m} = 0.529 \cdot ksf$
$\sigma_{10m_h} := K_{o_h} \cdot \gamma_m \cdot (10 \cdot ft)$	$\sigma_{10m_h} = 0.646 \cdot ksf$	$\sigma_{10m} := K_o \cdot \gamma_m \cdot (10 \cdot ft)$	$\sigma_{10m} = 0.588 \cdot ksf$

Model

Saturated Soil

Pressures: (NP)

$\sigma_{1_NP_h} := K_{o_h} \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_NP_h} = 0.065 \cdot ksf$ $\sigma_{1_NP} := K_o \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_NP} = 0.059 \cdot ksf$

$\sigma_{2_NP_h} := K_{o_h} \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_NP_h} = 0.129 \cdot ksf$ $\sigma_{2_NP} := K_o \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_NP} = 0.118 \cdot ksf$

$\sigma_{3_NP_h} := K_{o_h} \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_NP_h} = 0.194 \cdot ksf$ $\sigma_{3_NP} := K_o \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_NP} = 0.176 \cdot ksf$

$\sigma_{4_NP_h} := K_{o_h} \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_NP_h} = 0.258 \cdot ksf$ $\sigma_{4_NP} := K_o \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_NP} = 0.235 \cdot ksf$

$\sigma_{5_NP_h} := K_{o_h} \cdot \gamma_m \cdot (5 \cdot ft)$ $\sigma_{5_NP_h} = 0.323 \cdot ksf$ $\sigma_{5_NP} := K_o \cdot \gamma_m \cdot (5 \cdot ft)$ $\sigma_{5_NP} = 0.294 \cdot ksf$

$\sigma_{6_NP_h} := K_{o_h} \cdot \gamma_m \cdot (6 \cdot ft)$ $\sigma_{6_NP_h} = 0.388 \cdot ksf$ $\sigma_{6_NP} := K_o \cdot \gamma_m \cdot (6 \cdot ft)$ $\sigma_{6_NP} = 0.353 \cdot ksf$

$\sigma_{7_NP_h} := K_{o_h} \cdot \gamma_m \cdot (7 \cdot ft)$ $\sigma_{7_NP_h} = 0.452 \cdot ksf$ $\sigma_{7_NP} := K_o \cdot \gamma_m \cdot (7 \cdot ft)$ $\sigma_{7_NP} = 0.411 \cdot ksf$

$\sigma_{8_NP_h} := \sigma_{7_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$ $\sigma_{8_NP} := \sigma_{7_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$

$\sigma_{8_NP_h} = 0.487 \cdot ksf$ $\sigma_{8_NP} = 0.443 \cdot ksf$

$\sigma_{9_NP_h} := \sigma_{7_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$ $\sigma_{9_NP} := \sigma_{7_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$

$\sigma_{9_NP_h} = 0.522 \cdot ksf$ $\sigma_{9_NP} = 0.475 \cdot ksf$

$\sigma_{10_NP_h} := \sigma_{7_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$ $\sigma_{10_NP} := \sigma_{7_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$

$\sigma_{10_NP_h} = 0.557 \cdot ksf$ $\sigma_{10_NP} = 0.507 \cdot ksf$

Model

Pressures: (IDF)

$\sigma_{1_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_IDF_h} = 0.065 \cdot ksf$

$\sigma_{2_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_IDF_h} = 0.129 \cdot ksf$

$\sigma_{3_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_IDF_h} = 0.194 \cdot ksf$

$\sigma_{4_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_IDF_h} = 0.258 \cdot ksf$

$\sigma_{5_IDF_h} := \sigma_{4_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$

$\sigma_{5_IDF_h} = 0.293 \cdot ksf$

$\sigma_{6_IDF_h} := \sigma_{4_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$

$\sigma_{6_IDF_h} = 0.328 \cdot ksf$

$\sigma_{7_IDF_h} := \sigma_{4_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$

$\sigma_{7_IDF_h} = 0.363 \cdot ksf$

$\sigma_{8_IDF_h} := \sigma_{4_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (4 \cdot ft)$

$\sigma_{8_IDF_h} = 0.398 \cdot ksf$

$\sigma_{9_IDF_h} := \sigma_{4_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (5 \cdot ft)$

$\sigma_{9_IDF_h} = 0.433 \cdot ksf$

$\sigma_{10_IDF_h} := \sigma_{4_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (6 \cdot ft)$

$\sigma_{10_IDF_h} = 0.468 \cdot ksf$

$\sigma_{1_IDF} := K_o \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_IDF} = 0.059 \cdot ksf$

$\sigma_{2_IDF} := K_o \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_IDF} = 0.118 \cdot ksf$

$\sigma_{3_IDF} := K_o \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_IDF} = 0.176 \cdot ksf$

$\sigma_{4_IDF} := K_o \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_IDF} = 0.235 \cdot ksf$

$\sigma_{5_IDF} := \sigma_{4_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$

$\sigma_{5_IDF} = 0.267 \cdot ksf$

$\sigma_{6_IDF} := \sigma_{4_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$

$\sigma_{6_IDF} = 0.299 \cdot ksf$

$\sigma_{7_IDF} := \sigma_{4_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$

$\sigma_{7_IDF} = 0.330 \cdot ksf$

$\sigma_{8_IDF} := \sigma_{4_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (4 \cdot ft)$

$\sigma_{8_IDF} = 0.362 \cdot ksf$

$\sigma_{9_IDF} := \sigma_{4_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (5 \cdot ft)$

$\sigma_{9_IDF} = 0.394 \cdot ksf$

$\sigma_{10_IDF} := \sigma_{4_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (6 \cdot ft)$

$\sigma_{10_IDF} = 0.425 \cdot ksf$

Model

Saturated Soil

Pressures: (NP)

$\sigma_{1_NP_h} := K_{o_h} \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_NP_h} = 0.065 \cdot ksf$ $\sigma_{1_NP} := K_o \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_NP} = 0.059 \cdot ksf$

$\sigma_{2_NP_h} := K_{o_h} \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_NP_h} = 0.129 \cdot ksf$ $\sigma_{2_NP} := K_o \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_NP} = 0.118 \cdot ksf$

$\sigma_{3_NP_h} := K_{o_h} \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_NP_h} = 0.194 \cdot ksf$ $\sigma_{3_NP} := K_o \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_NP} = 0.176 \cdot ksf$

$\sigma_{4_NP_h} := K_{o_h} \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_NP_h} = 0.258 \cdot ksf$ $\sigma_{4_NP} := K_o \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_NP} = 0.235 \cdot ksf$

$\sigma_{5_NP_h} := K_{o_h} \cdot \gamma_m \cdot (5 \cdot ft)$ $\sigma_{5_NP_h} = 0.323 \cdot ksf$ $\sigma_{5_NP} := K_o \cdot \gamma_m \cdot (5 \cdot ft)$ $\sigma_{5_NP} = 0.294 \cdot ksf$

$\sigma_{6_NP_h} := K_{o_h} \cdot \gamma_m \cdot (6 \cdot ft)$ $\sigma_{6_NP_h} = 0.388 \cdot ksf$ $\sigma_{6_NP} := K_o \cdot \gamma_m \cdot (6 \cdot ft)$ $\sigma_{6_NP} = 0.353 \cdot ksf$

$\sigma_{7_NP_h} := \sigma_{6_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$
$\sigma_{7_NP_h} = 0.422 \cdot ksf$

$\sigma_{7_NP} := \sigma_{6_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$
$\sigma_{7_NP} = 0.384 \cdot ksf$

$\sigma_{8_NP_h} := \sigma_{6_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$
$\sigma_{8_NP_h} = 0.457 \cdot ksf$

$\sigma_{8_NP} := \sigma_{6_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$
$\sigma_{8_NP} = 0.416 \cdot ksf$

$\sigma_{9_NP_h} := \sigma_{6_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$
$\sigma_{9_NP_h} = 0.492 \cdot ksf$

$\sigma_{9_NP} := \sigma_{6_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$
$\sigma_{9_NP} = 0.448 \cdot ksf$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Design.xmcd
Page 7 of 21

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Model

Pressures: (IDF)

$\sigma_{1_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_IDF_h} = 0.065 \cdot ksf$

$\sigma_{2_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_IDF_h} = 0.129 \cdot ksf$

$\sigma_{3_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_IDF_h} = 0.194 \cdot ksf$

$\sigma_{4_IDF_h} := \sigma_{3_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$

$\sigma_{4_IDF_h} = 0.229 \cdot ksf$

$\sigma_{5_IDF_h} := \sigma_{3_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$

$\sigma_{5_IDF_h} = 0.264 \cdot ksf$

$\sigma_{6_IDF_h} := \sigma_{3_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$

$\sigma_{6_IDF_h} = 0.298 \cdot ksf$

$\sigma_{7_IDF_h} := \sigma_{3_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (4 \cdot ft)$

$\sigma_{7_IDF_h} = 0.333 \cdot ksf$

$\sigma_{8_IDF_h} := \sigma_{3_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (5 \cdot ft)$

$\sigma_{8_IDF_h} = 0.368 \cdot ksf$

$\sigma_{9_IDF_h} := \sigma_{3_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (6 \cdot ft)$

$\sigma_{9_IDF_h} = 0.403 \cdot ksf$

$\sigma_{1_IDF} := K_o \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_IDF} = 0.059 \cdot ksf$

$\sigma_{2_IDF} := K_o \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_IDF} = 0.118 \cdot ksf$

$\sigma_{3_IDF} := K_o \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_IDF} = 0.176 \cdot ksf$

$\sigma_{4_IDF} := \sigma_{3_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$

$\sigma_{4_IDF} = 0.208 \cdot ksf$

$\sigma_{5_IDF} := \sigma_{3_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$

$\sigma_{5_IDF} = 0.240 \cdot ksf$

$\sigma_{6_IDF} := \sigma_{3_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$

$\sigma_{6_IDF} = 0.271 \cdot ksf$

$\sigma_{7_IDF} := \sigma_{3_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (4 \cdot ft)$

$\sigma_{7_IDF} = 0.303 \cdot ksf$

$\sigma_{8_IDF} := \sigma_{3_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (5 \cdot ft)$

$\sigma_{8_IDF} = 0.335 \cdot ksf$

$\sigma_{9_IDF} := \sigma_{3_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (6 \cdot ft)$

$\sigma_{9_IDF} = 0.367 \cdot ksf$

Ayres Associates Inc
Eau Claire, WI

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Model

Saturated Soil

Pressures: (NP)

$\sigma_{1_NP_h} := K_{o_h}\cdot\gamma_m\cdot(1\cdot ft)$ $\sigma_{1_NP_h} = 0.065\cdot ksf$

$\sigma_{2_NP_h} := K_{o_h}\cdot\gamma_m\cdot(2\cdot ft)$ $\sigma_{2_NP_h} = 0.129\cdot ksf$

$\sigma_{3_NP_h} := K_{o_h}\cdot\gamma_m\cdot(3\cdot ft)$ $\sigma_{3_NP_h} = 0.194\cdot ksf$

$\sigma_{4_NP_h} := K_{o_h}\cdot\gamma_m\cdot(4\cdot ft)$ $\sigma_{4_NP_h} = 0.258\cdot ksf$

$\sigma_{5_NP_h} := K_{o_h}\cdot\gamma_m\cdot(5\cdot ft)$ $\sigma_{5_NP_h} = 0.323\cdot ksf$

$\sigma_{6_NP_h} := \sigma_{5_NP_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(1\cdot ft)$
$\sigma_{6_NP_h} = 0.358\cdot ksf$

$\sigma_{7_NP_h} := \sigma_{5_NP_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(2\cdot ft)$
$\sigma_{7_NP_h} = 0.393\cdot ksf$

$\sigma_{8_NP_h} := \sigma_{5_NP_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(3\cdot ft)$
$\sigma_{8_NP_h} = 0.428\cdot ksf$

$\sigma_{1_NP} := K_o\cdot\gamma_m\cdot(1\cdot ft)$ $\sigma_{1_NP} = 0.059\cdot ksf$

$\sigma_{2_NP} := K_o\cdot\gamma_m\cdot(2\cdot ft)$ $\sigma_{2_NP} = 0.118\cdot ksf$

$\sigma_{3_NP} := K_o\cdot\gamma_m\cdot(3\cdot ft)$ $\sigma_{3_NP} = 0.176\cdot ksf$

$\sigma_{4_NP} := K_o\cdot\gamma_m\cdot(4\cdot ft)$ $\sigma_{4_NP} = 0.235\cdot ksf$

$\sigma_{5_NP} := K_o\cdot\gamma_m\cdot(5\cdot ft)$ $\sigma_{5_NP} = 0.294\cdot ksf$

$\sigma_{6_NP} := \sigma_{5_NP} + K_o\cdot(\gamma_s - \gamma_w)\cdot(1\cdot ft)$
$\sigma_{6_NP} = 0.326\cdot ksf$

$\sigma_{7_NP} := \sigma_{5_NP} + K_o\cdot(\gamma_s - \gamma_w)\cdot(2\cdot ft)$
$\sigma_{7_NP} = 0.357\cdot ksf$

$\sigma_{8_NP} := \sigma_{5_NP} + K_o\cdot(\gamma_s - \gamma_w)\cdot(3\cdot ft)$
$\sigma_{8_NP} = 0.389\cdot ksf$

Pressures: (IDF)

$\sigma_{1_IDF_h} := K_{o_h}\cdot\gamma_m\cdot(1\cdot ft)$ $\sigma_{1_IDF_h} = 0.065\cdot ksf$

$\sigma_{2_IDF_h} := K_{o_h}\cdot\gamma_m\cdot(2\cdot ft)$ $\sigma_{2_IDF_h} = 0.129\cdot ksf$

$\sigma_{3_IDF_h} := \sigma_{2_IDF_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(1\cdot ft)$
$\sigma_{3_IDF_h} = 0.164\cdot ksf$

$\sigma_{4_IDF_h} := \sigma_{2_IDF_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(2\cdot ft)$
$\sigma_{4_IDF_h} = 0.199\cdot ksf$

$\sigma_{5_IDF_h} := \sigma_{2_IDF_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(3\cdot ft)$
$\sigma_{5_IDF_h} = 0.234\cdot ksf$

$\sigma_{6_IDF_h} := \sigma_{2_IDF_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(4\cdot ft)$
$\sigma_{6_IDF_h} = 0.269\cdot ksf$

$\sigma_{7_IDF_h} := \sigma_{2_IDF_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(5\cdot ft)$
$\sigma_{7_IDF_h} = 0.304\cdot ksf$

$\sigma_{8_IDF_h} := \sigma_{2_IDF_h} + K_{o_h}\cdot(\gamma_s - \gamma_w)\cdot(6\cdot ft)$
$\sigma_{8_IDF_h} = 0.338\cdot ksf$

$\sigma_{1_IDF} := K_o\cdot\gamma_m\cdot(1\cdot ft)$ $\sigma_{1_IDF} = 0.059\cdot ksf$

$\sigma_{2_IDF} := K_o\cdot\gamma_m\cdot(2\cdot ft)$ $\sigma_{2_IDF} = 0.118\cdot ksf$

$\sigma_{3_IDF} := \sigma_{2_IDF} + K_o\cdot(\gamma_s - \gamma_w)\cdot(1\cdot ft)$
$\sigma_{3_IDF} = 0.149\cdot ksf$

$\sigma_{4_IDF} := \sigma_{2_IDF} + K_o\cdot(\gamma_s - \gamma_w)\cdot(2\cdot ft)$
$\sigma_{4_IDF} = 0.181\cdot ksf$

$\sigma_{5_IDF} := \sigma_{2_IDF} + K_o\cdot(\gamma_s - \gamma_w)\cdot(3\cdot ft)$
$\sigma_{5_IDF} = 0.213\cdot ksf$

$\sigma_{6_IDF} := \sigma_{2_IDF} + K_o\cdot(\gamma_s - \gamma_w)\cdot(4\cdot ft)$
$\sigma_{6_IDF} = 0.244\cdot ksf$

$\sigma_{7_IDF} := \sigma_{2_IDF} + K_o\cdot(\gamma_s - \gamma_w)\cdot(5\cdot ft)$
$\sigma_{7_IDF} = 0.276\cdot ksf$

$\sigma_{8_IDF} := \sigma_{2_IDF} + K_o\cdot(\gamma_s - \gamma_w)\cdot(6\cdot ft)$
$\sigma_{8_IDF} = 0.308\cdot ksf$

Model

Saturated Soil

Pressures: (NP)

$\sigma_{1_NP_h} := K_{o_h} \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_NP_h} = 0.065 \cdot ksf$ $\sigma_{1_NP} := K_o \cdot \gamma_m \cdot (1 \cdot ft)$ $\sigma_{1_NP} = 0.059 \cdot ksf$

$\sigma_{2_NP_h} := K_{o_h} \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_NP_h} = 0.129 \cdot ksf$ $\sigma_{2_NP} := K_o \cdot \gamma_m \cdot (2 \cdot ft)$ $\sigma_{2_NP} = 0.118 \cdot ksf$

$\sigma_{3_NP_h} := K_{o_h} \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_NP_h} = 0.194 \cdot ksf$ $\sigma_{3_NP} := K_o \cdot \gamma_m \cdot (3 \cdot ft)$ $\sigma_{3_NP} = 0.176 \cdot ksf$

$\sigma_{4_NP_h} := K_{o_h} \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_NP_h} = 0.258 \cdot ksf$ $\sigma_{4_NP} := K_o \cdot \gamma_m \cdot (4 \cdot ft)$ $\sigma_{4_NP} = 0.235 \cdot ksf$

$\sigma_{5_NP_h} := \sigma_{4_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$
$\sigma_{5_NP_h} = 0.293 \cdot ksf$

$\sigma_{5_NP} := \sigma_{4_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$
$\sigma_{5_NP} = 0.267 \cdot ksf$

$\sigma_{6_NP_h} := \sigma_{4_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$
$\sigma_{6_NP_h} = 0.328 \cdot ksf$

$\sigma_{6_NP} := \sigma_{4_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$
$\sigma_{6_NP} = 0.299 \cdot ksf$

$\sigma_{7_NP_h} := \sigma_{4_NP_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$
$\sigma_{7_NP_h} = 0.363 \cdot ksf$

$\sigma_{7_NP} := \sigma_{4_NP} + K_o \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$
$\sigma_{7_NP} = 0.330 \cdot ksf$

Pressures: (IDF)

$\sigma_{1_IDF_h} := K_{o_h} \cdot \gamma_m \cdot (1 \cdot ft)$
$\sigma_{1_IDF_h} = 0.065 \cdot ksf$

$\sigma_{1_IDF} := K_o \cdot \gamma_m \cdot (1 \cdot ft)$
$\sigma_{1_IDF} = 0.059 \cdot ksf$

$\sigma_{2_IDF_h} := \sigma_{1_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$
$\sigma_{2_IDF_h} = 0.099 \cdot ksf$

$\sigma_{2_IDF} := \sigma_{1_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (1 \cdot ft)$
$\sigma_{2_IDF} = 0.090 \cdot ksf$

$\sigma_{3_IDF_h} := \sigma_{1_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$
$\sigma_{3_IDF_h} = 0.134 \cdot ksf$

$\sigma_{3_IDF} := \sigma_{1_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (2 \cdot ft)$
$\sigma_{3_IDF} = 0.122 \cdot ksf$

$\sigma_{4_IDF_h} := \sigma_{1_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$
$\sigma_{4_IDF_h} = 0.169 \cdot ksf$

$\sigma_{4_IDF} := \sigma_{1_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (3 \cdot ft)$
$\sigma_{4_IDF} = 0.154 \cdot ksf$

$\sigma_{5_IDF_h} := \sigma_{1_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (4 \cdot ft)$
$\sigma_{5_IDF_h} = 0.204 \cdot ksf$

$\sigma_{5_IDF} := \sigma_{1_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (4 \cdot ft)$
$\sigma_{5_IDF} = 0.186 \cdot ksf$

$\sigma_{6_IDF_h} := \sigma_{1_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (5 \cdot ft)$
$\sigma_{6_IDF_h} = 0.239 \cdot ksf$

$\sigma_{6_IDF} := \sigma_{1_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (5 \cdot ft)$
$\sigma_{6_IDF} = 0.217 \cdot ksf$

$\sigma_{7_IDF_h} := \sigma_{1_IDF_h} + K_{o_h} \cdot (\gamma_s - \gamma_w) \cdot (6 \cdot ft)$
$\sigma_{7_IDF_h} = 0.274 \cdot ksf$

$\sigma_{7_IDF} := \sigma_{1_IDF} + K_o \cdot (\gamma_s - \gamma_w) \cdot (6 \cdot ft)$
$\sigma_{7_IDF} = 0.249 \cdot ksf$

Ayres Associates Inc
Eau Claire, WI

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Model

Surcharge Uniform Pressure

$p_{surcharge_h} := K_{o_h} \cdot q_{surcharge}$ $\quad p_{surcharge_h} = 0.103 \cdot ksf$

$p_{surcharge} := K_{o} \cdot q_{surcharge}$ $\quad p_{surcharge} = 0.094 \cdot ksf$

Normal Water

- outside only

$\sigma_{1_NP_water} := \gamma_w \cdot (1 \cdot ft)$ $\quad \sigma_{1_NP_water} = 0.063 \cdot ksf$

$\sigma_{2_NP_water} := \gamma_w \cdot (2 \cdot ft)$ $\quad \sigma_{2_NP_water} = 0.125 \cdot ksf$

$\sigma_{3_NP_water} := \gamma_w \cdot (3 \cdot ft)$ $\quad \sigma_{3_NP_water} = 0.188 \cdot ksf$

Pipe Water Jet IDF only

$F = \rho A v^2$ where $\rho_{water} = \gamma_w / g = 1.94$ slug/cf and $\quad g = 32.17 \cdot \frac{ft}{s^2}$

Pipe Diameter: $D_{pipe} := 29 \cdot in$

Velocity: $v := 34.2 \frac{ft}{s}$

Force: $Fx_{jet} := \frac{\gamma_w}{g} \cdot \left[\left(\frac{\pi}{4} \right) \cdot D_{pipe}^{\,2} \right] \cdot v^2$ $\quad Fx_{jet} = 10.42 \cdot kip$

Area on Baffle: $A_{jet} := (4.0 \cdot ft) \cdot (8.0 \cdot ft)$ $\quad A_{jet} = 32.00 \cdot ft^2$

Uniform Pressure: $p_{PipeJet_IDF} := \frac{Fx_{jet}}{A_{jet}}$ $\quad p_{PipeJet_IDF} = 0.326 \cdot ksf$

Earthquake

- Headwall and abutment concrete and baffle concrete horizontal uniform pressures.

$p_{concrete_OBE} := k_{h_OBE} \cdot \gamma_c \cdot t_{walls}$ $\quad p_{concrete_OBE} = 0.018 \cdot ksf$

$p_{concrete_MDE} := k_{h_MDE} \cdot \gamma_c \cdot t_{walls}$ $\quad p_{concrete_MDE} = 0.042 \cdot ksf$

$p_{concrete_OBE_b} := k_{h_OBE} \cdot \gamma_c \cdot t_{baffle}$ $\quad p_{concrete_OBE_b} = 0.012 \cdot ksf$

$p_{concrete_MDE_b} := k_{h_MDE} \cdot \gamma_c \cdot t_{baffle}$ $\quad p_{concrete_MDE_b} = 0.028 \cdot ksf$

Model Concrete Load Cases

CONCRETE STRUCTURE

Load Descriptions and Factors per Ref. 3, Section 3.3

Lp = permanent loads
Lt = temporary (intermittent static) loads -> i.e. floods
Ld = dynamic (impulse) loads -> i.e. earthquake

Lp = 1] concrete (D), soil (EH)
2] horizontal normal water (Hs)
3] normal uplift (Hs)

Lt = 1] vertical and horizontal flood (Hs)
2] flood uplift (Hs)
3] operating equipment (Q)

Ld = horizontal earthquake loads (EQ)

Load Cases (see previous) and Load Factors

LC 1 - Unusual: Construction
Equ. 3-3: U = 1.6 [D + EH + Q]

LC 2 - Extreme: Construction with OBE
Equ. 3-8 and Table 3-1:
U = (1.2 add to effect or 0.9 subtract) [D] + (1.35 driving for At-Rest) [EH] + 1.25 EQ

LC 3 - Usual: Normal Pool
Equ. 3-2: U = 2.2 [D + EH + Hs]

LC 4 - Unusual: Normal Pool + Surcharge
Equ. 3-3: U = 1.6 [D + EH + Hs + Q]

LC 5, 6, 7 - Unusual: Coincident Pool = Normal Pool with OBE
Equ. 3-7: U = 1.5 [D + EH + Hs + EQ]

LC 8, 9, 10 - Extreme: Coincident Pool = Normal Pool with MDE
Equ. 3-8 and Table 3-1:
U = (1.2 add to effect or 0.9 subtract) [D] + (1.35 driving for At-Rest) [EH] + 1.0 [Hs] + 1.25 EQ

LC 11 - Extreme: Maximum Design Flood = IDF
Equ. 3-6 and Table 3-1:
U = (1.2 add to effect or 0.9 subtract) [D] + (1.35 driving for At-Rest) [EH] + 1.3 [Hs]

where:

D = Dead Load
EH = Moist or Effective Lateral Earth Pressures
EQ = Earthquake
Hs = Hydrostatic (Lateral and Vertical)
Q = Horizontal Operating Equipment (Live Loads)

Model Concrete Load Cases

Staad.Pro Load Cases

1 Concrete Dead Load
2 IDF Water Weight
3 Uplift - NP
4 Uplift - IDF
5 Moist Soil
6 Saturated Soil - NP
7 Saturated Soil - IDF
8 Surcharge
9 Lateral Water - NP
10 Pipe Jet - IDF
11, 12, 13 Earthquake - OBE
14, 15, 16 Earthquake - MDE

Staad.Pro Load Combinations

[Format: 'Load Case' 'Load Factor']

LC 101 -> 1 1.6 5 1.6 8 1.6
LC 102 -> 1 1.2 5 1.35 11 1.25
LC 103 -> 1 2.2 3 2.2 6 2.2 9 2.2
LC 104 -> 1 1.6 3 1.6 6 1.6 8 1.6 9 1.6
LC 105 -> 1 1.5 3 1.5 6 1.5 9 1.5 11 1.5
LC 106 -> 1 1.5 3 1.5 6 1.5 9 1.5 12 1.5
LC 107 -> 1 1.5 3 1.5 6 1.5 9 1.5 13 1.5
LC 108 -> 1 1.2 3 1.0 6 1.35 9 1.0 14 1.25
LC 109 -> 1 1.2 3 1.0 6 1.35 9 1.0 15 1.25
LC 110 -> 1 1.2 3 1.0 6 1.35 9 1.0 16 1.25
LC 111 -> 1 1.2 2 0.9 4 1.3 7 1.35 10 1.3

Foundation

Ref.: *Correlation between Soil Bearing Capacity and Modulus of Subgrade Reaction*, Apurba Tribedi, Structure Magazine, December 2013.

Allowable Bearing Capacity
Ref. 7

$BearingCap_{allow} := 2000 \cdot psf$ and $BearingCap_{allow_eq} := 2666 \cdot psf$

Modulus of Subgrade Reaction, Eqn. 7: $Ks = \frac{I \cdot q_a}{\delta}$

Deflection Limit: $\delta := \frac{1}{2} \cdot in$

Where I is Safey Factor: $SF := 3$

$Ks := \frac{SF \cdot BearingCap_{allow}}{\delta}$ $Ks = 83 \cdot \frac{lbf}{in^3}$ (use for all load cases)

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Design.xmcd
Page 13 of 21

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

CONCRETE REINFORCEMENT

Structural

Summary

The stability was checked in a separate computation.
This document first presented the input loads for the simplified finite element model using STAAD.Pro.
The output from the model will be used for the structural design.

Design sections with the same thickness for the maximum shears and moments (use absolute values of Mx/My + Mxy).
Therefore, check 18" Section 1 and 12" Section 2.

Use one mat of reinforcement in each face and in each direction. Develop hooks, etc. as required.

Unit Width: $b := 12 \cdot in$

Concrete Cover: (Ref. 3, Table 2-1)
$c1 := 3 \cdot in$
$c2 := 2 \cdot in$

Resistance Factors (Ref. 2, Table 21.2.1)
$\phi V := 0.75$
$\phi M := 0.90$

Total Section Thicknesses:
$t1 := 18 \cdot in$
$t2 := 12 \cdot in$

Concrete Compressive Strength: $f'_c = 4000\,psi$

Reinforcement Yield Strength: $f_y := 60000 \cdot psi$ [ASTM A615, Grade 60]

(Ref. 2, 22.2.2.4.3)

$$\beta_1 := if\left[f'_c > 4000 \cdot psi, if\left[\left(0.85 - 0.05 \cdot \frac{f'_c - 4000 \cdot psi}{1000 \cdot psi}\right) > 0.65, \left(0.85 - 0.05 \cdot \frac{f'_c - 4000 \cdot psi}{1000 \cdot psi}\right), 0.65\right], 0.85\right] \qquad \beta_1 = 0.850$$

--> Forces From STAAD.Pro computer model.

Maximum Factored Shear:
1 Headwall, abutments, slab
[Plates 10101 to 41418]
2 Baffle
[Plates 50501 to 50814]

$V1_{up} := 5.97 \cdot ksf$ [LC 103, Plate 40118, Qy]

$V2_{up} := 2.94 \cdot ksf$ [LC 111, Plate 50501, Qx]

$V1_u := V1_{up} \cdot b \cdot t1 \qquad V1_u = 8.95 \cdot kip$

$V2_u := V2_{up} \cdot b \cdot t2 \qquad V2_u = 2.94 \cdot kip$

Maximum Factored Moment:
1 Headwall, abutments, slab
[Plates 10101 to 41418]
2 Baffle
[Plates 50501 to 50814]

$M1_u := (10.692 + 1.195) \cdot kip \cdot ft \qquad M1_u = 11.887 \cdot kip \cdot ft$ [LC 103, Plate 10118, My + Mxy]

$M2_u := (6.429 + 0.059) \cdot kip \cdot ft \qquad M2_u = 6.488 \cdot kip \cdot ft$ [LC 111, Plate 50814, Mx + Mxy]

Define Check Function: $Check(x, y) := if(x \geq y, "OK", "NO GOOD")$

Structural

TEMPERATURE STEEL

COE (Ref. 3, Section 2.6): 12" center-to-center spacing of both primary and secondary bars.
COE (Ref. 3, Sections 2.9.1 and 2.9.2): 0.003 for Grade 60, with maximum area equivalent to #9 @ 12" in each face

Section 1 Both Directions

$BarNo_{t_COE} := 5$ $Spacing_{t_COE} := 11\cdot in$ $B := BarNo_{t_COE}$

$A_{temp_COE} := \left(\frac{0.003\cdot b\cdot t1}{2}\right)$ $A_{temp_COE} = 0.324\cdot in^2$

$db_t := if\left(B = 4, \frac{4}{8}\cdot in, if\left(B = 5, \frac{5}{8}\cdot in, if\left(B = 6, \frac{6}{8}\cdot in, if\left(B = 7, \frac{7}{8}\cdot in, if\left(B = 8, \frac{8}{8}\cdot in, if(B = 9, 1.128\cdot in, 0\cdot in)\right)\right)\right)\right)\right)$

$A_{sprov_bt} := \frac{\frac{\pi\cdot db_t^2}{4}\cdot(12\cdot in)}{Spacing_{t_COE}}$ $A_{sprov_bt} = 0.335\cdot in^2$ / foot

$Check\left[A_{sprov_bt}, \left(A_{temp_COE}\right)\right] = "OK"$

Section 2 Both Directions

$BarNo_{t_COE} := 4$ $Spacing_{t_COE} := 10\cdot in$ $B := BarNo_{t_COE}$

$A_{temp_COE} := \left(\frac{0.003\cdot b\cdot t2}{2}\right)$ $A_{temp_COE} = 0.216\cdot in^2$

$db_t := if\left(B = 4, \frac{4}{8}\cdot in, if\left(B = 5, \frac{5}{8}\cdot in, if\left(B = 6, \frac{6}{8}\cdot in, if\left(B = 7, \frac{7}{8}\cdot in, if\left(B = 8, \frac{8}{8}\cdot in, if(B = 9, 1.128\cdot in, 0\cdot in)\right)\right)\right)\right)\right)$

$A_{sprov_bt} := \frac{\frac{\pi\cdot db_t^2}{4}\cdot(12\cdot in)}{Spacing_{t_COE}}$ $A_{sprov_bt} = 0.236\cdot in^2$ / foot

$Check\left[A_{sprov_bt}, \left(A_{temp_COE}\right)\right] = "OK"$

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Design.xmcd
Page 15 of 21

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Structural

REINFORCEMENT DESIGN
See Ref. 2 unless noted.

[d_1: remove bar thickness as moment force location used was maximum]

$d_1 := t1 - c1 - \left(\frac{5}{8}\text{in}\right) - \frac{1}{2}\cdot\left(\frac{5}{8}\text{in}\right)$ $d_1 = 14.0625\cdot\text{in}$

$d_2 := t2 - c2 - \frac{1}{2}\cdot\left(\frac{4}{8}\text{in}\right)$ $d_2 = 9.7500\cdot\text{in}$

$R_1 := \frac{M1_u}{\phi M\cdot b\cdot d_1^{\,2}}$ $R_1 = 66.79\cdot\text{psi}$

$R_2 := \frac{M2_u}{\phi M\cdot b\cdot d_2^{\,2}}$ $R_2 = 75.83\cdot\text{psi}$

$\rho_1 := \frac{0.85\cdot f'_c}{f_y}\cdot\left[1 - \sqrt{\left[1 - \left(\frac{2\cdot R_1}{0.85\cdot f'_c}\right)\right]}\right]$

$\rho_1 = 0.00112$

$\rho_2 := \frac{0.85\cdot f'_c}{f_y}\cdot\left[1 - \sqrt{\left[1 - \left(\frac{2\cdot R_2}{0.85\cdot f'_c}\right)\right]}\right]$

$\rho_2 = 0.00128$

9.6.1.2 $\rho_{min} := \text{if}\left[\left(\frac{200\cdot\text{psi}}{f_y}\right) > \left(\frac{3\sqrt{f'_c\cdot\text{psi}}}{f_y}\right), \left(\frac{200\cdot\text{psi}}{f_y}\right), \left(\frac{3\sqrt{f'_c\cdot\text{psi}}}{f_y}\right)\right]$ $\rho_{min} = 0.00333$

9.6.1.3 $\frac{4}{3}\cdot\rho_1 = 0.0015$ $\frac{4}{3}\cdot\rho_2 = 0.0017$

$\rho\rho_1 := \text{if}\left(\rho_{min} > \rho_1, \text{if}\left(\rho_{min} > \frac{4}{3}\cdot\rho_1, \frac{4}{3}\cdot\rho_1, \rho_{min}\right), \rho_1\right)$ $\rho\rho_1 = 0.00150$

$\rho\rho_2 := \text{if}\left(\rho_{min} > \rho_2, \text{if}\left(\rho_{min} > \frac{4}{3}\cdot\rho_2, \frac{4}{3}\cdot\rho_2, \rho_{min}\right), \rho_2\right)$ $\rho\rho_2 = 0.00170$

Ref. 3, Appendix C $\rho_{balanced} := \left(\frac{0.85\cdot f'_c\cdot\beta_1}{f_y}\right)\cdot\left(\frac{87000\cdot\text{psi}}{87000\cdot\text{psi} + f_y}\right)$ $\rho_{balanced} = 0.02851$

$\rho_{max} := 0.25\cdot\rho_{balanced}$ Ref. 3, Chapter 3, Section 3.5 $\rho_{max} = 0.00713$

$\text{rein_ratio}_1 := \text{if}(\rho_{max} > \rho\rho_1, \text{"OK"}, \text{"NG"})$ $\text{rein_ratio}_1 = \text{"OK"}$

$\text{rein_ratio}_2 := \text{if}(\rho_{max} > \rho\rho_2, \text{"OK"}, \text{"NG"})$ $\text{rein_ratio}_2 = \text{"OK"}$

$A_{s_1} := \rho\rho_1\cdot b\cdot d_1$ $A_{s_1} = 0.253\cdot\text{in}^2$ / foot

$A_{s_2} := \rho\rho_2\cdot b\cdot d_2$ $A_{s_2} = 0.199\cdot\text{in}^2$ / foot

$A_{smin_1} := \rho_{min}\cdot b\cdot d_1$ $A_{smin_1} = 0.563\cdot\text{in}^2$ / foot

$A_{smin_2} := \rho_{min}\cdot b\cdot d_2$ $A_{smin_2} = 0.390\cdot\text{in}^2$ / foot

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Design.xmcd
Page 16 of 21

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Structural

SECTION 1

****

$BarNo := 5$ $B := BarNo$

Use in both directions.
#5 @ 11" previously computed for temperature

$Spacing := 11 \cdot in$

****

$$db := if\left(B = 4, \frac{4}{8}\cdot in, if\left(B = 5, \frac{5}{8}\cdot in, if\left(B = 6, \frac{6}{8}\cdot in, if\left(B = 7, \frac{7}{8}\cdot in, if\left(B = 8, \frac{8}{8}\cdot in, if(B = 9, 1.128\cdot in, 1.270\cdot in)\right)\right)\right)\right)\right)$$

$$BarArea := \frac{\pi \cdot db^2}{4} \qquad BarArea = 0.307 \cdot in^2$$

$db = 0.63 \cdot in$

$$A_{sprov} := \frac{BarArea \cdot (12 \cdot in)}{Spacing} \qquad A_{sprov} = 0.335 \cdot in^2 \text{ / foot}$$

Check: $A_{sprov} = 0.335 \cdot in^2$ / foot >? $A_{s_1} = 0.253 \cdot in^2$ / foot $Check(A_{sprov}, A_{s_1}) = "OK"$

$$d_{w_1} := t1 - c1 - \left(\frac{5}{8}in\right) - \frac{db}{2} \qquad d_{w_1} = 14.063 \cdot in$$

[remove bar thickness as moment force location used was maximum]

$$a := \frac{A_{sprov} \cdot f_y}{0.85 \cdot f'_c \cdot b} \qquad a = 0.492 \cdot in$$

$$\phi M_{n_1} := \phi M \cdot \left[A_{sprov} \cdot f_y \cdot \left(d_{w_1} - \frac{a}{2}\right)\right]$$

$\phi M_{n_1} = 20.81 \cdot kip \cdot ft$ >? $M1_u = 11.89 \cdot kip \cdot ft$ $Check(\phi M_{n_1}, M1_u) = "OK"$

SHEAR

Shear Strength of Normal-Weight Concrete (22.5.5.1)

$$\phi V1_c := \phi V \cdot \left(2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d_{w_1}\right)$$

$\phi V1_c = 16.01 \cdot kip$

$Check(\phi V1_c, V1_u) = "OK"$

Maximum Shear

$V1_u = 8.95 \cdot kip$

Minimum Shear Reinforcement (7.6.3.2)
None required if M_n and V_n can be developed.

AYRES

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Basin Design.xmcd
Page 17 of 21

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Structural

SECTION 2

****

BarNo := 4 B := BarNo

Use in both directions.
#4 @ 10" previously computed for temperature

Spacing := 10·in

****

$$db := if\left(B = 4, \frac{4}{8}\cdot in, if\left(B = 5, \frac{5}{8}\cdot in, if\left(B = 6, \frac{6}{8}\cdot in, if\left(B = 7, \frac{7}{8}\cdot in, if\left(B = 8, \frac{8}{8}\cdot in, if(B = 9, 1.128\cdot in, 1.270\cdot in)\right)\right)\right)\right)\right)$$

$$BarArea := \frac{\pi\cdot db^2}{4} \qquad BarArea = 0.196\cdot in^2$$

$db = 0.5\cdot in$

$$A_{sprov} := \frac{BarArea\cdot(12\cdot in)}{Spacing} \qquad A_{sprov} = 0.236\cdot in^2 \text{ / foot}$$

Check: $A_{sprov} = 0.236\cdot in^2$ / foot >? $A_{s_2} = 0.199\cdot in^2$ / foot $Check(A_{sprov}, A_{s_2}) = "OK"$

$$d_{w_2} := t2 - c2 - \frac{db}{2} \qquad d_{w_2} = 9.750\cdot in$$

$$a := \frac{A_{sprov}\cdot f_y}{0.85\cdot f'_c\cdot b} \qquad a = 0.346\cdot in$$

$$\phi M_{n_2} := \phi M\cdot\left[A_{sprov}\cdot f_y\cdot\left(d_{w_2} - \frac{a}{2}\right)\right]$$

$\phi M_{n_2} = 10.15\cdot kip\cdot ft$ >? $M2_u = 6.49\cdot kip\cdot ft$ $Check(\phi M_{n_2}, M2_u) = "OK"$

SHEAR

Shear Strength of Normal-Weight Concrete (22.5.5.1)

$$\phi V2_c := \phi V\cdot\left(2\cdot\sqrt{f'_c\cdot psi}\cdot b\cdot d_{w_2}\right)$$

$\phi V2_c = 11.10\cdot kip$

$Check(\phi V2_c, V2_u) = "OK"$

Maximum Shear

$V2_u = 2.94\cdot kip$

Minimum Shear Reinforcement (7.6.3.2)
None required if M_n and V_n can be developed.

Structural

UPSTREAM KEY

-- Check moment and shear based on stability resistance in Ref. 10.
-- Unfactored Load Case 6 controls per foot of width.
-- To simplify from above load factor summary, use the 1.35 maximum Load Factor.
-- Key thickness same as Section 1.

$F_{resist} := 3.18 \cdot kip$ $\quad Load_Factor_{key} := 1.35$

$M_{resist} := 2.91 \cdot kip \cdot ft$

$F_{resist_u} := Load_Factor_{key} \cdot F_{resist}$ $\quad F_{resist_u} = 4.29 \cdot kip$

$M_{resist_u} := Load_Factor_{key} \cdot M_{resist}$ $\quad M_{resist_u} = 3.93 \cdot kip \cdot ft$

Shear Strength of Normal-Weight Concrete (22.5.5.1)

$\phi Vkey_c := \phi V \cdot \left(2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d_{w_1}\right)$ $\quad \phi Vkey_c = 16.01 \cdot kip$

$Check\left(\phi Vkey_c, F_{resist_u}\right) = "OK"$

Minimum Shear Reinforcement (7.6.3.2)
None required if M_n and V_n can be developed.

Moment: See Section 1 design as $M_{1u} > M_{resist_u}$

$M1_u = 11.89 \cdot kip \cdot ft$ $\quad > \quad M_{resist_u} = 3.93 \cdot kip \cdot ft$

$Check\left(M1_u, M_{resist_u}\right) = "OK"$

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
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Structural

WINGWALLS

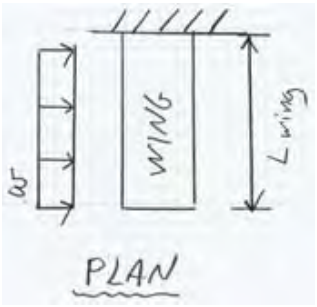
-- Use information from stability in Ref. 10.
-- Conservatively use pressures at wall base along a 1-foot height.
-- Ignore resisting soil.
-- K_a = 0.340
-- Use the following Load Factors: Moist = 1.6, Normal Pool (NP) = 2.2, IDF = 1.35
-- Wall thickness same as section 2.

$L_{wing} := 4 \cdot ft + 0 \cdot in$

$Load_Factor_{moist} := 1.6$

$Load_Factor_{NP} := 2.2$

$Load_Factor_{IDF} := 1.35$



Pressures

$\omega_{moist_u} := Load_Factor_{moist} \cdot (0.340) \cdot (0.761 \cdot ksf) \cdot (1.0 \cdot ft)$ $\omega_{moist_u} = 0.414 \cdot \frac{kip}{ft}$

$\omega_{sat_NP_u} := Load_Factor_{NP} \cdot (0.340) \cdot (0.618 \cdot ksf) \cdot (1.0 \cdot ft)$ $\omega_{sat_NP_u} = 0.462 \cdot \frac{kip}{ft}$

$\omega_{sat_IDF_u} := Load_Factor_{IDF} \cdot (0.340) \cdot (0.423 \cdot ksf) \cdot (1.0 \cdot ft)$ $\omega_{sat_IDF_u} = 0.194 \cdot \frac{kip}{ft}$

Water not computed since equal on both sides so cancel each other out.

$\omega_{max_u} := \max(\omega_{moist_u}, \omega_{sat_NP_u}, \omega_{sat_IDF_u})$ $\omega_{max_u} = 0.462 \cdot \frac{kip}{ft}$

Factored Shear and Moment on Cantilever Section

$V_{u_wing} := \omega_{max_u} \cdot L_{wing}$ $V_{u_wing} = 1.85 \cdot kip$

$M_{u_wing} := \frac{\omega_{max_u} \cdot L_{wing}^{2}}{2}$ $M_{u_wing} = 3.70 \cdot kip \cdot ft$

Ayres Associates Inc
Eau Claire, WI

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Structural

REINFORCEMENT DESIGN
See Ref. 2 unless noted.

$d := t2 - c2 - \left(\frac{4}{8}\text{in}\right) - \frac{1}{2}\cdot\left(\frac{4}{8}\text{in}\right)$ $d = 9.2500\cdot\text{in}$ [conservatively remove bar thickness due to potential variable placement]

$R := \frac{M_{u_wing}}{\phi M\cdot b\cdot d^2}$ $R = 48.02\cdot\text{psi}$

$\rho := \frac{0.85\cdot f'_c}{f_y}\cdot\left[1 - \sqrt{\left[1 - \left(\frac{2\cdot R_1}{0.85\cdot f'_c}\right)\right]}\right]$ $\rho = 0.00112$

9.6.1.2 $\rho_{min} := \text{if}\left[\left(\frac{200\cdot\text{psi}}{f_y}\right) > \left(\frac{3\sqrt{f'_c\cdot\text{psi}}}{f_y}\right), \left(\frac{200\cdot\text{psi}}{f_y}\right), \left(\frac{3\sqrt{f'_c\cdot\text{psi}}}{f_y}\right)\right]$ $\rho_{min} = 0.00333$

9.6.1.3 $\frac{4}{3}\cdot\rho = 0.0015$

$\rho\rho := \text{if}\left(\rho_{min} > \rho, \text{if}\left(\rho_{min} > \frac{4}{3}\cdot\rho, \frac{4}{3}\cdot\rho, \rho_{min}\right), \rho\right)$ $\rho\rho_1 = 0.00150$

Ref. 3, Appendix C $\rho_{balanced} := \left(\frac{0.85\cdot f'_c\cdot\beta_1}{f_y}\right)\cdot\left(\frac{87000\cdot\text{psi}}{87000\cdot\text{psi} + f_y}\right)$ $\rho_{balanced} = 0.02851$

$\rho_{max} := 0.25\cdot\rho_{balanced}$ Ref. 3, Chapter 3, Section 3.5 $\rho_{max} = 0.00713$

$\text{rein_ratio}_1 := \text{if}(\rho_{max} > \rho\rho, \text{"OK"}, \text{"NG"})$ $\text{rein_ratio}_1 = \text{"OK"}$

$A_s := \rho\rho\cdot b\cdot d$ $A_s = 0.166\cdot\text{in}^2$ / foot

$A_{smin} := \rho_{min}\cdot b\cdot d$ $A_{smin} = 0.370\cdot\text{in}^2$ / foot

Ayres Associates Inc
Eau Claire, WI

Project No.: 26-1144.00
Project Name: Hogchute Dam

USBR Type VI Basin

Prepared By: TMR Jan 2021
Checked By: AEB Feb 2021

Structural

****

$BarNo := 4$ $B := BarNo$ Use in both directions.
#4 @ 10" previously computed for temperature

$Spacing := 10 \cdot in$

****

$$db := if\left(B = 4, \frac{4}{8}\cdot in, if\left(B = 5, \frac{5}{8}\cdot in, if\left(B = 6, \frac{6}{8}\cdot in, if\left(B = 7, \frac{7}{8}\cdot in, if\left(B = 8, \frac{8}{8}\cdot in, if(B = 9, 1.128\cdot in, 1.270\cdot in)\right)\right)\right)\right)\right)$$

$$BarArea := \frac{\pi \cdot db^2}{4} \qquad BarArea = 0.196 \cdot in^2$$

$db = 0.5 \cdot in$

$$A_{sprov} := \frac{BarArea \cdot (12 \cdot in)}{Spacing} \qquad A_{sprov} = 0.236 \cdot in^2 \text{ / foot}$$

Check: $A_{sprov} = 0.236 \cdot in^2$ / foot >? $A_s = 0.166 \cdot in^2$ / foot $Check(A_{sprov}, A_s) = "OK"$

$$d_w := t2 - c2 - \left(\frac{4}{8}in\right) - \frac{db}{2} \qquad d_w = 9.250 \cdot in$$

$$a := \frac{A_{sprov} \cdot f_y}{0.85 \cdot f'_c \cdot b} \qquad a = 0.346 \cdot in$$

$$\phi M_n := \phi M \cdot \left[A_{sprov} \cdot f_y \cdot \left(d_w - \frac{a}{2}\right)\right]$$

$\phi M_n = 9.62 \cdot kip \cdot ft$ >? $M_{u_wing} = 3.70 \cdot kip \cdot ft$ $Check(\phi M_n, M_{u_wing}) = "OK"$

SHEAR

Shear Strength of Normal-Weight Concrete (22.5.5.1)

$$\phi V_c := \phi V \cdot \left(2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d_w\right)$$

$\phi V_c = 10.53 \cdot kip$

$Check(\phi V_c, V_{u_wing}) = "OK"$

Maximum Shear

$V_{u_wing} = 1.85 \cdot kip$

Minimum Shear Reinforcement (7.6.3.2)
None required if M_n and V_n can be developed.

REINFORCEMENT

Headwall and Abutments: 1'-6" thick w/ #5@11" OC, EW, EF

Baffle, Endsill, and Wing Walls: 1'-0" thick w/ #4@10" OC, EW, EF

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	Part		
Job Title HOGCHUTE	Ref BASIN FINITE		
	By AYRES-TMR	Date JAN-2021	Chd AYRES-AEB
Client COLORADO	File TypeVI.std	Date/Time 05-Feb-2021 10:16	

Job Information

	Engineer	Checked	Approved
Name:	AYRES-TMR	AYRES-AEB	
Date:	JAN-2021	FEB-2021	

Project ID	
Project Name	

Comments

12-INCH GRID
MEMBER CENTERLINE

Structure Type	SPACE FRAME

Number of Nodes	828	Highest Node	40912
Number of Plates	776	Highest Plate	50814

Number of Basic Load Cases	16
Number of Combination Load Cases	22

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Combination	101	UNUSUAL
Combination	102	EXTREME
Combination	103	USUAL
Combination	104	UNUSUAL
Combination	105	UNUSUAL
Combination	106	UNUSUAL
Combination	107	UNUSUAL
Combination	108	EXTREME
Combination	109	EXTREME
Combination	110	EXTREME
Combination	111	EXTREME

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
101	UNUSUAL	1	SELFWEIGHT	1.60
		5	MOIST SOIL	1.60
		8	SURCHARGE	1.60
102	EXTREME	1	SELFWEIGHT	1.20
		5	MOIST SOIL	1.35
		11	EQ - OBE1	1.25
103	USUAL	1	SELFWEIGHT	2.20
		3	NP UPLIFT	2.20
		6	SAT SOIL - NP	2.20
		9	WATER - NP	2.20
104	UNUSUAL	1	SELFWEIGHT	1.60
		3	NP UPLIFT	1.60
		6	SAT SOIL - NP	1.60
		8	SURCHARGE	1.60
		9	WATER - NP	1.60
105	UNUSUAL	1	SELFWEIGHT	1.50
		3	NP UPLIFT	1.50
		6	SAT SOIL - NP	1.50
		9	WATER - NP	1.50
		11	EQ - OBE1	1.50
106	UNUSUAL	1	SELFWEIGHT	1.50
		3	NP UPLIFT	1.50
		6	SAT SOIL - NP	1.50
		9	WATER - NP	1.50
		12	EQ - OBE2	1.50
107	UNUSUAL	1	SELFWEIGHT	1.50
		3	NP UPLIFT	1.50
		6	SAT SOIL - NP	1.50
		9	WATER - NP	1.50
		13	EQ - OBE3	1.50
108	EXTREME	1	SELFWEIGHT	1.20
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.35
		9	WATER - NP	1.00
		14	EQ - MDE1	1.25
109	EXTREME	1	SELFWEIGHT	1.20
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.35
		9	WATER - NP	1.00
		15	EQ - MDE2	1.25
110	EXTREME	1	SELFWEIGHT	1.20
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.35
		9	WATER - NP	1.00
		16	EQ - MDE3	1.25

Combination Load Cases Cont...

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
111	EXTREME	1	SELFWEIGHT	1.20
		2	IDF WATER WT	0.90
		4	IDF UPLIFT	1.30
		7	SAT SOIL - IDF	1.35
		10	PIPE JET - IDF	1.30
201	UNUSUAL	1	SELFWEIGHT	1.00
		5	MOIST SOIL	1.00
		8	SURCHARGE	1.00
202	EXTREME	1	SELFWEIGHT	1.00
		5	MOIST SOIL	1.00
		11	EQ - OBE1	1.00
203	USUAL	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		9	WATER - NP	1.00
204	UNUSUAL	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		8	SURCHARGE	1.00
		9	WATER - NP	1.00
205	UNUSUAL	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		9	WATER - NP	1.00
		11	EQ - OBE1	1.00
206	UNUSUAL	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		9	WATER - NP	1.00
		12	EQ - OBE2	1.00
207	UNUSUAL	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		9	WATER - NP	1.00
		13	EQ - OBE3	1.00
208	EXTREME	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		9	WATER - NP	1.00
		14	EQ - MDE1	1.00
209	EXTREME	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		9	WATER - NP	1.00
		15	EQ - MDE2	1.00

Combination Load Cases Cont...

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
210	EXTREME	1	SELFWEIGHT	1.00
		3	NP UPLIFT	1.00
		6	SAT SOIL - NP	1.00
		9	WATER - NP	1.00
		16	EQ - MDE3	1.00
211	EXTREME	1	SELFWEIGHT	1.00
		2	IDF WATER WT	1.00
		4	IDF UPLIFT	1.00
		7	SAT SOIL - IDF	1.00
		10	PIPE JET - IDF	1.00

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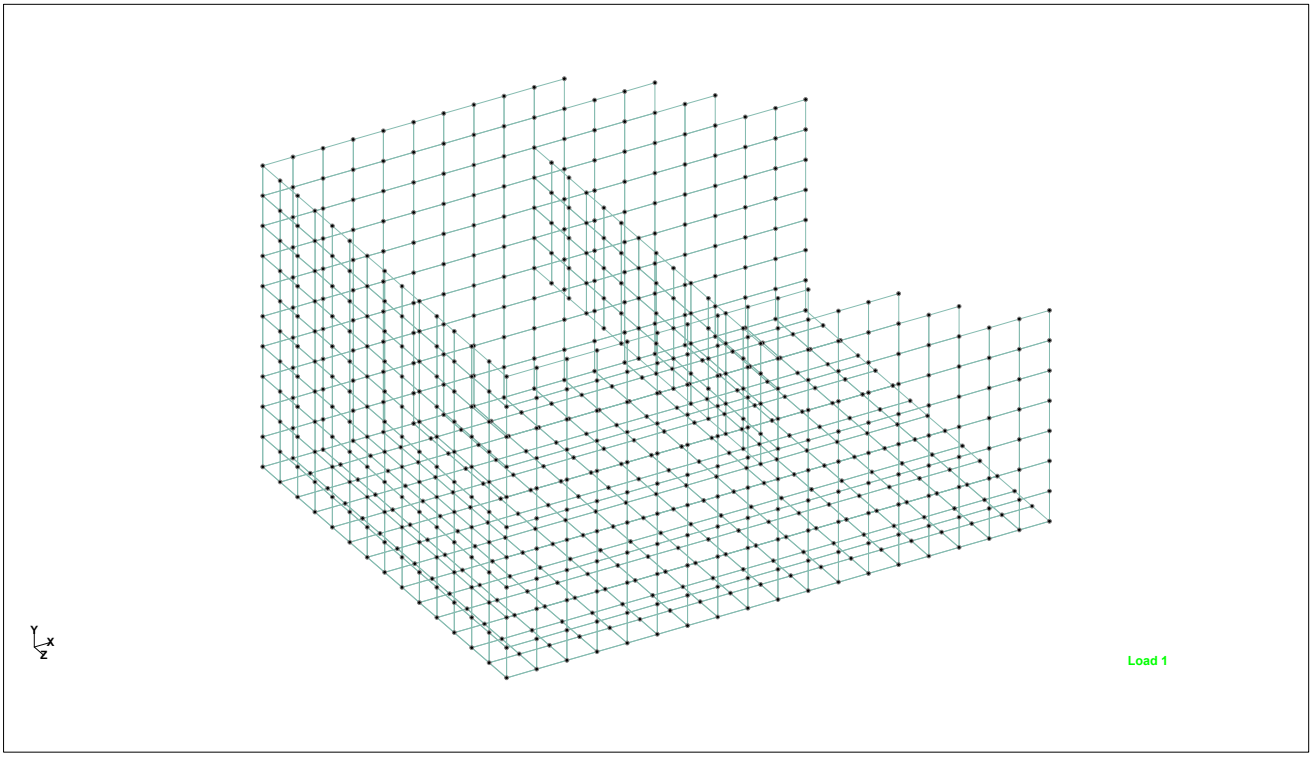
	Part		
Job Title HOGCHUTE	Ref BASIN FINITE		
	By AYRES-TMR	Date JAN-2021	Chd AYRES-AEB
Client COLORADO	File TypeVI.std	Date/Time 05-Feb-2021 10:16	



Whole Structure

Plate Center Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (kip/ft^2)	Qy (kip/ft^2)	Sx (kip/ft^2)	Sy (kip/ft^2)	Sxy (kip/ft^2)	Mx (kip-ft/ft)	My (kip-ft/ft)	Mxy (kip-ft/ft)
Max Qx	31014	111:EXTREME	**3.16**	0.14	-1.08	0.27	0.75	5.508	0.261	-0.564
Min Qx	31001	104:UNUSUAL	**-2.74**	0.16	-0.98	0.04	-0.41	4.723	0.228	0.401
Max Qy	41418	103:USUAL	-0.79	**5.63**	0	0	0	-0.420	-8.868	-0.914
Min Qy	40118	103:USUAL	-0.67	**-5.97**	0	0	0	-0.351	-9.248	1.056
Max Sx	20716	111:EXTREME	0.04	-0.02	**1.46**	1.26	-0.33	-0.100	-0.035	0.046
Min Sx	31008	111:EXTREME	1.15	0.19	**-3.54**	-0.08	-0.03	-3.836	-0.304	-0.452
Max Sy	30114	111:EXTREME	-0.32	-0.50	0.86	**7.42**	0.76	1.237	2.653	-0.601
Min Sy	10118	103:USUAL	0.03	2.31	-0.51	**-7.18**	1.55	-1.235	-10.692	1.195
Max Sxy	20101	111:EXTREME	-0.79	0.29	1.46	6.08	**5.67**	0.752	0.235	0.512
Min Sxy	30401	111:EXTREME	-1.45	0.71	-1.45	0.54	**-1.99**	2.164	0.010	0.834
Max Mx	21001	103:USUAL	-1.18	-0.64	-3.10	-0.32	0.13	**7.154**	0.324	-0.198
Min Mx	11001	104:UNUSUAL	1.11	0.55	-3.01	-0.26	0.21	**-5.670**	-0.260	0.229
Max My	20118	103:USUAL	-0.03	-2.37	-0.33	-5.51	1.20	1.169	**10.115**	-1.012
Min My	10118	103:USUAL	0.03	2.31	-0.51	-7.18	1.55	-1.235	**-10.692**	1.195
Max Mxy	40217	103:USUAL	-0.10	-3.19	0	0	0	-0.406	-3.263	**2.465**
Min Mxy	30312	103:USUAL	0.81	-0.49	-1.54	-1.15	-0.45	-0.284	-0.693	**-2.321**

Plate Center Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (kip/ft^2)	Qy (kip/ft^2)	Sx (kip/ft^2)	Sy (kip/ft^2)	Sxy (kip/ft^2)	Mx (kip-ft/ft)	My (kip-ft/ft)	Mxy (kip-ft/ft)
Max Qx	50813	111:EXTREME	**2.90**	0.01	-0.51	0.40	0.10	3.700	0.116	-0.167
Min Qx	50501	111:EXTREME	**-2.94**	-0.93	-2.19	-0.17	-0.84	5.368	0.620	0.340
Max Qy	50513	111:EXTREME	2.47	**0.26**	-0.97	-0.12	0.05	3.603	0.102	0.124
Min Qy	50814	111:EXTREME	2.74	**-1.25**	-0.21	-0.02	0.08	6.429	0.374	-0.059
Max Sx	50507	102:EXTREME	-0.00	-0.00	**0.42**	0.05	-0.06	-0.050	-0.000	-0.000
Min Sx	50807	101:UNUSUAL	-0.00	0.00	**-10.20**	-0.07	-0.08	-0.003	0.000	-0.000
Max Sy	50804	103:USUAL	0.01	-0.01	-6.22	**0.47**	-0.33	-0.217	-0.020	-0.060
Min Sy	50501	101:UNUSUAL	0.01	-0.22	-7.08	**-1.76**	-2.39	-0.144	0.018	-0.006
Max Sxy	50514	101:UNUSUAL	-0.01	-0.22	-7.08	-1.76	**2.39**	-0.144	0.018	0.006
Min Sxy	50601	103:USUAL	-0.30	-0.86	-4.33	-1.22	**-2.40**	0.490	0.057	0.041
Max Mx	50814	111:EXTREME	2.74	-1.25	-0.21	-0.02	0.08	**6.429**	0.374	-0.059
Min Mx	50808	111:EXTREME	0.43	0.08	-1.46	-0.10	0.24	**-4.244**	0.028	-0.049
Max My	50614	111:EXTREME	2.56	-0.81	-1.35	-1.11	0.16	6.052	**0.801**	-0.201
Min My	50708	111:EXTREME	0.29	-0.00	-1.65	0.11	0.18	-4.229	**-0.114**	-0.089
Max Mxy	50501	111:EXTREME	-2.94	-0.93	-2.19	-0.17	-0.84	5.368	0.620	**0.340**
Min Mxy	50514	111:EXTREME	2.65	-0.59	-1.18	-0.38	0.23	6.217	0.706	**-0.414**

PAGE NO. 1

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*          Version  22.05.00.131                 *
*          Proprietary Program of                *
*          Bentley Systems, Inc.                 *
*          Date=    FEB  5, 2021                 *
*          Time=    10:16:18                     *
*                                                *
*  Licensed to: Ayres Associates                 *
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     4. JOB CLIENT COLORADO
     5. JOB NO 26-1144.00
     6. JOB REF BASIN FINITE
     7. JOB COMMENT 12-INCH GRID
     8. JOB COMMENT MEMBER CENTERLINE
     9. ENGINEER NAME AYRES-TMR
    10. CHECKER NAME AYRES-AEB
    11. ENGINEER DATE JAN-2021
    12. CHECKER DATE FEB-2021
    13. END JOB INFORMATION
    14. INPUT WIDTH 79
    15. SET NL 20
    16. UNIT FEET KIP
    17. JOINT COORDINATES
    18. * ABUTMENT 1
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    20. 1200 0 1 0; REPEAT 18 1 0 0
    21. 1300 0 2 0; REPEAT 18 1 0 0
    22. 1400 0 3 0; REPEAT 18 1 0 0
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    28. 2000 0 9 0; REPEAT 13 1 0 0
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40. 11900 0 8 14; REPEAT 15 1 0 0
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42. 12100 0 10 14; REPEAT 10 1 0 0
43. *
44. * HEADWALL
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47. 20300 0 2 1; REPEAT 12 0 0 1
48. 20400 0 3 1; REPEAT 12 0 0 1
49. 20500 0 4 1; REPEAT 12 0 0 1
50. 20600 0 5 1; REPEAT 12 0 0 1
51. 20700 0 6 1; REPEAT 12 0 0 1
52. 20800 0 7 1; REPEAT 12 0 0 1
53. 20900 0 8 1; REPEAT 12 0 0 1
54. 21000 0 9 1; REPEAT 12 0 0 1
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56. *
57. * SLAB
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76. 40800 9 7 1; REPEAT 12 0 0 1
77. 40900 9 8 1; REPEAT 12 0 0 1
78. *
79. ELEMENT INCIDENCES SHELL
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83. 10301 1300 1301 1401 1400; REPEAT 17 1 1
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87. 10701 1700 1701 1801 1800; REPEAT 17 1 1
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92. * ABUTMENT 2
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94. 20201 11200 11201 11301 11300; REPEAT 17 1 1
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TYPE VI BASIN -- PAGE NO. 3

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114. 30302 20300 20301 20401 20400; REPEAT 11 1 1
115. 30314 20312 11300 11400 20412
116. *
117. 30401 1400 20400 20500 1500
118. 30402 20400 20401 20501 20500; REPEAT 11 1 1
119. 30414 20412 11400 11500 20512
120. *
121. 30501 1500 20500 20600 1600
122. 30502 20500 20501 20601 20600; REPEAT 11 1 1
123. 30514 20512 11500 11600 20612
124. *
125. 30601 1600 20600 20700 1700
126. 30602 20600 20601 20701 20700; REPEAT 11 1 1
127. 30614 20612 11600 11700 20712
128. *
129. 30701 1700 20700 20800 1800
130. 30702 20700 20701 20801 20800; REPEAT 11 1 1
131. 30714 20712 11700 11800 20812
132. *
133. 30801 1800 20800 20900 1900
134. 30802 20800 20801 20901 20900; REPEAT 11 1 1
135. 30814 20812 11800 11900 20912
136. *
137. 30901 1900 20900 21000 2000
138. 30902 20900 20901 21001 21000; REPEAT 11 1 1
139. 30914 20912 11900 12000 21012
140. *
141. 31001 2000 21000 21100 2100
142. 31002 21000 21001 21101 21100; REPEAT 11 1 1
143. 31014 21012 12000 12100 21112
144. *
145. * SLAB
146. 40101 1100 20100 30100 1101
147. 40102 30100 30101 1102 1101; REPEAT 16 1 1
148. *
149. 40201 20100 20101 30200 30100
150. 40202 30200 30201 30101 30100; REPEAT 16 1 1
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151. *
152. 40301 20101 20102 30300 30200
153. 40302 30300 30301 30201 30200; REPEAT 16 1 1
154. *
155. 40401 20102 20103 30400 30300
156. 40402 30400 30401 30301 30300; REPEAT 16 1 1
157. *
158. 40501 20103 20104 30500 30400
159. 40502 30500 30501 30401 30400; REPEAT 16 1 1
160. *
161. 40601 20104 20105 30600 30500
162. 40602 30600 30601 30501 30500; REPEAT 16 1 1
163. *
164. 40701 20105 20106 30700 30600
165. 40702 30700 30701 30601 30600; REPEAT 16 1 1
166. *
167. 40801 20106 20107 30800 30700
168. 40802 30800 30801 30701 30700; REPEAT 16 1 1
169. *
170. 40901 20107 20108 30900 30800
171. 40902 30900 30901 30801 30800; REPEAT 16 1 1
172. *
173. 41001 20108 20109 31000 30900
174. 41002 31000 31001 30901 30900; REPEAT 16 1 1
175. *
176. 41101 20109 20110 31100 31000
177. 41102 31100 31101 31001 31000; REPEAT 16 1 1
178. *
179. 41201 20110 20111 31200 31100
180. 41202 31200 31201 31101 31100; REPEAT 16 1 1
181. *
182. 41301 20111 20112 31300 31200
183. 41302 31300 31301 31201 31200; REPEAT 16 1 1
185. *
186. 41401 20112 11100 11101 31300
187. 41402 11101 11102 31301 31300; REPEAT 16 1 1
188. *
189. * BAFFLE
190. *
191. 50501 1509 40500 40600 1609
192. 50502 40500 40501 40601 40600; REPEAT 11 1 1
193. 50514 40512 11509 11609 40612
194. *
195. 50601 1609 40600 40700 1709
196. 50602 40600 40601 40701 40700; REPEAT 11 1 1
197. 50614 40612 11609 11709 40712
198. *
199. 50701 1709 40700 40800 1809
200. 50702 40700 40701 40801 40800; REPEAT 11 1 1
201. 50714 40712 11709 11809 40812
202. *
203. 50801 1809 40800 40900 1909
204. 50802 40800 40801 40901 40900; REPEAT 11 1 1
205. 50814 40812 11809 11909 40912
206. * ------------------------------------------------------------- -
207. UNIT INCHES KIP
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208. ELEMENT PROPERTY
209. * ABUTMENT 1
210. 10101 TO 10118 10201 TO 10218 10301 TO 10318 10401 TO 10418 -
211. 10501 TO 10518 10601 TO 10618 10701 TO 10718 10801 TO 10815 -
212. 10901 TO 10913 11001 TO 11010 THICKNESS 18
213. * ABUTMENT 2
214. 20101 TO 20118 20201 TO 20218 20301 TO 20318 20401 TO 20418 -
215. 20501 TO 20518 20601 TO 20618 20701 TO 20718 20801 TO 20815 -
216. 20901 TO 20913 21001 TO 21010 THICKNESS 18
217. * HEADWALL
218. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
219. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
220. 30901 TO 30914 31001 TO 31014 THICKNESS 18
221. * SLAB
222. 40101 TO 40118 40201 TO 40218 40301 TO 40318 40401 TO 40418 -
223. 40501 TO 40518 40601 TO 40618 40701 TO 40718 40801 TO 40818 -
224. 40901 TO 40918 41001 TO 41018 41101 TO 41118 41201 TO 41218 -
225. 41301 TO 41318 41401 TO 41418 THICKNESS 18
226. * BAFFLE
227. 50501 TO 50514 50601 TO 50614 50701 TO 50714 50801 TO 50814 -
228. THICKNESS 12
229. *
230. DEFINE MATERIAL START
231. UNIT FEET KIP
232. * FC = 4000 PSI
233. ISOTROPIC CONCRETE40
234. E 519120
235. POISSON 0.17
236. DENSITY 0.150
237. ALPHA 5.0E-006
238. END DEFINE MATERIAL
239. UNIT INCHES KIP
240. CONSTANTS
241. MATERIAL CONCRETE40 ALL
242. *
243. SPRING COMPRESSION
244. 1101 TO 1118 11101 TO 11118 -
245. 30100 TO 30117 30200 TO 30217 30300 TO 30317 -
246. 30400 TO 30417 30500 TO 30517 30600 TO 30617 -
247. 30700 TO 30717 30800 TO 30817 30900 TO 30917 -
248. 31000 TO 31017 31100 TO 31117 31200 TO 31217 -
249. 31300 TO 31317 KFY
250. UNIT INCHES POUND
251. SUPPORTS
252. * SUBGRADE REACTION IS 83 LB/CU. IN.
253. 1101 TO 1118 11101 TO 11118 -
254. 30100 TO 30117 30200 TO 30217 30300 TO 30317 -
255. 30400 TO 30417 30500 TO 30517 30600 TO 30617 -
256. 30700 TO 30717 30800 TO 30817 30900 TO 30917 -
257. 31000 TO 31017 31100 TO 31117 31200 TO 31217 -
258. 31300 TO 31317 ELASTIC MAT DIREC Y SUBG 83
259. *
260. * LOCATION OF PINNED SUPPORTS DOES NOT
261. * SIGNIFICANTLY IMPACT RESULTS
262. 1100 20100 TO 20112 11100 PINNED
263. *
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264. *
265. * LOADS
266. UNIT FEET KIP
267. LOAD 1 SELFWEIGHT
268. SELFWEIGHT Y -1
269. *
270. LOAD 2 IDF WATER WT
271. ELEMENT LOAD
272. 40101 TO 40118 40201 TO 40218 40301 TO 40318 40401 TO 40418 -
273. 40501 TO 40518 40601 TO 40618 40701 TO 40718 40801 TO 40818 -
274. 40901 TO 40918 41001 TO 41018 41101 TO 41118 41201 TO 41218 -
275. 41301 TO 41318 41401 TO 41418 PRES GY -0.274
276. *
277. LOAD 3 NP UPLIFT
278. ELEMENT LOAD
279. 40101 TO 40118 40201 TO 40218 40301 TO 40318 40401 TO 40418 -
280. 40501 TO 40518 40601 TO 40618 40701 TO 40718 40801 TO 40818 -
281. 40901 TO 40918 41001 TO 41018 41101 TO 41118 41201 TO 41218 -
282. 41301 TO 41318 41401 TO 41418 PRES GY 0.156
283. *
284. LOAD 4 IDF UPLIFT
285. ELEMENT LOAD
286. 40101 TO 40118 40201 TO 40218 40301 TO 40318 40401 TO 40418 -
287. 40501 TO 40518 40601 TO 40618 40701 TO 40718 40801 TO 40818 -
288. 40901 TO 40918 41001 TO 41018 41101 TO 41118 41201 TO 41218 -
289. 41301 TO 41318 41401 TO 41418 PRES GY 0.368
290. *
291. LOAD 5 MOIST SOIL
292. ELEMENT LOAD
293. *
294. * ABUTMENT 1
295. * 10
296. 10101 TO 10110 TRAP Y 0.646 0.581
297. 10201 TO 10210 TRAP Y 0.581 0.517
298. 10301 TO 10310 TRAP Y 0.517 0.452
299. 10401 TO 10410 TRAP Y 0.452 0.388
300. 10501 TO 10510 TRAP Y 0.388 0.323
301. 10601 TO 10610 TRAP Y 0.323 0.258
302. 10701 TO 10710 TRAP Y 0.258 0.194
303. 10801 TO 10810 TRAP Y 0.194 0.129
304. 10901 TO 10910 TRAP Y 0.129 0.065
305. 11001 TO 11010 TRAP Y 0.065 0
306. * 9
307. 10111 TO 10113 TRAP Y 0.581 0.517
308. 10211 TO 10213 TRAP Y 0.517 0.452
309. 10311 TO 10313 TRAP Y 0.452 0.388
310. 10411 TO 10413 TRAP Y 0.388 0.323
311. 10511 TO 10513 TRAP Y 0.323 0.258
312. 10611 TO 10613 TRAP Y 0.258 0.194
313. 10711 TO 10713 TRAP Y 0.194 0.129
314. 10811 TO 10813 TRAP Y 0.129 0.065
315. 10911 TO 10913 TRAP Y 0.065 0
316. * 8
317. 10114 10115 TRAP Y 0.517 0.452
318. 10214 10215 TRAP Y 0.452 0.388
319. 10314 10315 TRAP Y 0.388 0.323
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320. 10414 10415 TRAP Y 0.323 0.258
321. 10514 10515 TRAP Y 0.258 0.194
322. 10614 10615 TRAP Y 0.194 0.129
323. 10714 10715 TRAP Y 0.129 0.065
324. 10814 10815 TRAP Y 0.065 0
325. * 7
326. 10116 TO 10118 TRAP Y 0.452 0.388
327. 10216 TO 10218 TRAP Y 0.388 0.323
328. 10316 TO 10318 TRAP Y 0.323 0.258
329. 10416 TO 10418 TRAP Y 0.258 0.194
330. 10516 TO 10518 TRAP Y 0.194 0.129
331. 10616 TO 10618 TRAP Y 0.129 0.065
332. 10716 TO 10718 TRAP Y 0.065 0
333. *
334. * ABUTMENT 2
335. * 10
336. 20101 TO 20110 TRAP Y -0.646 -0.581
337. 20201 TO 20210 TRAP Y -0.581 -0.517
338. 20301 TO 20310 TRAP Y -0.517 -0.452
339. 20401 TO 20410 TRAP Y -0.452 -0.388
340. 20501 TO 20510 TRAP Y -0.388 -0.323
341. 20601 TO 20610 TRAP Y -0.323 -0.258
342. 20701 TO 20710 TRAP Y -0.258 -0.194
343. 20801 TO 20810 TRAP Y -0.194 -0.129
344. 20901 TO 20910 TRAP Y -0.129 -0.065
345. 21001 TO 21010 TRAP Y -0.065 0
346. * 9
347. 20111 TO 20113 TRAP Y -0.581 -0.517
348. 20211 TO 20213 TRAP Y -0.517 -0.452
349. 20311 TO 20313 TRAP Y -0.452 -0.388
350. 20411 TO 20413 TRAP Y -0.388 -0.323
351. 20511 TO 20513 TRAP Y -0.323 -0.258
352. 20611 TO 20613 TRAP Y -0.258 -0.194
353. 20711 TO 20713 TRAP Y -0.194 -0.129
354. 20811 TO 20813 TRAP Y -0.129 -0.065
355. 20911 TO 20913 TRAP Y -0.065 0
356. * 8
357. 20114 20115 TRAP Y -0.517 -0.452
358. 20214 20215 TRAP Y -0.452 -0.388
359. 20314 20315 TRAP Y -0.388 -0.323
360. 20414 20415 TRAP Y -0.323 -0.258
361. 20514 20515 TRAP Y -0.258 -0.194
362. 20614 20615 TRAP Y -0.194 -0.129
363. 20714 20715 TRAP Y -0.129 -0.065
364. 20814 20815 TRAP Y -0.065 0
365. * 7
366. 20116 TO 20118 TRAP Y -0.452 -0.388
367. 20216 TO 20218 TRAP Y -0.388 -0.323
368. 20316 TO 20318 TRAP Y -0.323 -0.258
369. 20416 TO 20418 TRAP Y -0.258 -0.194
370. 20516 TO 20518 TRAP Y -0.194 -0.129
371. 20616 TO 20618 TRAP Y -0.129 -0.065
372. 20716 TO 20718 TRAP Y -0.065 0
373. *
374. * HEADWALL
375. 30101 TO 30114 TRAP Y -0.646 -0.581
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376. 30201 TO 30214 TRAP Y -0.581 -0.517
377. 30301 TO 30314 TRAP Y -0.517 -0.452
378. 30401 TO 30414 TRAP Y -0.452 -0.388
379. 30501 TO 30514 TRAP Y -0.388 -0.323
380. 30601 TO 30614 TRAP Y -0.323 -0.258
381. 30701 TO 30714 TRAP Y -0.258 -0.194
382. 30801 TO 30814 TRAP Y -0.194 -0.129
383. 30901 TO 30914 TRAP Y -0.129 -0.065
384. 31001 TO 31014 TRAP Y -0.065 0
385. *
386. LOAD 6 SAT SOIL - NP
387. ELEMENT LOAD
388. *
389. * ABUTMENT 1
390. * 10
391. 10101 TO 10110 TRAP Y 0.557 0.522
392. 10201 TO 10210 TRAP Y 0.522 0.487
393. 10301 TO 10310 TRAP Y 0.487 0.452
394. 10401 TO 10410 TRAP Y 0.452 0.388
395. 10501 TO 10510 TRAP Y 0.388 0.323
396. 10601 TO 10610 TRAP Y 0.323 0.258
397. 10701 TO 10710 TRAP Y 0.258 0.194
398. 10801 TO 10810 TRAP Y 0.194 0.129
399. 10901 TO 10910 TRAP Y 0.129 0.065
400. 11001 TO 11010 TRAP Y 0.065 0
401. * 9
402. 10111 TO 10113 TRAP Y 0.492 0.457
403. 10211 TO 10213 TRAP Y 0.457 0.422
404. 10311 TO 10313 TRAP Y 0.422 0.388
405. 10411 TO 10413 TRAP Y 0.388 0.323
406. 10511 TO 10513 TRAP Y 0.323 0.258
407. 10611 TO 10613 TRAP Y 0.258 0.194
408. 10711 TO 10713 TRAP Y 0.194 0.129
409. 10811 TO 10813 TRAP Y 0.129 0.065
410. 10911 TO 10913 TRAP Y 0.065 0
411. * 8
412. 10114 10115 TRAP Y 0.428 0.393
413. 10214 10215 TRAP Y 0.393 0.358
414. 10314 10315 TRAP Y 0.358 0.323
415. 10414 10415 TRAP Y 0.323 0.258
416. 10514 10515 TRAP Y 0.258 0.194
417. 10614 10615 TRAP Y 0.194 0.129
418. 10714 10715 TRAP Y 0.129 0.065
419. 10814 10815 TRAP Y 0.065 0
420. * 7
421. 10116 TO 10118 TRAP Y 0.363 0.328
422. 10216 TO 10218 TRAP Y 0.328 0.293
423. 10316 TO 10318 TRAP Y 0.293 0.258
424. 10416 TO 10418 TRAP Y 0.258 0.194
425. 10516 TO 10518 TRAP Y 0.194 0.129
426. 10616 TO 10618 TRAP Y 0.129 0.065
427. 10716 TO 10718 TRAP Y 0.065 0
428. *
429. * ABUTMENT 2
430. * 10
431. 20101 TO 20110 TRAP Y -0.557 -0.522
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432. 20201 TO 20210 TRAP Y -0.522 -0.487
433. 20301 TO 20310 TRAP Y -0.487 -0.452
434. 20401 TO 20410 TRAP Y -0.452 -0.388
435. 20501 TO 20510 TRAP Y -0.388 -0.323
436. 20601 TO 20610 TRAP Y -0.323 -0.258
437. 20701 TO 20710 TRAP Y -0.258 -0.194
438. 20801 TO 20810 TRAP Y -0.194 -0.129
439. 20901 TO 20910 TRAP Y -0.129 -0.065
440. 21001 TO 21010 TRAP Y -0.065 0
441. * 9
442. 20111 TO 20113 TRAP Y -0.492 -0.457
443. 20211 TO 20213 TRAP Y -0.457 -0.422
444. 20311 TO 20313 TRAP Y -0.422 -0.388
445. 20411 TO 20413 TRAP Y -0.388 -0.323
446. 20511 TO 20513 TRAP Y -0.323 -0.258
447. 20611 TO 20613 TRAP Y -0.258 -0.194
448. 20711 TO 20713 TRAP Y -0.194 -0.129
449. 20811 TO 20813 TRAP Y -0.129 -0.065
450. 20911 TO 20913 TRAP Y -0.065 0
451. * 8
452. 20114 20115 TRAP Y -0.428 -0.393
453. 20214 20215 TRAP Y -0.393 -0.358
454. 20314 20315 TRAP Y -0.358 -0.323
455. 20414 20415 TRAP Y -0.323 -0.258
456. 20514 20515 TRAP Y -0.258 -0.194
457. 20614 20615 TRAP Y -0.194 -0.129
458. 20714 20715 TRAP Y -0.129 -0.065
459. 20814 20815 TRAP Y -0.065 0
460. * 7
461. 20116 TO 20118 TRAP Y -0.363 -0.328
462. 20216 TO 20218 TRAP Y -0.328 -0.293
463. 20316 TO 20318 TRAP Y -0.293 -0.258
464. 20416 TO 20418 TRAP Y -0.258 -0.194
465. 20516 TO 20518 TRAP Y -0.194 -0.129
466. 20616 TO 20618 TRAP Y -0.129 -0.065
467. 20716 TO 20718 TRAP Y -0.065 0
468. *
469. * HEADWALL
470. 30101 TO 30114 TRAP Y -0.557 -0.522
471. 30201 TO 30214 TRAP Y -0.522 -0.487
472. 30301 TO 30314 TRAP Y -0.487 -0.452
473. 30401 TO 30414 TRAP Y -0.452 -0.388
474. 30501 TO 30514 TRAP Y -0.388 -0.323
475. 30601 TO 30614 TRAP Y -0.323 -0.258
476. 30701 TO 30714 TRAP Y -0.258 -0.194
477. 30801 TO 30814 TRAP Y -0.194 -0.129
478. 30901 TO 30914 TRAP Y -0.129 -0.065
479. 31001 TO 31014 TRAP Y -0.065 0
480. *
481. LOAD 7 SAT SOIL - IDF
482. ELEMENT LOAD
483. *
484. * ABUTMENT 1
485. * 10
486. 10101 TO 10110 TRAP Y 0.468 0.433
487. 10201 TO 10210 TRAP Y 0.433 0.398
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488. 10301 TO 10310 TRAP Y 0.398 0.363
489. 10401 TO 10410 TRAP Y 0.363 0.328
490. 10501 TO 10510 TRAP Y 0.328 0.293
491. 10601 TO 10610 TRAP Y 0.293 0.258
492. 10701 TO 10710 TRAP Y 0.258 0.194
493. 10801 TO 10810 TRAP Y 0.194 0.129
494. 10901 TO 10910 TRAP Y 0.129 0.065
495. 11001 TO 11010 TRAP Y 0.065 0
496. * 9
497. 10111 TO 10113 TRAP Y 0.403 0.368
498. 10211 TO 10213 TRAP Y 0.368 0.333
499. 10311 TO 10313 TRAP Y 0.333 0.298
500. 10411 TO 10413 TRAP Y 0.298 0.264
501. 10511 TO 10513 TRAP Y 0.264 0.229
502. 10611 TO 10613 TRAP Y 0.229 0.194
503. 10711 TO 10713 TRAP Y 0.194 0.129
504. 10811 TO 10813 TRAP Y 0.129 0.065
505. 10911 TO 10913 TRAP Y 0.065 0
506. * 8
507. 10114 10115 TRAP Y 0.338 0.304
508. 10214 10215 TRAP Y 0.304 0.269
509. 10314 10315 TRAP Y 0.269 0.234
510. 10414 10415 TRAP Y 0.234 0.199
511. 10514 10515 TRAP Y 0.199 0.164
512. 10614 10615 TRAP Y 0.164 0.129
513. 10714 10715 TRAP Y 0.129 0.065
514. 10814 10815 TRAP Y 0.065 0
515. * 7
516. 10116 TO 10118 TRAP Y 0.274 0.239
517. 10216 TO 10218 TRAP Y 0.239 0.204
518. 10316 TO 10318 TRAP Y 0.204 0.169
519. 10416 TO 10418 TRAP Y 0.169 0.134
520. 10516 TO 10518 TRAP Y 0.134 0.099
521. 10616 TO 10618 TRAP Y 0.099 0.065
522. 10716 TO 10718 TRAP Y 0.065 0
523. *
524. * ABUTMENT 2
525. * 10
526. 20101 TO 20110 TRAP Y -0.468 -0.433
527. 20201 TO 20210 TRAP Y -0.433 -0.398
528. 20301 TO 20310 TRAP Y -0.398 -0.363
529. 20401 TO 20410 TRAP Y -0.363 -0.328
530. 20501 TO 20510 TRAP Y -0.328 -0.293
531. 20601 TO 20610 TRAP Y -0.293 -0.258
532. 20701 TO 20710 TRAP Y -0.258 -0.194
533. 20801 TO 20810 TRAP Y -0.194 -0.129
534. 20901 TO 20910 TRAP Y -0.129 -0.065
535. 21001 TO 21010 TRAP Y -0.065 0
536. * 9
537. 20111 TO 20113 TRAP Y -0.403 -0.368
538. 20211 TO 20213 TRAP Y -0.368 -0.333
539. 20311 TO 20313 TRAP Y -0.333 -0.298
540. 20411 TO 20413 TRAP Y -0.298 -0.264
541. 20511 TO 20513 TRAP Y -0.264 -0.229
542. 20611 TO 20613 TRAP Y -0.229 -0.194
543. 20711 TO 20713 TRAP Y -0.194 -0.129
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544. 20811 TO 20813 TRAP Y -0.129 -0.065
545. 20911 TO 20913 TRAP Y -0.065 0
546. * 8
547. 20114 20115 TRAP Y -0.338 -0.304
548. 20214 20215 TRAP Y -0.304 -0.269
549. 20314 20315 TRAP Y -0.269 -0.234
550. 20414 20415 TRAP Y -0.234 -0.199
551. 20514 20515 TRAP Y -0.199 -0.164
552. 20614 20615 TRAP Y -0.164 -0.129
553. 20714 20715 TRAP Y -0.129 -0.065
554. 20814 20815 TRAP Y -0.065 0
555. * 7
556. 20116 TO 20118 TRAP Y -0.274 -0.239
557. 20216 TO 20218 TRAP Y -0.239 -0.204
558. 20316 TO 20318 TRAP Y -0.204 -0.169
559. 20416 TO 20418 TRAP Y -0.169 -0.134
560. 20516 TO 20518 TRAP Y -0.134 -0.099
561. 20616 TO 20618 TRAP Y -0.099 -0.065
562. 20716 TO 20718 TRAP Y -0.065 0
563. *
564. * HEADWALL
565. 30101 TO 30114 TRAP Y -0.468 -0.433
566. 30201 TO 30214 TRAP Y -0.433 -0.398
567. 30301 TO 30314 TRAP Y -0.398 -0.363
568. 30401 TO 30414 TRAP Y -0.363 -0.328
569. 30501 TO 30514 TRAP Y -0.328 -0.293
570. 30601 TO 30614 TRAP Y -0.293 -0.258
571. 30701 TO 30714 TRAP Y -0.258 -0.194
572. 30801 TO 30814 TRAP Y -0.194 -0.129
573. 30901 TO 30914 TRAP Y -0.129 -0.065
574. 31001 TO 31014 TRAP Y -0.065 0
575. *
576. LOAD 8 SURCHARGE
577. ELEMENT LOAD
578. *
579. * ABUTMENT 1
580. 10101 TO 10118 10201 TO 10218 10301 TO 10318 10401 TO 10418 -
581. 10501 TO 10518 10601 TO 10618 10701 TO 10718 10801 TO 10815 -
582. 10901 TO 10913 11001 TO 11010 PRES GZ 0.094
583. * ABUTMENT 2
584. 20101 TO 20118 20201 TO 20218 20301 TO 20318 20401 TO 20418 -
585. 20501 TO 20518 20601 TO 20618 20701 TO 20718 20801 TO 20815 -
586. 20901 TO 20913 21001 TO 21010 PRES GZ -0.094
587. * HEADWALL
588. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
589. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
590. 30901 TO 30914 31001 TO 31014 PRES GX 0.103
591. *
592. LOAD 9 WATER - NP
593. ELEMENT LOAD
594. *
595. * ABUTMENT 1
596. 10101 TO 10118 TRAP Y 0.188 0.125
597. 10201 TO 10218 TRAP Y 0.125 0.063
598. 10301 TO 10318 TRAP Y 0.063 0
599. *
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600. * ABUTMENT 2
601. 20101 TO 20118 TRAP Y -0.188 -0.125
602. 20201 TO 20218 TRAP Y -0.125 -0.063
603. 20301 TO 20318 TRAP Y -0.063 0
604. *
605. * HEADWALL
606. 30101 TO 30114 TRAP Y -0.188 -0.125
607. 30201 TO 30214 TRAP Y -0.125 -0.063
608. 30301 TO 30314 TRAP Y -0.063 0
609. *
610. LOAD 10 PIPE JET - IDF
611. ELEMENT LOAD
612. *
613. * BAFFLE
614. 50504 TO 50511 50604 TO 50611 50704 TO 50711 -
615. 50804 TO 50811 PRES GX 0.326
616. *
617. LOAD 11 EQ - OBE1
618. ELEMENT LOAD
619. * HEADWALL
620. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
621. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
622. 30901 TO 30914 31001 TO 31014 PRES GX 0.018
623. *
624. LOAD 12 EQ - OBE2
625. ELEMENT LOAD
626. * HEADWALL
627. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
628. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
629. 30901 TO 30914 31001 TO 31014 PRES GX 0.018
630. * BAFFLE
631. 50501 TO 50514 50601 TO 50614 50701 TO 50714 50801 TO 50814 -
632. PRES GX 0.012
633. *
634. LOAD 13 EQ - OBE3
635. ELEMENT LOAD
636. * ONLY ON TWO SIDES (ELIMINATE ABUTMENT 2)
637. * ABUTMENT 1
638. 10101 TO 10118 10201 TO 10218 10301 TO 10318 10401 TO 10418 -
639. 10501 TO 10518 10601 TO 10618 10701 TO 10716 10801 TO 10814 -
640. 10901 TO 10912 11001 TO 11010 PRES GZ 0.018
641. * HEADWALL
642. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
643. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
644. 30901 TO 30914 31001 TO 31014 PRES GX 0.018
645. * BAFFLE
646. 50501 TO 50514 50601 TO 50614 50701 TO 50714 50801 TO 50814 -
647. PRES GX 0.012
648. *
649. LOAD 14 EQ - MDE1
650. ELEMENT LOAD
651. * HEADWALL
652. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
653. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
654. 30901 TO 30914 31001 TO 31014 PRES GX 0.042
655. *
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```
656. LOAD 15 EQ - MDE2
657. ELEMENT LOAD
658. * HEADWALL
659. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
660. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
661. 30901 TO 30914 31001 TO 31014 PRES GX 0.042
662. * BAFFLE
663. 50501 TO 50514 50601 TO 50614 50701 TO 50714 50801 TO 50814 -
664. PRES GX 0.028
665. *
666. LOAD 16 EQ - MDE3
667. ELEMENT LOAD
668. * ONLY ON TWO SIDES (ELIMINATE ABUTMENT 2)
669. * ABUTMENT 1
670. 10101 TO 10118 10201 TO 10218 10301 TO 10318 10401 TO 10418 -
671. 10501 TO 10518 10601 TO 10618 10701 TO 10716 10801 TO 10814 -
672. 10901 TO 10912 11001 TO 11010 PRES GZ 0.042
673. * HEADWALL
674. 30101 TO 30114 30201 TO 30214 30301 TO 30314 30401 TO 30414 -
675. 30501 TO 30514 30601 TO 30614 30701 TO 30714 30801 TO 30814 -
676. 30901 TO 30914 31001 TO 31014 PRES GX 0.042
677. * BAFFLE
678. 50501 TO 50514 50601 TO 50614 50701 TO 50714 50801 TO 50814 -
679. PRES GX 0.028
680. *
681. *
682. * FACTORED LOAD COMBINATIONS
683. *
684. LOAD COMB 101 UNUSUAL
685. 1 1.6 5 1.6 8 1.6
686. LOAD COMB 102 EXTREME
687. 1 1.2 5 1.35 11 1.25
688. LOAD COMB 103 USUAL
689. 1 2.2 3 2.2 6 2.2 9 2.2
690. LOAD COMB 104 UNUSUAL
691. 1 1.6 3 1.6 6 1.6 8 1.6 9 1.6
692. LOAD COMB 105 UNUSUAL
693. 1 1.5 3 1.5 6 1.5 9 1.5 11 1.5
694. LOAD COMB 106 UNUSUAL
695. 1 1.5 3 1.5 6 1.5 9 1.5 12 1.5
696. LOAD COMB 107 UNUSUAL
697. 1 1.5 3 1.5 6 1.5 9 1.5 13 1.5
698. LOAD COMB 108 EXTREME
699. 1 1.2 3 1.0 6 1.35 9 1.0 14 1.25
700. LOAD COMB 109 EXTREME
701. 1 1.2 3 1.0 6 1.35 9 1.0 15 1.25
702. LOAD COMB 110 EXTREME
703. 1 1.2 3 1.0 6 1.35 9 1.0 16 1.25
704. LOAD COMB 111 EXTREME
705. 1 1.2 2 0.9 4 1.3 7 1.35 10 1.3
706. *
707. * UNFACTORED LOAD COMBINATIONS
708. *
709. LOAD COMB 201 UNUSUAL
710. 1 1.0 5 1.0 8 1.0
711. LOAD COMB 202 EXTREME
```

```
  712. 1 1.0 5 1.0 11 1.0
  713. LOAD COMB 203 USUAL
  714. 1 1.0 3 1.0 6 1.0 9 1.0
  715. LOAD COMB 204 UNUSUAL
  716. 1 1.0 3 1.0 6 1.0 8 1.0 9 1.0
  717. LOAD COMB 205 UNUSUAL
  718. 1 1.0 3 1.0 6 1.0 9 1.0 11 1.0
  719. LOAD COMB 206 UNUSUAL
  720. 1 1.0 3 1.0 6 1.0 9 1.0 12 1.0
  721. LOAD COMB 207 UNUSUAL
  722. 1 1.0 3 1.0 6 1.0 9 1.0 13 1.0
  723. LOAD COMB 208 EXTREME
  724. 1 1.0 3 1.0 6 1.0 9 1.0 14 1.0
  725. LOAD COMB 209 EXTREME
  726. 1 1.0 3 1.0 6 1.0 9 1.0 15 1.0
  727. LOAD COMB 210 EXTREME
  728. 1 1.0 3 1.0 6 1.0 9 1.0 16 1.0
  729. LOAD COMB 211 EXTREME
  730. 1 1.0 2 1.0 4 1.0 7 1.0 10 1.0
  731. *
  732. *
  733. PERFORM ANALYSIS


           P R O B L E M   S T A T I S T I C S
           -----------------------------------

     NUMBER OF JOINTS        828  NUMBER OF MEMBERS       0
     NUMBER OF PLATES        776  NUMBER OF SOLIDS        0
     NUMBER OF SURFACES        0  NUMBER OF SUPPORTS    285

           Using 64-bit analysis engine.

           SOLVER USED IS THE IN-CORE ADVANCED MATH SOLVER

   TOTAL      PRIMARY LOAD CASES =    16, TOTAL DEGREES OF FREEDOM =    4113
   TOTAL LOAD COMBINATION  CASES =    22  SO FAR.


  **NOTE-Tension/Compression converged after  1 iterations, Case=    1



  **NOTE-Tension/Compression converged after  1 iterations, Case=    2


*** LOAD CASE     3 -- START ITERATION NO.   2

 ***WARNING -  DIAGONAL DECAY AT JOINT  40903   DIRECTION = FY
  K-MATRIX DIAG=  4.0227397E+04 L-MATRIX DIAG=  2.6080790E-09 EQN NO    4055

    **NOTE** EXCESSIVE LOSS OF SIGNIFICANT DIGITS GREATER
             THAN 9 IS DETECTED DURING DECOMPOSITION.
```

```
  **NOTE-Tension/Compression converged after  2 iterations, Case=    3


*** LOAD CASE     4 -- START ITERATION NO.   2

 ***WARNING -  DIAGONAL DECAY AT JOINT  40903   DIRECTION = FY
  K-MATRIX DIAG=  4.0227397E+04 L-MATRIX DIAG=  2.6080790E-09 EQN NO    4055

  **NOTE-Tension/Compression converged after  2 iterations, Case=    4


  **NOTE-Tension/Compression converged after  1 iterations, Case=    5


  **NOTE-Tension/Compression converged after  1 iterations, Case=    6


  **NOTE-Tension/Compression converged after  1 iterations, Case=    7


  **NOTE-Tension/Compression converged after  1 iterations, Case=    8


  **NOTE-Tension/Compression converged after  1 iterations, Case=    9


  **NOTE-Tension/Compression converged after  1 iterations, Case=   10


  **NOTE-Tension/Compression converged after  1 iterations, Case=   11


  **NOTE-Tension/Compression converged after  1 iterations, Case=   12


  **NOTE-Tension/Compression converged after  1 iterations, Case=   13


  **NOTE-Tension/Compression converged after  1 iterations, Case=   14
```

**NOTE-Tension/Compression converged after 1 iterations, Case= 16

```
  734. LOAD LIST 101 TO 111
  735. UNIT INCHES KIP
  736. PRINT ELEMENT STRESSES LIST 10118 40118 50501 50814
ELEMENT  STRESSES LIST     10118
```

TYPE VI BASIN -- PAGE NO. 17

ELEMENT STRESSES FORCE,LENGTH UNITS= KIP INCH

STRESS = FORCE/UNIT WIDTH/THICK, MOMENT = FORCE-LENGTH/UNIT WIDTH

ELEMENT	LOAD	SQX / VONT / TRESCAT	SQY / VONB / TRESCAB	MX / SX	MY / SY	MXY / SXY
10118	101	0.00	0.02	-1.13	-9.99	0.78
		0.21	0.14	-0.00	-0.04	0.01
		0.22	0.15			
	TOP : SMAX=	-0.02	SMIN= -0.22	TMAX= 0.10	ANGLE= 5.7	
	BOTT: SMAX=	0.15	SMIN= 0.02	TMAX= 0.07	ANGLE=-86.1	
	102	0.00	0.01	-0.76	-6.69	0.73
		0.15	0.09	-0.00	-0.03	0.00
		0.15	0.10			
	TOP : SMAX=	-0.01	SMIN= -0.15	TMAX= 0.07	ANGLE= 7.4	
	BOTT: SMAX=	0.10	SMIN= 0.01	TMAX= 0.04	ANGLE= 90.0	
	103	0.00	0.02	-1.24	-10.69	1.20
		0.24	0.14	-0.00	-0.05	0.01
		0.25	0.15			
	TOP : SMAX=	-0.02	SMIN= -0.25	TMAX= 0.12	ANGLE= 8.3	
	BOTT: SMAX=	0.15	SMIN= 0.02	TMAX= 0.07	ANGLE=-85.0	
	104	0.00	0.02	-1.07	-9.26	0.66
		0.20	0.13	-0.00	-0.04	0.01
		0.21	0.13			
	TOP : SMAX=	-0.02	SMIN= -0.21	TMAX= 0.10	ANGLE= 6.0	
	BOTT: SMAX=	0.13	SMIN= 0.02	TMAX= 0.06	ANGLE=-87.8	
	105	0.00	0.01	-0.85	-7.38	0.83
		0.17	0.10	-0.00	-0.03	0.01
		0.17	0.10			
	TOP : SMAX=	-0.01	SMIN= -0.17	TMAX= 0.08	ANGLE= 8.3	
	BOTT: SMAX=	0.10	SMIN= 0.01	TMAX= 0.05	ANGLE= 90.0	
	106	0.00	0.01	-0.85	-7.40	0.84
		0.17	0.10	-0.00	-0.03	0.01
		0.18	0.10			
	TOP : SMAX=	-0.01	SMIN= -0.18	TMAX= 0.08	ANGLE= 8.4	
	BOTT: SMAX=	0.10	SMIN= 0.01	TMAX= 0.05	ANGLE= 90.0	
	107	0.00	0.01	-0.88	-7.58	0.80
		0.17	0.10	-0.00	-0.03	0.01
		0.18	0.11			
	TOP : SMAX=	-0.02	SMIN= -0.18	TMAX= 0.08	ANGLE= 7.9	
	BOTT: SMAX=	0.11	SMIN= 0.01	TMAX= 0.05	ANGLE= 90.0	
	108	0.00	0.01	-0.73	-6.33	0.68
		0.14	0.08	-0.00	-0.03	0.01
		0.15	0.09			
	TOP : SMAX=	-0.01	SMIN= -0.15	TMAX= 0.07	ANGLE= 7.9	
	BOTT: SMAX=	0.09	SMIN= 0.01	TMAX= 0.04	ANGLE= 90.0	
	109	0.00	0.01	-0.73	-6.36	0.70
		0.14	0.08	-0.00	-0.03	0.01
		0.15	0.09			
	TOP : SMAX=	-0.01	SMIN= -0.15	TMAX= 0.07	ANGLE= 8.0	
	BOTT: SMAX=	0.09	SMIN= 0.01	TMAX= 0.04	ANGLE= 90.0	

TYPE VI BASIN -- PAGE NO. 18

ELEMENT STRESSES FORCE,LENGTH UNITS= KIP INCH

STRESS = FORCE/UNIT WIDTH/THICK, MOMENT = FORCE-LENGTH/UNIT WIDTH

ELEMENT	LOAD	SQX VONT TRESCAT	SQY VONB TRESCAB	MX SX	MY SY	MXY SXY
	110	0.00	0.01	-0.77	-6.72	0.61
		0.15	0.09	-0.00	-0.03	0.01
		0.15	0.10			
	TOP : SMAX=	-0.01	SMIN=	-0.15	TMAX=	0.07 ANGLE= 6.9
	BOTT: SMAX=	0.10	SMIN=	0.01	TMAX=	0.04 ANGLE= 90.0
	111	-0.00	0.01	-0.64	-5.49	0.90
		0.13	0.07	-0.00	-0.03	0.01
		0.14	0.07			
	TOP : SMAX=	-0.01	SMIN=	-0.14	TMAX=	0.06 ANGLE= 11.7
	BOTT: SMAX=	0.07	SMIN=	0.01	TMAX=	0.03 ANGLE= 90.0
40118	101	-0.00	-0.04	-0.34	-8.79	0.84
		0.16	0.16	0.00	0.00	0.00
		0.16	0.16			
	TOP : SMAX=	-0.00	SMIN=	-0.16	TMAX=	0.08 ANGLE= 5.6
	BOTT: SMAX=	0.16	SMIN=	0.00	TMAX=	0.08 ANGLE=-84.4
	102	-0.00	-0.03	-0.22	-5.70	0.67
		0.11	0.11	0.00	0.00	0.00
		0.11	0.11			
	TOP : SMAX=	-0.00	SMIN=	-0.11	TMAX=	0.05 ANGLE= 6.9
	BOTT: SMAX=	0.11	SMIN=	0.00	TMAX=	0.05 ANGLE=-83.1
	103	-0.00	-0.04	-0.35	-9.25	1.06
		0.17	0.17	0.00	0.00	0.00
		0.17	0.17			
	TOP : SMAX=	-0.00	SMIN=	-0.17	TMAX=	0.08 ANGLE= 6.7
	BOTT: SMAX=	0.17	SMIN=	0.00	TMAX=	0.08 ANGLE=-83.3
	104	-0.00	-0.03	-0.32	-8.37	0.72
		0.15	0.15	0.00	0.00	0.00
		0.16	0.16			
	TOP : SMAX=	-0.00	SMIN=	-0.16	TMAX=	0.08 ANGLE= 5.1
	BOTT: SMAX=	0.16	SMIN=	0.00	TMAX=	0.08 ANGLE=-84.9
	105	-0.00	-0.03	-0.24	-6.37	0.73
		0.12	0.12	0.00	0.00	0.00
		0.12	0.12			
	TOP : SMAX=	-0.00	SMIN=	-0.12	TMAX=	0.06 ANGLE= 6.7
	BOTT: SMAX=	0.12	SMIN=	0.00	TMAX=	0.06 ANGLE=-83.3
	106	-0.00	-0.03	-0.24	-6.37	0.74
		0.12	0.12	0.00	0.00	0.00
		0.12	0.12			
	TOP : SMAX=	-0.00	SMIN=	-0.12	TMAX=	0.06 ANGLE= 6.8
	BOTT: SMAX=	0.12	SMIN=	0.00	TMAX=	0.06 ANGLE=-83.2
	107	-0.00	-0.03	-0.25	-6.63	0.71
		0.12	0.12	0.00	0.00	0.00
		0.12	0.12			
	TOP : SMAX=	-0.00	SMIN=	-0.12	TMAX=	0.06 ANGLE= 6.3
	BOTT: SMAX=	0.12	SMIN=	0.00	TMAX=	0.06 ANGLE=-83.7

TYPE VI BASIN -- PAGE NO. 19

ELEMENT STRESSES FORCE,LENGTH UNITS= KIP INCH

STRESS = FORCE/UNIT WIDTH/THICK, MOMENT = FORCE-LENGTH/UNIT WIDTH

ELEMENT	LOAD	SQX / VONT / TRESCAT	SQY / VONB / TRESCAB	MX / SX	MY / SY	MXY / SXY
	108	-0.00	-0.02	-0.21	-5.50	0.61
		0.10	0.10	0.00	0.00	0.00
		0.10	0.10			
	TOP : SMAX=	-0.00	SMIN= -0.10	TMAX= 0.05	ANGLE= 90.0	
	BOTT: SMAX=	0.10	SMIN= 0.00	TMAX= 0.05	ANGLE= 90.0	
	109	-0.00	-0.02	-0.21	-5.50	0.63
		0.10	0.10	0.00	0.00	0.00
		0.10	0.10			
	TOP : SMAX=	-0.00	SMIN= -0.10	TMAX= 0.05	ANGLE= 90.0	
	BOTT: SMAX=	0.10	SMIN= 0.00	TMAX= 0.05	ANGLE= 90.0	
	110	-0.00	-0.02	-0.23	-5.99	0.57
		0.11	0.11	0.00	0.00	0.00
		0.11	0.11			
	TOP : SMAX=	-0.00	SMIN= -0.11	TMAX= 0.05	ANGLE= 5.6	
	BOTT: SMAX=	0.11	SMIN= 0.00	TMAX= 0.05	ANGLE=-84.4	
	111	-0.01	-0.03	-0.18	-4.29	0.65
		0.08	0.08	0.00	0.00	0.00
		0.08	0.08			
	TOP : SMAX=	-0.00	SMIN= -0.08	TMAX= 0.04	ANGLE= 90.0	
	BOTT: SMAX=	0.08	SMIN= 0.00	TMAX= 0.04	ANGLE= 90.0	
50501	101	0.00	-0.00	-0.14	0.02	-0.01
		0.06	0.05	-0.05	-0.01	-0.02
		0.06	0.05			
	TOP : SMAX=	-0.01	SMIN= -0.06	TMAX= 0.03	ANGLE= 90.0	
	BOTT: SMAX=	-0.01	SMIN= -0.05	TMAX= 0.02	ANGLE= 90.0	
	102	-0.00	-0.00	-0.11	0.02	0.01
		0.04	0.03	-0.03	-0.01	-0.01
		0.04	0.03			
	TOP : SMAX=	-0.00	SMIN= -0.04	TMAX= 0.02	ANGLE= 90.0	
	BOTT: SMAX=	-0.00	SMIN= -0.03	TMAX= 0.01	ANGLE= 90.0	
	103	-0.01	-0.01	0.59	0.19	0.12
		0.03	0.07	-0.04	-0.01	-0.02
		0.03	0.08			
	TOP : SMAX=	0.00	SMIN= -0.03	TMAX= 0.01	ANGLE= 90.0	
	BOTT: SMAX=	-0.01	SMIN= -0.08	TMAX= 0.03	ANGLE= 90.0	
	104	-0.00	-0.00	0.42	0.14	0.06
		0.04	0.07	-0.05	-0.01	-0.01
		0.04	0.07			
	TOP : SMAX=	0.00	SMIN= -0.03	TMAX= 0.02	ANGLE= 90.0	
	BOTT: SMAX=	-0.01	SMIN= -0.07	TMAX= 0.03	ANGLE= 90.0	
	105	-0.00	-0.00	0.40	0.13	0.08
		0.02	0.05	-0.03	-0.01	-0.01
		0.02	0.05			
	TOP : SMAX=	0.00	SMIN= -0.02	TMAX= 0.01	ANGLE= 90.0	
	BOTT: SMAX=	-0.01	SMIN= -0.05	TMAX= 0.02	ANGLE= 90.0	

TYPE VI BASIN -- PAGE NO. 20

ELEMENT STRESSES FORCE,LENGTH UNITS= KIP INCH

STRESS = FORCE/UNIT WIDTH/THICK, MOMENT = FORCE-LENGTH/UNIT WIDTH

ELEMENT	LOAD	SQX	SQY	MX	MY	MXY
		VONT	VONB	SX	SY	SXY
		TRESCAT	TRESCAB			
	106	-0.00	-0.00	0.64	0.15	0.09
		0.01	0.06	-0.03	-0.01	-0.01
		0.01	0.06			
	TOP : SMAX=	0.01	SMIN=	-0.01	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	-0.01	SMIN=	-0.06	TMAX=	0.03 ANGLE= 90.0
	107	-0.00	-0.00	0.68	0.15	0.09
		0.01	0.06	-0.03	-0.01	-0.01
		0.01	0.06			
	TOP : SMAX=	0.01	SMIN=	-0.01	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	-0.01	SMIN=	-0.06	TMAX=	0.03 ANGLE= 90.0
	108	-0.00	-0.00	0.25	0.09	0.05
		0.02	0.04	-0.03	-0.01	-0.01
		0.02	0.04			
	TOP : SMAX=	0.00	SMIN=	-0.02	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	-0.01	SMIN=	-0.04	TMAX=	0.02 ANGLE= 90.0
	109	-0.00	-0.00	0.70	0.13	0.07
		0.01	0.06	-0.03	-0.01	-0.01
		0.01	0.06			
	TOP : SMAX=	0.01	SMIN=	-0.01	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	-0.01	SMIN=	-0.06	TMAX=	0.03 ANGLE= 90.0
	110	-0.00	-0.00	0.79	0.14	0.07
		0.01	0.06	-0.02	-0.00	-0.01
		0.01	0.06			
	TOP : SMAX=	0.01	SMIN=	-0.00	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	-0.01	SMIN=	-0.06	TMAX=	0.03 ANGLE= 90.0
	111	-0.02	-0.01	5.37	0.62	0.34
		0.20	0.23	-0.02	-0.00	-0.01
		0.21	0.24			
	TOP : SMAX=	0.21	SMIN=	0.02	TMAX=	0.09 ANGLE= 2.6
	BOTT: SMAX=	-0.03	SMIN=	-0.24	TMAX=	0.11 ANGLE=-84.7
50814	101	0.00	-0.00	0.15	-0.01	0.02
		0.03	0.04	-0.04	-0.00	0.00
		0.03	0.04			
	TOP : SMAX=	-0.00	SMIN=	-0.03	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	-0.00	SMIN=	-0.04	TMAX=	0.02 ANGLE= 90.0
	102	0.00	-0.00	0.02	-0.01	0.00
		0.01	0.01	-0.01	0.00	0.00
		0.01	0.02			
	TOP : SMAX=	0.00	SMIN=	-0.01	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	0.00	SMIN=	-0.01	TMAX=	0.01 ANGLE= 90.0
	103	0.01	-0.01	1.64	0.00	-0.10
		0.05	0.08	-0.02	-0.00	0.00
		0.05	0.08			
	TOP : SMAX=	0.05	SMIN=	-0.00	TMAX=	0.03 ANGLE= 90.0
	BOTT: SMAX=	0.00	SMIN=	-0.08	TMAX=	0.04 ANGLE= 90.0

TYPE VI BASIN -- PAGE NO. 21

ELEMENT STRESSES FORCE,LENGTH UNITS= KIP INCH

STRESS = FORCE/UNIT WIDTH/THICK, MOMENT = FORCE-LENGTH/UNIT WIDTH

ELEMENT	LOAD	SQX	SQY	MX	MY	MXY
		VONT	VONB	SX	SY	SXY
		TRESCAT	TRESCAB			
	104	0.00	-0.00	1.35	0.01	-0.05
		0.02	0.09	-0.04	-0.00	-0.00
		0.02	0.09			
	TOP : SMAX=	0.02	SMIN=	-0.00	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	-0.00	SMIN=	-0.09	TMAX=	0.04 ANGLE= 90.0
	105	0.00	-0.00	1.13	0.00	-0.07
		0.04	0.06	-0.01	-0.00	0.00
		0.04	0.06			
	TOP : SMAX=	0.04	SMIN=	-0.00	TMAX=	0.02 ANGLE= 90.0
	BOTT: SMAX=	0.00	SMIN=	-0.06	TMAX=	0.03 ANGLE= 90.0
	106	0.00	-0.00	1.35	0.02	-0.06
		0.05	0.07	-0.01	-0.00	0.00
		0.05	0.07			
	TOP : SMAX=	0.05	SMIN=	0.00	TMAX=	0.02 ANGLE= 90.0
	BOTT: SMAX=	-0.00	SMIN=	-0.07	TMAX=	0.03 ANGLE= 90.0
	107	0.00	-0.00	1.23	0.01	-0.07
		0.04	0.06	-0.01	-0.00	0.00
		0.04	0.06			
	TOP : SMAX=	0.04	SMIN=	0.00	TMAX=	0.02 ANGLE= 90.0
	BOTT: SMAX=	-0.00	SMIN=	-0.06	TMAX=	0.03 ANGLE= 90.0
	108	0.00	-0.00	0.78	-0.00	-0.04
		0.02	0.04	-0.01	-0.00	0.00
		0.02	0.04			
	TOP : SMAX=	0.02	SMIN=	-0.00	TMAX=	0.01 ANGLE= 90.0
	BOTT: SMAX=	0.00	SMIN=	-0.04	TMAX=	0.02 ANGLE= 90.0
	109	0.00	-0.00	1.21	0.04	-0.03
		0.04	0.06	-0.01	-0.00	0.00
		0.04	0.06			
	TOP : SMAX=	0.04	SMIN=	0.00	TMAX=	0.02 ANGLE= 90.0
	BOTT: SMAX=	-0.00	SMIN=	-0.06	TMAX=	0.03 ANGLE= 90.0
	110	0.00	-0.00	0.99	0.02	-0.04
		0.03	0.05	-0.01	-0.00	0.00
		0.03	0.05			
	TOP : SMAX=	0.03	SMIN=	0.00	TMAX=	0.02 ANGLE= 90.0
	BOTT: SMAX=	-0.00	SMIN=	-0.05	TMAX=	0.03 ANGLE= 90.0
	111	0.02	-0.01	6.43	0.37	-0.06
		0.26	0.26	-0.00	-0.00	0.00
		0.27	0.27			
	TOP : SMAX=	0.27	SMIN=	0.02	TMAX=	0.13 ANGLE= -0.4
	BOTT: SMAX=	-0.02	SMIN=	-0.27	TMAX=	0.13 ANGLE= 89.3

```
   TYPE VI BASIN                                          -- PAGE NO.   22


         **** MAXIMUM STRESSES AMONG SELECTED PLATES AND CASES ****
            MAXIMUM       MINIMUM       MAXIMUM       MAXIMUM       MAXIMUM
           PRINCIPAL     PRINCIPAL        SHEAR       VONMISES       TRESCA
            STRESS        STRESS        STRESS        STRESS        STRESS

         2.664653E-01 -2.693506E-01  1.268163E-01  2.618456E-01  2.693506E-01
PLATE NO.   50814         50814         50814         50814         50814
CASE  NO.     111           111           111           111           111


  ********************END OF ELEMENT FORCES********************


  737. *
  738. LOAD LIST 201 TO 211
  739. PRINT JOINT DISPL LIST 2110 21106 40905
JOINT    DISPL    LIST     2110
```

JOINT DISPLACEMENT (INCH RADIANS) STRUCTURE TYPE = SPACE

JOINT	LOAD	X-TRANS	Y-TRANS	Z-TRANS	X-ROTAN	Y-ROTAN	Z-ROTAN
2110	201	0.04617	-0.04646	0.00147	0.00002	-0.00001	-0.00038
	202	0.04438	-0.04474	0.00072	0.00001	0.00000	-0.00037
	203	-1.675E+10	1.675E+10	-3.617E-04	-4.991E-07	6.311E-06	1.396E+08
	204	-1.675E+10	1.675E+10	3.672E-04	7.466E-06	-3.770E-06	1.396E+08
	205	-1.675E+10	1.675E+10	-3.782E-04	-7.706E-07	5.929E-06	1.396E+08
	206	-1.675E+10	1.675E+10	-3.952E-04	-7.705E-07	6.910E-06	1.396E+08
	207	-1.675E+10	1.675E+10	1.799E-04	3.400E-06	3.218E-06	1.396E+08
	208	-1.675E+10	1.675E+10	-4.002E-04	-1.133E-06	5.420E-06	1.396E+08
	209	-1.675E+10	1.675E+10	-4.399E-04	-1.132E-06	7.707E-06	1.396E+08
	210	-1.675E+10	1.675E+10	9.020E-04	8.598E-06	-9.060E-07	1.396E+08
	211	-3.951E+10	3.951E+10	-2.371E-03	-1.417E-05	3.599E-05	3.292E+08
21106	201	0.04977	-0.00017	0.00000	0.00000	0.00000	-0.00040
	202	0.04689	-0.00018	0.00000	0.00000	0.00000	-0.00038
	203	-1.675E+10	-1.229E-04	-6.407E-05	4.435E-08	-1.024E-05	1.396E+08
	204	-1.675E+10	-1.129E-04	-6.407E-05	4.435E-08	-1.024E-05	1.396E+08
	205	-1.675E+10	-1.215E-04	-6.407E-05	4.435E-08	-1.024E-05	1.396E+08
	206	-1.675E+10	-1.210E-04	-6.407E-05	4.435E-08	-1.024E-05	1.396E+08
	207	-1.675E+10	-1.208E-04	-2.852E-05	1.089E-08	-6.995E-06	1.396E+08
	208	-1.675E+10	-1.197E-04	-6.407E-05	4.435E-08	-1.024E-05	1.396E+08
	209	-1.675E+10	-1.185E-04	-6.407E-05	4.435E-08	-1.024E-05	1.396E+08
	210	-1.675E+10	-1.180E-04	1.888E-05	-3.373E-08	-2.667E-06	1.396E+08
	211	-3.951E+10	-5.097E-05	-1.510E-04	1.173E-07	-2.408E-05	3.292E+08
40905	201	0.03689	-0.04290	0.00017	0.00000	0.00000	-0.00038
	202	0.03581	-0.04119	0.00012	0.00000	0.00000	-0.00037
	203	-1.340E+10	1.507E+10	-6.772E-04	8.382E-06	9.976E-07	1.396E+08
	204	-1.340E+10	1.507E+10	-6.256E-04	9.091E-06	-1.437E-07	1.396E+08
	205	-1.340E+10	1.507E+10	-6.790E-04	8.365E-06	9.801E-07	1.396E+08
	206	-1.340E+10	1.507E+10	-6.795E-04	8.361E-06	3.383E-06	1.396E+08
	207	-1.340E+10	1.507E+10	-3.205E-04	1.048E-05	6.467E-06	1.396E+08
	208	-1.340E+10	1.507E+10	-6.813E-04	8.342E-06	9.569E-07	1.396E+08
	209	-1.340E+10	1.507E+10	-6.826E-04	8.333E-06	6.564E-06	1.396E+08
	210	-1.340E+10	1.507E+10	1.551E-04	1.328E-05	1.376E-05	1.396E+08
	211	-3.161E+10	3.556E+10	-1.785E-03	1.500E-05	6.086E-05	3.292E+08

********** END OF LATEST ANALYSIS RESULT **********

740. FINISH

TYPE VI BASIN -- PAGE NO. 24

********** END OF THE STAAD.Pro RUN **********

**** DATE= FEB 5,2021 TIME= 10:16:29 ****

****
* For technical assistance on STAAD.Pro, please visit *
* http://www.bentley.com/en/support/ *
* *
* Details about additional assistance from *
* Bentley and Partners can be found at program menu *
* Help->Technical Support *
* *
* Copyright (c) Bentley Systems, Inc. *
* http://www.bentley.com *
****

Appendix K
Outlet Hydraulics

AYRES

Air Vent Requirements for Low-Level Outlet

References:

Handbook of Applied Hydraulics by Davis and Sorenson

Diameter, D	2.5 ft
Cd	0.80
Opening, O	80%
Head, H	50.4 ft
Area, A	3.9 sf
Angle, θ	1.20
Topwidth, T	2.3 ft

1. Determine depth of water (Y) at vena contracta:

Y = D x Cd X O (square conduits) OR Y = A/T (for circular conduits) — Y = 1.7 ft

2. Calculate spouting velocity (V)

V = sqrt(2GH) — V = 57 fps

3. Calculate discharge of water (Qw)

Qw = V x Y x W — Qw = 224 cfs

4. Determine Froude number (F) at vena contracta

F = V/sqrt(G x Y) = 57/sqrt(32.2 x 1) = 10 — F = 7.7

5. Evaluate F - 1 — F-1 = 6.7

6. Determine β on curves A or B using F - 1

Hydraulic Jump in Conduit	Curve A, β	0.10
No Hydraulic Jump	Curve B, β	0.23

Chosen β = 0.23

7. Determine air demand (Qa)

Qa = Qw x β — Qa = 51 cfs

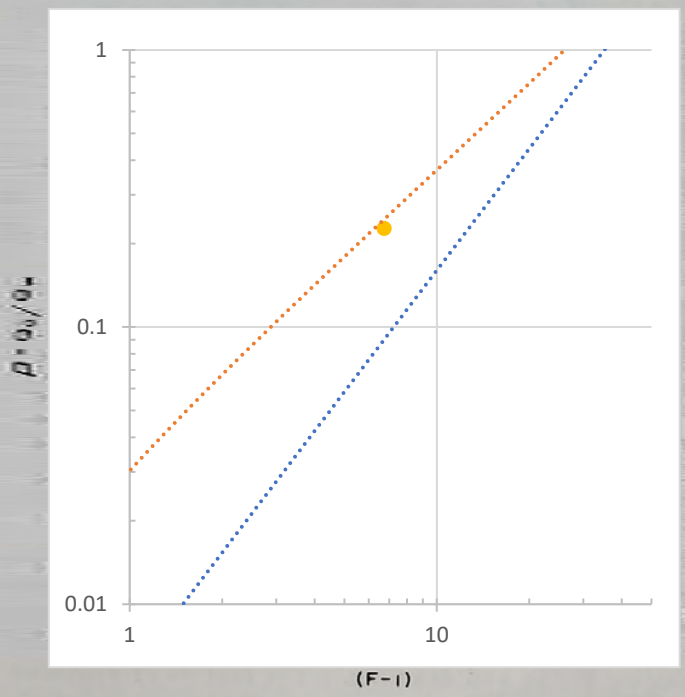
8. Determine vent area based on allowed air velocity

Allowable air vent velocity = 150 fps

Vent Area, Ad	0.34 sf
Vent Diameter, d	7.9 in

Use 8" vent pipe

AIR DEMAND DESIGN CRITERIA



TERMINOLOGY

Y = Water depth in feet at vena contracta
D = Height of gate opening in feet
C_d = Discharge coefficient
V = Velocity in feet per second
G = Gravitational acceleration = 32.2 ft./sec.²
H = Head across valve in feet (For short conduits with fairly small losses use the difference in head from the reservoir surface to the top of the vena contracta)
W = Gate width
Q_w = Water discharge in cubic feet per second.
F = Froude number
Q_a = Estimated air demand in cubic feet per second.
β = Ratio of estimated air demand to water flow.

Curve A - Kalinske and Robertson tests - Use where a hydraulic jump forms in the downstream conduit.

Curve B - U.S. Army Engineers - Suggested curve for free surface flow discharges. (No jump)

AIR VENT SIZE DETERMINATION PROCEDURE:

1. Determine depth of water (Y) at vena contracta
   $Y = D(C_d)$(gate opening)
   (See Note 1.)
2. Calculate spouting velocity (V)
   $V = \sqrt{2GH}$
3. Calculate discharge of water (Q_w)
   $Q_w = (V)(Y)(W)$
4. Determine Froude number (F) at vena contracta
   $F = V/\sqrt{GY}$
5. Evaluate (F - 1)
6. Determine β on curves A or B using (F - 1)
7. Determine air demand (Q_a)
   $Q_a = Q_w \beta$
8. Determine vent area based on allowed air velocity. (See Note 2.)
9. Check losses in vent to be sure they are less than 5 feet of water head.

NOTES

Note 1. Use 0.8 for 45° gate bottom and 0.6 for sharp bottom lip. Assume gate opening is 0.8.

Note 2. Allowable air velocities range from 150 fps to 300 fps.

FIG. 52. Air-demand design criteria. (*Based on data from Engineer Manual EM 1110-2-1602, Department of the Army, Office of the Chief of Engineers.*)

From "Handbook of Applied Hydraulics" by Davis and Sorenson

USBR Type VI Impact Structure dimensions for pipe discharge at IDF pool elevation of 9,900.4 ft. Flow area for the 30-in pipe is reduced to account for CIPP lining, which is estimated to be 1 inch thick.

HY-8 Energy Dissipation Report

External Energy Dissipator

Parameter	Value	Units
Select Culvert and Flow		
Crossing	With Bend Loss IDF Pool CIPP	
Culvert	30-in Reduced with CIPP (With Bend Loss)	
Flow	146.10	cfs
Culvert Data		
Culvert Width (including multiple barrels)	2.3	ft
Culvert Height	2.3	ft
Outlet Depth	2.33	ft
Outlet Velocity	34.18	ft/s
Froude Number	3.94	
Tailwater Depth	3.40	ft
Tailwater Velocity	2.65	ft/s
Tailwater Slope (SO)	0.0331	
External Dissipator Data		
External Dissipator Category	Streambed Level Structures	
External Dissipator Type	USBR Type VI	
Restrictions		
TailWater	DESIRABLE	
Special Limitations	Q < 400 cfs	
Special Limitations	VO < 50 fps	
Results		
Basin Height (H1)	9.167	ft
Basin Length (L)	16.000	ft
Basin Width (WB)	12.000	ft
Basin Top (W1)	0.917	ft
Exit Depth (Yc)	1.667	ft
Exit Velocity (Vc)	7.303	ft/s
Exit Energy (HB)	6.584	ft
Baffle Dimensions		
Height to Bottom (H3)	2.000	ft
Height baffle (H2)	4.500	ft
Thickness Top (T1)	0.667	ft
Thickness Baffle (T3)	0.833	ft
Miscellaneous Dimensions		
Thickness Bottom (T2)	0.833	ft
Thickness Back (T4)	0.750	ft
Thickness Fillets (T5)	0.333	ft
Length to Baffle (L1)	6.833	ft
Length to Sill (L2)	9.167	ft
End Sill		
End Sill Height Weir (H3)	2.000	ft
End Sill Height Top (H4)	5.000	ft
End Sill Wing Wall (W2)	3.000	ft

Sizing of 45-degree bevel at entrance to 30-inch outlet pipe

From FHWA Hydraulic Design of Highway Culverts (HDS-5), Third Edition, April 2012

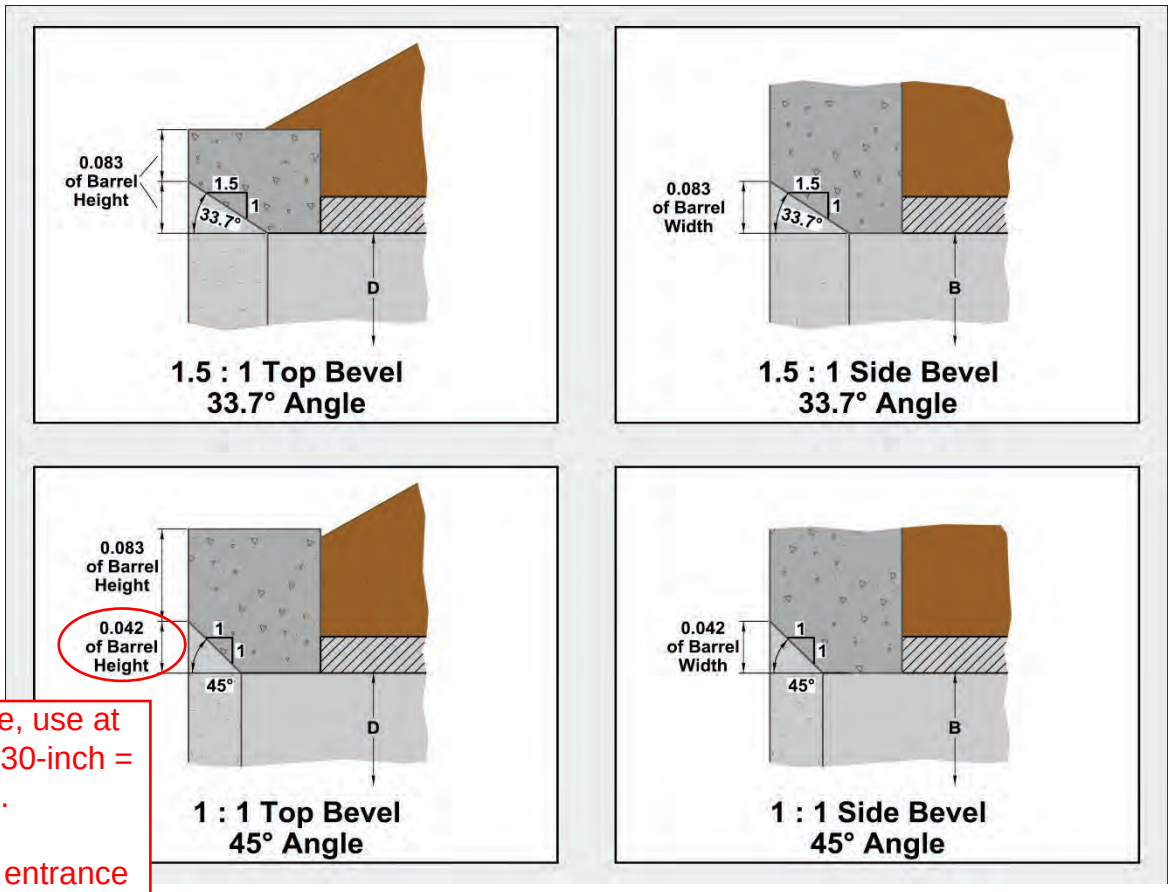


Figure 3.3. Beveled edges.

For 30-inch pipe, use at least a 0.042 x 30-inch = 1.26-inch bevel.

Design conduit entrance with 1.5-inch, 45-degree bevel.

Inlet Depression. Inlet depression is created by constructing the entrance inlet below the streambed. The amount of inlet depression is defined as the depth from the natural streambed at the face to the inlet invert. The inlet control equations or nomographs provide the depth of headwater above the inlet invert required to convey a given discharge through the inlet. This relationship remains constant regardless of the elevation of the inlet invert. If the entrance end of the culvert is constructed below the streambed, more head can be exerted on the inlet for the same headwater elevation.

Two methods of constructing inlet depression at the entrance end of a culvert are shown in Figures 3.5 and Figure 3.6. Figure 3.5 depicts the use of an approach apron with the fill retained by wingwalls. Paving the apron is desirable. Figure 3.6 shows a sump constructed upstream of the culvert face. Usually the sump is paved, but for small depressions, an unpaved excavation that is lined with riprap to prevent headcutting may be adequate.

3.1.4 Outlet Control

Figure 3.7 illustrates the types of outlet control flow. The USGS flow type depends on the submergence of the inlet and outlet ends of the culvert. In all cases, the control section is at the outlet end of the culvert or further downstream. For the partly full flow situations, the flow in the barrel is subcritical.

Rating Curve Development for Hogchute Dam Low-Level Outlet Pipe - Gate Fully Open

References:

Bureau of Reclamation (Reclamation) (1987). Design of Small Dams, Third Edition.
US Department of the Interior, Washington, DC.

D (ft)	2.333333	(assume 30-inch pipe reduced in diameter by 1-inch thick CIPP lining)
Theta	0.038869	(radians)
Ke	0.2	(entrance loss coefficient, beveled)
Kv	1	(exit loss coefficient)
Kb	0.27	(bend loss coefficient, 68 degrees)
n	0.01	(Manning's n for CIPP lining)
L (ft)	288	
r (ft)	0.583333	(hydraulic radius, pipe flowing full)
Inlet (ft)	9851	(inlet invert elevation, ft NAVD88)

HW (ft)	H (ft)	H/D	$Q/D^{5/2}$	Q (cfs)[1]	Notes
9900.4	49.4	21.17143	17.62504	146.5791	IDF Pool
9900	49	21	17.56443	146.075	
9899.5	48.5	20.78571	17.48837	145.4425	
9899	48	20.57143	17.41198	144.8072	
9898.5	47.5	20.35714	17.33525	144.1691	
9898	47	20.14286	17.25818	143.5281	
9897.5	46.5	19.92857	17.18077	142.8843	
9897	46	19.71429	17.103	142.2376	
9896.5	45.5	19.5	17.02488	141.5879	
9896	45	19.28571	16.9464	140.9352	
9895.5	44.5	19.07143	16.86756	140.2795	
9895	44	18.85714	16.78834	139.6207	Normal Pool
9890	39	16.71429	15.97459	132.8531	
9885	34	14.57143	15.1171	125.7218	
9880	29	12.42857	14.20796	118.1608	
9875	24	10.28571	13.23651	110.0818	
9870	19	8.142857	12.18788	101.3608	
9865	14	6	11.04009	91.81521	
9860	9	3.857143	9.758218	81.15448	

1. Equation 33 from Small Dams page 425 used to solve for Q. Full flow conditions discharging into a channel with a supported outflowing jet.

$H/D>1.2$, the flow characteristics are those of orifice or sluice flow. Because the conduit is considered to be flowing partly full at supercritical stage for all H/D ranges indicated, the downstream conditions have no influence on the discharge.

On figure 9-68, the head-discharge relationship for the rounded inlet for values of $H/D<1.2$ (approx.) lie to the right of those for the square-edged inlet. This indicates slightly greater discharges for equal size conduits. The increased discharge capacity through the critical-depth flow range is the result of improved streamlined flow brought about by the suppression of the inlet contractions. For $H/D>1.2$, the pulsating flow characteristics begin, and the discharge-head relationship in this range of flow is uncertain; it cannot be determined until the flow stabilizes at full flow stage. Because full pipe flow is governed by control at the outlet, the head-discharge relationship can be determined by the application of Bernoulli's theorem. Referring to figure 9-69:

$$H_T = H + L\sin\theta - \frac{D}{2} \qquad (30)$$

Similarly,

$$H_T = h_v + h_e + h_f$$

or

$$H_T = \left(1 + K_e + \frac{29.1n^2L}{r^{4/3}}\right)\frac{v^2}{2g} \qquad (31)$$

where:

K_e = entrance loss coefficient, and
n = friction factor in Mannings equation,
$h_f = (29.1\ n^2L/r^{4/3})(v^2/2g)$.

Combining equations (30) and (31), dividing by D, and stating the equation in terms of Q instead of v yields:

$$\frac{H}{D} + \frac{L}{D}\sin\theta - \frac{1}{2} =$$

$$0.0252\left(1 + K_e + \frac{29.1n^2L}{r^{4/3}}\right)\left(\frac{Q}{D^{5/2}}\right)^2 \qquad (32)$$

In equation (32), it is assumed that the culvert discharges freely at the outlet and that the pressure line at the outlet is approximately at the center of the pipe. If the outlet discharges into a channel so that the outflowing jet is supported, equation (32) becomes:

$$\frac{H}{D} + \frac{L}{D}\sin\theta - 1.0 =$$

$$0.0252\left(1 + K_e + \frac{29.1n^2L}{r^{4/3}}\right)\left(\frac{Q}{D^{5/2}}\right)^2 \qquad (33)$$

Equations (32) and (33) are for full-flow conditions. They are expressed in terms of H/D and $Q/D^{5/2}$ so that by referring to figures 9-67 and 9-68, it can be determined whether or not the full-flow condition exists.

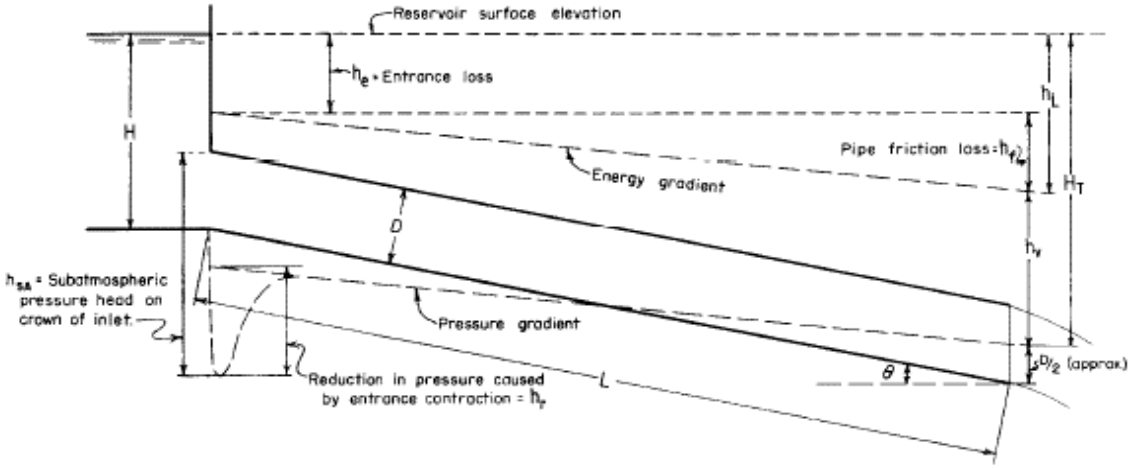
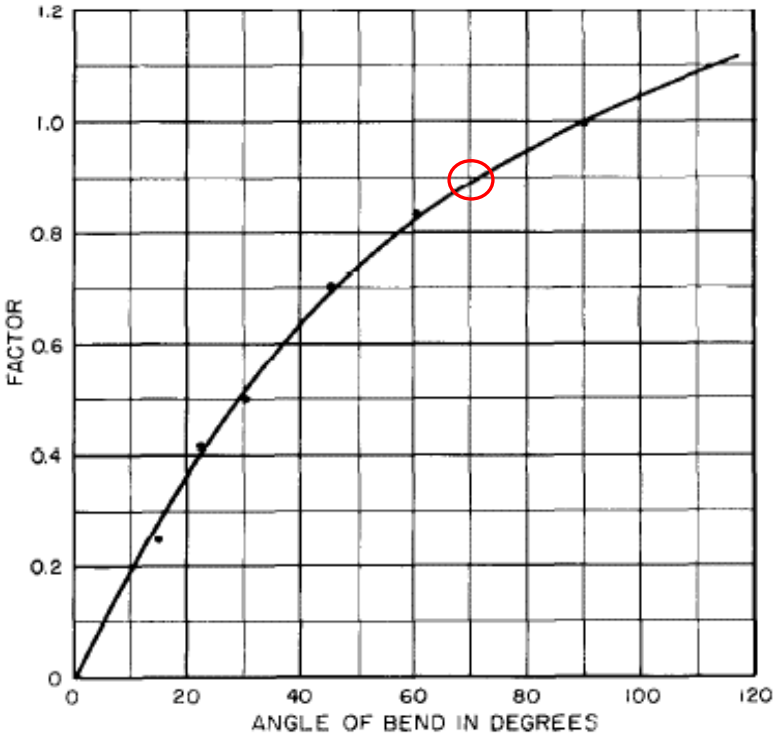


Figure 9-69.—Hydraulic characteristics of full pipe flow for culvert spillways. 288-D-2519.



(B) FACTORS FOR OTHER THAN 90° BENDS

Figure 10-12.—Bend loss coefficients. 288–D–2533.

Small Dams page 459

Kb for 90 degree flanged bend = 0.3

Proposed inlet structure has a 68 degree bend

Kb for proposed inlet structure = 0.27

Accounting for head loss in the 30-inch pipe resulting from the 68 degree bend at the outlet works intake structure

Estimate that the installed CIPP lining reduces the diameter of the flow area from 30 inches to 28 inches (i.e., a 1-inch thick lining)

1. Loss coefficient for 90 degree bend is 0.3. Since we have a 68-degree bend, we can use Design of Small Dams Figure 10-12 to determine an adjusted bend loss coefficient of 0.90 (from figure) x 0.3 = 0.27.

2. We can estimate an equivalent added pipe length to account for the bend loss coefficient of 0.27:

a. Manning's equation applied to closed-circuit flow

$$h_f = 29.1n^2\left(\frac{L}{r^{4/3}}\right)\frac{v^2}{2g}$$

Loss coefficient component

b. Solve for L required to produce a loss coefficient of 0.27

n	0.010	Manning's n for CIPP lining
r (ft)	0.5825	Hydraulic radius (2.33-ft pipe flowing full)
Kb	0.27	estimated loss coefficient for 68 degree bend
L	45.13693	equivalent pipe length to account for bend

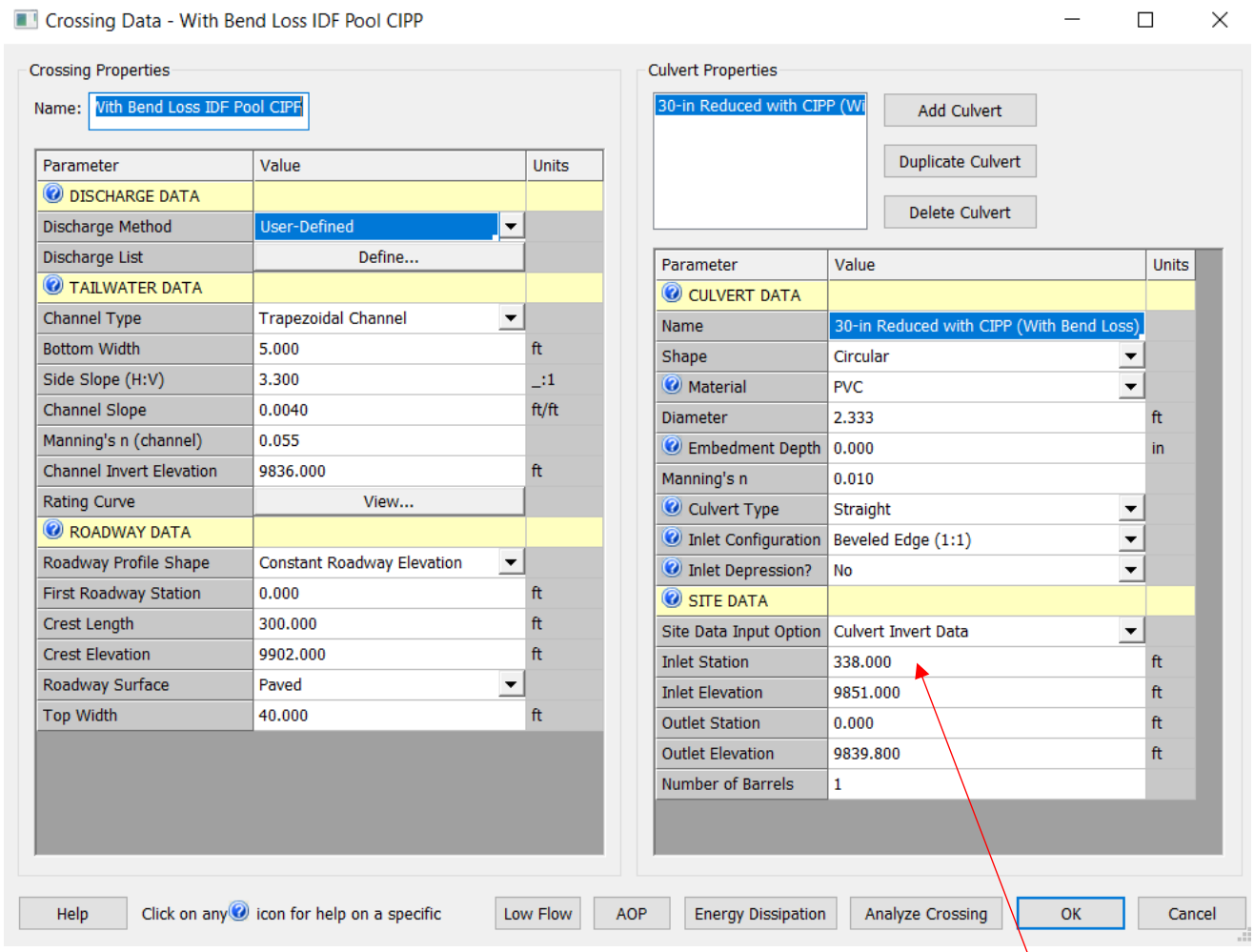
Increase pipe length by 50 ft to account for headloss due to bend.

Rating Curve Development for Hogchute Dam Low-Level Outlet Pipe - Gate Fully Open
Computed using HY-8 7.50

HW (ft)	H (ft)	Q (cfs)[1]	Notes
9851	0	0	
9852.5	1.5	10	
9853.3	2.3	20	
9854.27	3.27	30	
9855.63	4.63	40	
9857.31	6.31	50	
9859.44	8.44	60	
9862.07	11.07	70	
9865.1	14.1	80	
9868.54	17.54	90	
9872.38	21.38	100	
9876.62	25.62	110	
9881.44	30.44	120	
9888.27	37.27	130	
9895.1	44.1	139.3	Normal Pool
9895.64	44.64	140	
9900.4	49.4	146.1	IDF Pool
9902	51	148.09	

1. Elevation-discharge pairs estimated using HY-8 model of fully open, reduced pipe

HY-8 Input for Outlet Pipe Rating Curve Development
30-inch pipe reduced to 28-inch diamater as a result of CIPP application, estimated to be 1 inch thick
Controlling slide gate operating fully open



PIpe length increased by 50 ft to account for the bend loss: 288 ft + 50 ft = 338 ft.

HY-8 Output for Outlet Rating Curve Development

Culvert Summary Table - 30-in Reduced with CIPP (With Bend Loss)

Discharge Names	Total Discharge	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
10	10.00	10.00	9852.50	**1.50**	-9.85	1-S2n	0.54	1.08	0.54	0.94	12.87	1.30
20	20.00	20.00	9853.30	**2.30**	-8.56	1-S2n	0.77	1.55	0.77	1.34	15.68	1.58
30	30.00	30.00	9854.27	**3.27**	-6.63	5-S2n	0.96	1.89	0.96	1.64	17.51	1.76
40	40.00	40.00	9855.63	**4.63**	-4.61	5-S2n	1.13	2.11	1.18	1.88	17.86	1.90
50	50.00	50.00	9857.31	**6.31**	-2.10	5-S2n	1.29	2.22	1.36	2.09	18.70	2.01
60	60.00	60.00	9859.44	**8.44**	0.96	5-S2n	1.46	2.33	1.46	2.27	20.71	2.11
70	70.00	70.00	9862.07	**11.07**	4.51	5-S2n	1.63	2.33	1.63	2.44	21.29	2.20
80	80.00	80.00	9865.10	**14.10**	8.60	5-S2n	1.84	2.33	1.94	2.59	20.59	2.28
90	90.00	90.00	9868.54	**17.54**	13.24	6-FFc	2.33	2.33	9853.33	2.74	21.05	2.35
100	100.00	100.00	9872.38	**21.38**	18.43	6-FFc	2.33	2.33	9853.33	2.87	23.39	2.41
110	110.00	110.00	9876.62	**25.62**	24.16	6-FFc	2.33	2.33	9853.33	2.99	25.73	2.47
120	120.00	120.00	9881.44	30.27	**30.44**	6-FFc	2.33	2.33	2.33	3.11	28.07	2.52
130	130.00	130.00	9888.27	35.33	**37.27**	6-FFc	2.33	2.33	2.33	3.23	30.41	2.58
139.3	139.30	139.30	9895.10	40.39	**44.10**	6-FFc	2.33	2.33	2.33	3.33	32.59	2.62
140	140.00	140.00	9895.64	40.78	**44.64**	6-FFc	2.33	2.33	2.33	3.33	32.75	2.63
146.1	146.10	146.10	9900.40	44.31	**49.40**	6-FFc	2.33	2.33	2.33	3.40	34.18	2.65
148.09	148.09	148.09	9902.00	45.50	**51.00**	6-FFc	2.33	2.33	2.33	3.42	34.64	2.66

Display
- ○ Crossing Summary Table
- ◉ Culvert Summary Tabl — 30-in Reduced with CIPP (With Ben
- ○ Water Surface Profiles
- ○ Tapered Inlet Table
- ○ Customized Table — Options...

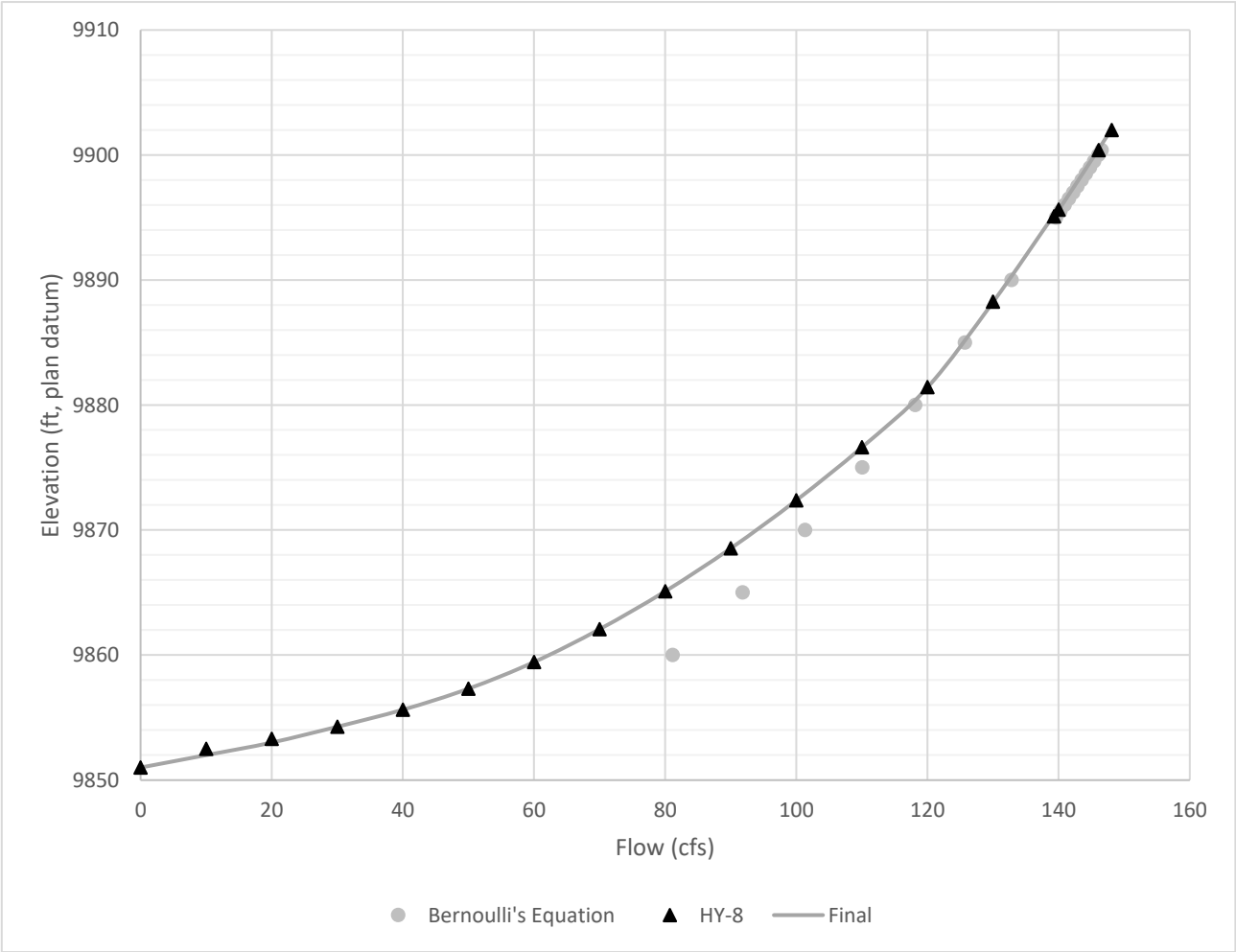
Geometry
- Inlet Elevation: 9851.00 ft
- Outlet Elevation: 9839.80 ft
- Culvert Length: 338.19 ft
- Culvert Slope: 0.0331
- Inlet Crest: 0.00 ft
- Inlet Throat: 0.00 ft

Outlet Control: Profiles

Plot
- Crossing Rating Curve
- Culvert Performance Curve
- Selected Water Profile
- Water Surface Profile Data

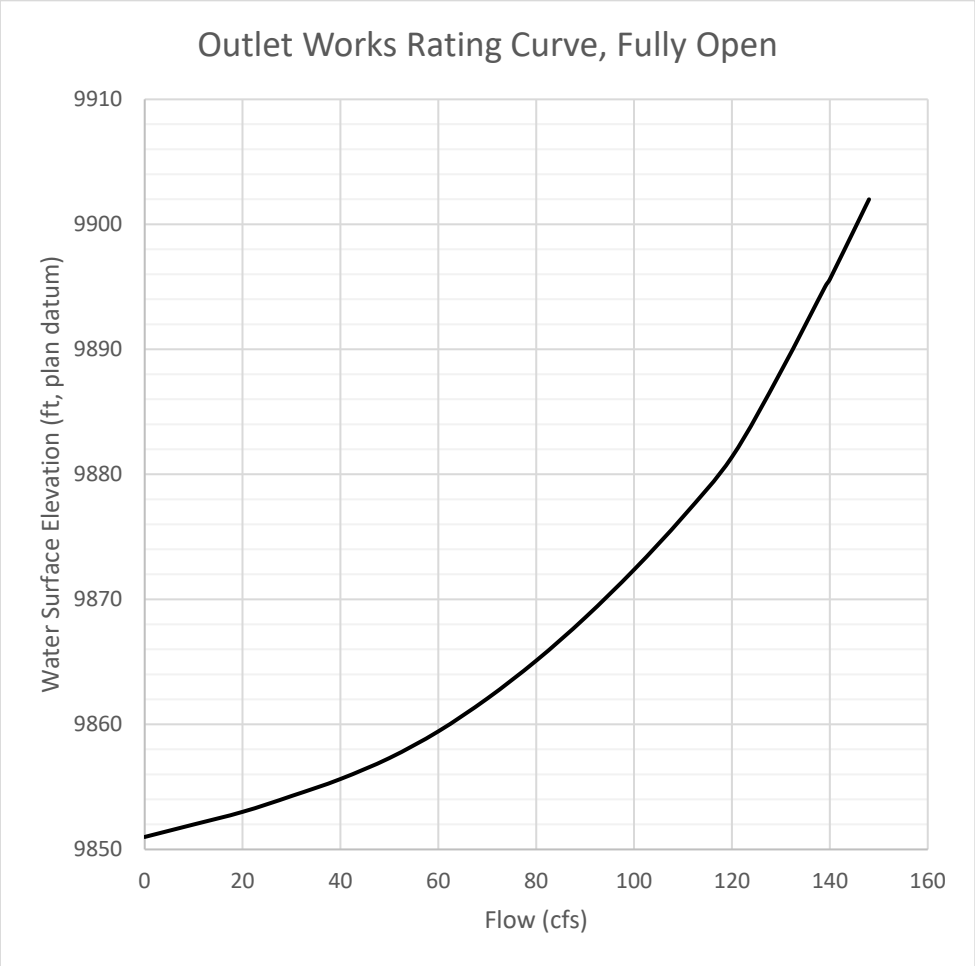
Help | Flow Types... | Edit Input Data... | Energy Dissipation... | AOP... | Low Flow... | Export Report | Adobe PDF (*.pdf) | Close

Outlet Rating Curve Comparisons



Final Curve

HW (ft)	Q (cfs)
9851	0
9852	10
9853	20
9854.27	30
9855.63	40
9857.31	50
9859.44	60
9862.07	70
9865.1	80
9868.54	90
9872.38	100
9876.62	110
9881.41	120
9888.23	130
9895	139
9895.6	140
9900.4	146
9902	148



Appendix L

Early Warning System Feasibility Study

Hogchute Dam Early Warning System Feasibility Study

Mesa County, Colorado
DAM ID 420127
CONSTRUCTION NUMBER C-0454A
Water Division 4
Water District 42

Prepared for:

City of Grand Junction

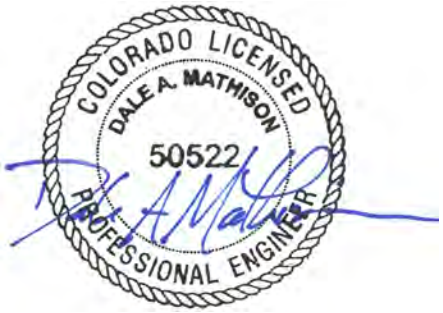
December 14, 2020

Hogchute Dam Early Warning System Feasibility Study

Mesa County, Colorado
DAM ID 420127
CONSTRUCTION NUMBER C-0454A
Water Division 4
Water District 42

Engineer's Certification

I hereby certify that I am a Professional Engineer licensed in the state of Colorado and that this report was prepared by me or under my supervision.



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Contents

Page No.

Background 1
Proposed Early Warning System Program 2
Program purpose 2
Program Components 2
Stakeholders 6
Instrumentation and Data Dissemination 9
Data Monitoring 11
Warning Issuance 15
Emergency Notification Protocol 19
Education 20
Maintenance 21
Cost 22
References 23

List of Appendices

Appendix A Inundation Maps

Appendix B Documentation of Stakeholder Coordination

Appendix C Supplemental Cost Information

List of Figures

Page No.

Figure 1. Satellite telemetry for transmitting data from site to secured database (photo courtesy of OneRain Inc.) 10
Figure 2. Example on-line interface (dashboard) for viewing and monitoring real-time conditions at the site (image courtesy of OneRain Inc.) 12
Figure 3. Example of current sensor reading summary viewed through the online interface (image courtesy of OneRain Inc.) 13
Figure 4. Example alarm notification message. The system issued an alarm when the water level reached 3 feet below the top of bank (image courtesy of OneRain Inc.) 14
Figure 5. Example plots of real-time water level and incremental precipitation data, accessed after issuance of an alarm notification message (image courtesy of OneRain Inc.) 14

List of Tables

Page No.

Table 1. Structures at risk and predicted fatalities due to an overtopping failure of Hogchute Dam (from Ayres [2020]) assuming little or no warning. Houses listed are mapped in Appendix A. 4
Table 2. Structures at risk and predicted fatalities due to an overtopping failure of Hogchute Dam (from Ayres [2020]) assuming adequate warning. Houses listed are mapped in Appendix A. 5
Table 3. Roster of Operational Stakeholders for Hogchute Dam EWSP 7
Table 4. Event level thresholds for Hogchute Dam EWSP 16

Background

The State of Colorado Division of Water Resources (DWR) Dam Safety Branch (DSB) allows for a dam's Hydrologic Hazard to be reduced through the implementation of an early warning system (EWS). Effective EWSs reduce risk for high- and significant-hazard dams by providing advanced warning that hydrologic conditions may be capable of causing a dam failure or that a dam failure is developing. In Colorado, EWSs—which consist of the physical equipment needed to provide advanced warning—are implemented using an Early Warning System Program (EWSP). While not required for all dams in Colorado, implementation of an EWSP at a candidate dam may affect the spillway's hazard classification and therefore reduce the spillway's design requirements. Hogchute Dam is an example of such a candidate.

In 2020, Ayres completed a Reclamation Consequence Estimating Methodology (RCEM) study for Hogchute Dam (Ayres 2020) based on technical guidelines from USBR (2015). The RCEM study results presented in Ayres (2020) indicated that, with the implementation of an effective EWS, the hydrologic hazard classification for Hogchute Dam's spillway could be "high." With this classification, Hogchute Dam's existing spillway capacity appears to be sufficient and only repairs and enhancements to the spillway chute are likely needed going forward.

The first step in developing an EWSP is to carry out a feasibility study to describe the warning system, roles and responsibilities of key stakeholders, operational and maintenance requirements, estimated costs (both upfront and ongoing), and how the system would be tested. Ayres was retained by the City of Grand Junction to complete the EWS feasibility study, which is documented in this report.

Proposed Early Warning System Program

Program purpose

The hydraulic analysis completed for the RCEM study (Ayres 2020) revealed that 33 residential structures downstream of the dam may be at risk of inundation should an overtopping failure of Hogchute Dam occur. A map of these structure locations is provided in Appendix A, and a listing is provided Table 1. For reference, the flood event capable of causing an overtopping failure of Hogchute Dam is a flood rarer (i.e. larger in magnitude) than the estimated 10,000-yr (0.01% annual exceedance probability) event.

The predicted fatalities shown in Table 1 were computed presuming that a failure of Hogchute Dam occurred with little to no advanced warning (i.e., a hypothetical effective EWS was not in place). The Depth × Velocity (DV) values in Table 1 were extracted from the results of the 2-dimensional hydraulic model used to simulate the overtopping dam breach of Hogchute Dam. We correlated these DV values with fatality rates using the approach described in USBR (2015) to compute the predicted fatalities at each structure. The anticipated flood wave arrival time for each structure is also tabulated. Put simply, these arrival times represent the estimated time after dam failure available for residents of the structures to evacuate before the dam breach flood wave reaches their properties.

Following USBR (2015) procedures, we also computed predicted fatalities assuming adequate warning (i.e., a hypothetical effective EWS was in place). Results of this analysis are presented in Table 2. As can be seen, implementation of an effective EWS decreases estimated total expected fatalities from 6.5 (no advanced warning) to 0.26 (adequate advanced warning). The ultimate purpose of the proposed EWSP for Hogchute Dam is to provide adequate warning, in the event of a possible dam failure, to allow for the residents of the 33 structures in the inundation zone to evacuate prior to arrival of the dam breach flood wave.

Program Components

Considering the goal described above, the proposed EWSP will consist of the following components:

- **Stakeholders**: All key stakeholders must be engaged in the development of the EWSP, must approve of the EWSP, and must be aware of their ongoing roles in enacting and carrying out the EWSP. Key stakeholders include the City, the DSB, state and local emergency management officials, local law enforcement and emergency responders, the National Weather Service, local government, and the individuals living in the at-risk residences downstream.
- **Instrumentation and Data Dissemination**: Water level and rain gages will be installed at the dam to provide constant monitoring of hydrologic conditions. Data collected at the dam by the installed instruments will be disseminated in real time to an on-line interface for access by key stakeholders.
- **Data Monitoring**: As the Hogchute Dam's owner and operator, the City will provide an on-call representative at all times responsible for regularly viewing data through the online interface and monitoring weather conditions at the dam. The City's on-call representative will be first in the line of activating the dam's emergency action plan (EAP) should a hazardous situation develop. Therefore, this representative must maintain constant awareness of existing and forecasted conditions at the dam.

- **Warning Issuance**: The online interface used to view data will be configured with rules to issue automated warnings should various data thresholds be met. Warning alarms will be sent to the City's on-call representative and other key stakeholders for validation and to activate the dam's EAP, if required.
- **Emergency Notification**: If hazardous conditions indicate that a dam failure is in progress or imminent, the City's on-call representative and other stakeholders will be notified and will initiate the emergency notification protocol to ensure that people at risk downstream of the dam are warned and have appropriate time to act.
- **Education**: All key stakeholders must be made aware of their roles and responsibilities in the EWSP. Stakeholder education will consist of regular training activities, such as tabletop exercises and system tests and drills, and regular communication with persons at risk to ensure they are reminded and informed of the EWSP and know their responsibilities. Education also includes training for City staff and other key stakeholders to ensure that they know how to use the EWS interface.
- **Maintenance**: After implementation, regular maintenance of the EWS will be required. Maintenance activities include not only actual maintenance of instruments and data systems, but regular reviews and updates of program protocol, as well.

The EAP for Hogchute Dam must be updated to reflect the EWSP and the roles and responsibilities of all key stakeholders. The EWSP will enhance the safety of individuals downstream of the dam by linking real-time conditions (as monitored by the installed instruments) to the updated EAP.

Table 1. Structures at risk and predicted fatalities due to an overtopping failure of Hogchute Dam (from Ayres [2020]) assuming little or no warning. Houses listed are mapped in Appendix A.

Predicted Fatalities Due to Overtopping Failure (Peak Outflow = 53,000 cfs), No Warning					
House ID #	House Type	Depth x Velocity (ft^2/s)	Fatality Rate*	Predicted Fatalities**	Wave Arrival Time (H:MM after failure)
1	Mobile Home	69	0.4535	1.0883	0:25
2	Guest House	72	0.1964	0.4713	0:40
3	Single Family Residence	8	0.0010	0.0024	1:25
4	Single Family Residence	39	0.0281	0.0673	1:00
5	Single Family Residence	53	0.0849	0.2038	1:00
6	Single Family Residence	6	0.0010	0.0024	0:55
7	Single Family Residence	44	0.0427	0.1026	0:40
8	Single Family Residence	4	0.0010	0.0024	0:45
9	Single Family Residence	40	0.0296	0.0710	0:50
10	Single Family Residence / Monastery	129	0.5220	1.2529	0:35
11	Single Family Residence	52	0.0759	0.1822	0:30
12	Mobile Home	54	0.2490	0.5976	0:25
13	Single Family Residence	85	0.2844	0.6826	0:25
14	Single Family Residence	111	0.4392	1.0540	0:20
15	Single Family Residence	30	0.0107	0.0257	0:55
16	Mobile Home	22	0.0185	0.0444	1:45
17	Single Family Residence	5	0.0010	0.0024	1:45
18	Single Family Residence	8	0.0010	0.0024	1:25
19	Mobile Home	52	0.2274	0.5457	1:05
20	Single Family Residence	10	0.0010	0.0024	1:05
21	Single Family Residence	23	0.0023	0.0056	1:00
22	Single Family Residence	7	0.0010	0.0024	0:45
23	Single Family Residence	16	0.0010	0.0024	1:00
24	Single Family Residence	2	0.0010	0.0024	1:30
25	Single Family Residence	1	0.0010	0.0024	-
26	Mobile Home	4	0.0100	0.0240	1:30
27	Single Family Residence	0	0.0000	0.0000	1:40
28	Single Family Residence	0	0.0010	0.0024	1:45
29	Mobile Home	0	0.0100	0.0240	1:55
30	Single Family Residence	-	0.0000	0.0000	-
31	Mobile Home	5	0.0100	0.0240	2:30
32	Single Family Residence	-	0.0010	0.0024	-
33	Mobile Home	-	0.0000	0.0000	-
	Total Fatalities: 6.5				

*** Fatality rate estimates are based on the Depth x Velocity relationship found on Figures 3 and 4 of USBR's *Reclamation Consequence Estimating Methodology* (July 2015). The high "Suggested Limit" curve is used for all homes except mobile homes which use the high "Overall Limit" curve to account for the greater risk expected at lower quality shelters.**

**** Based on 2.4 persons per household (US Census Bureau records for Mesa County, Colorado, 2018); "Single Family Residence and Townhouse" treated as two households.**

Table 2. Structures at risk and predicted fatalities due to an overtopping failure of Hogchute Dam (from Ayres [2020]) assuming adequate warning. Houses listed are mapped in Appendix A.

Predicted Fatalities Due to Overtopping Failure (Peak Outflow = 53,000 cfs), Adequate Warning					
House ID #	House Type	Depth x Velocity (ft^2/s)	Fatality Rate*	Predicted Fatalities**	Wave Arrival Time (H:MM after failure)
1	Mobile Home	69	0.0266	0.0638	0:25
2	Guest House	72	0.0041	0.0098	0:40
3	Single Family Residence	8	0.0002	0.0005	1:25
4	Single Family Residence	39	0.0017	0.0042	1:00
5	Single Family Residence	53	0.0027	0.0065	1:00
6	Single Family Residence	6	0.0002	0.0005	0:55
7	Single Family Residence	44	0.0021	0.0049	0:40
8	Single Family Residence	4	0.0002	0.0005	0:45
9	Single Family Residence	40	0.0018	0.0042	0:50
10	Single Family Residence / Monastery	129	0.0078	0.0188	0:35
11	Single Family Residence	52	0.0026	0.0062	0:30
12	Mobile Home	54	0.0172	0.0414	0:25
13	Single Family Residence	85	0.0051	0.0121	0:25
14	Single Family Residence	111	0.0067	0.0162	0:20
15	Single Family Residence	30	0.0012	0.0028	0:55
16	Mobile Home	22	0.0042	0.0100	1:45
17	Single Family Residence	5	0.0002	0.0005	1:45
18	Single Family Residence	8	0.0002	0.0005	1:25
19	Mobile Home	52	0.0163	0.0390	1:05
20	Single Family Residence	10	0.0002	0.0005	1:05
21	Single Family Residence	23	0.0007	0.0018	1:00
22	Single Family Residence	7	0.0002	0.0005	0:45
23	Single Family Residence	16	0.0004	0.0010	1:00
24	Single Family Residence	2	0.0002	0.0005	1:30
25	Single Family Residence	1	0.0002	0.0005	-
26	Mobile Home	4	0.0011	0.0026	1:30
27	Single Family Residence	0	0.0000	0.0000	1:40
28	Single Family Residence	0	0.0002	0.0005	1:45
29	Mobile Home	0	0.0011	0.0026	1:55
30	Single Family Residence	-	0.0000	0.0000	-
31	Mobile Home	5	0.0011	0.0026	2:30
32	Single Family Residence	-	0.0002	0.0005	-
33	Mobile Home	-	0.0000	0.0000	-
	Total Fatalities: 0.26				

*** Fatality rate estimates are based on the Depth x Velocity relationship found on Figures 3 and 4 of USBR's *Reclamation Consequence Estimating Methodology* (July 2015). The high "Suggested Limit" curve is used for all homes except mobile homes which use the high "Overall Limit" curve to account for the greater risk expected at lower quality shelters.**

**** Based on 2.4 persons per household (US Census Bureau records for Mesa County, Colorado, 2018); "Single Family Residence and Townhouse" treated as two households.**

Stakeholders

Success of the Hogchute Dam EWSP relies on the input and cooperation of key stakeholders. Stakeholders fall within one of two groups:

- **Operational**: Those whose buyoff is required to move the program forward. These stakeholders have responsibilities for action under the EWSP. Thus, their approval of the EWSP is required prior to, and throughout, its implementation. Operational stakeholders include, but are not limited to, the City, the Mesa County Emergency Management Coordinator, the Colorado Division of Water Resources Dam Safety Engineer, and the National Weather Service (NWS).
- **Informational**: Those who must be kept informed of the EWSP, but their approval is not necessarily required. Examples of informational stakeholders are the individuals at risk downstream of the dam and irrigators who draw water from Kannah Creek or buy water from the City.

The list of operational stakeholders is provided in Table 3. The EAP for Hogchute Dam will be updated to include the operational stakeholders shown in the table, as well as informational stakeholders to be identified as the EWSP is developed.

We have involved operational stakeholders in the development of the EWSP throughout this feasibility study. Following are major milestones and opportunities for stakeholders to provide feedback:

- 60-percent draft submitted to City for review: September 21, 2020
- 60-percent draft submitted to DSB for review: September 22, 2020
- 90-percent draft submitted to operational stakeholders for review: November 4, 2020
- First stakeholder workshop: November 9, 2020
- Second stakeholder workshop: December 2, 2020
- Final EWSP feasibility study report submitted to DSB: December 14, 2020

A list of stakeholders involved in the review process, as well as comments and responses resulting from their reviews, are provided in Appendix B.

Table 3. Roster of Operational Stakeholders for Hogchute Dam EWSP

Stakeholder	Role	Representative	Email	Phone
City of Grand Junction (Dam Owner)	Water Services Manager	Mark Ritterbush	markri@gjcity.org	970.256.4185 970.210.1027
	Utilities Director	Randi Kim	randik@gjcity.org	970.244.1429 727.519.8857
	Water Supply Supervisor	Slade Connell	sladec@gjcity.org	970.270.4360 970.241.3889
	IT Department	Richard White	richardw@gjcity.org	970.244.1526 970.210.2840
Mesa County Emergency Management Coordinator	Local Emergency Manager	Andrew Martsolf	andrew.martsolf@mesacounty.us	970.242.6707 970.244.1763
Colorado Department of Homeland Security and Emergency Management	Office of Emergency Management: West Region Field Manager	Drew Petersen	drew.petersen@state.co.us	970.633.0201
	Colorado Department of Transportation: Region 3	Chad Ray	chad.ray@state.co.us	719.240.1531 970.683.6284
	Colorado State Patrol: District 4	David Aldridge	david.aldridge@state.co.us	970.824.6501 970.249.4392
Colorado Division of Water Resources	Dam Safety Engineer	Jackie Blumberg	jackie.blumberg@state.co.us	303.505.6469 970.945.5665
	Chief	Bill McCormick	bill.mccormick@state.co.us	719.338.6124 303.866.3581 x8262
NWS	Warning Meteorologist	Jeff Colton	jeff.colton@noaa.gov	970.243.7007 x726 800.868.7964 x726
	Warning Hydrologist	Aldis Strautins	aldis.strautins@noaa.gov	970.243.7007 800.868.7964
Mesa County Sheriff's Office	Sheriff	Matt Lewis	matt.lewis@mesacounty.us	970.244.3950

Stakeholder	Role	Representative	Email	Phone
Lands End Fire Protection District	Chief	Brian Lurvey	---	970.242.2975
Grand Junction Regional Communication Center	Combined Public Safety Dispatch (911)	Gus Hendricks	gush@gjcity.org	970.549.5800
US Forest Service	Owner of land on which dam is located	Loren Paulson	loren.paulson@usda.gov	970.263.5812
Placeholder for selected EWSP vendor	Instrumentation and data for EWSP	To be determined	To be determined	To be determined

Instrumentation and Data Dissemination

Currently, the EAP for Hogchute Dam relies on first-person observations of conditions at the dam, as no instrumentation or telemetry are currently in place to provide remote monitoring. One of the primary benefits of the proposed EWSP will be the installation of a system capable of measuring conditions at the dam and transmitting those data, in real-time, to a database from where they can be viewed and monitored by City staff.

Instruments will likely consist of the following:

- Two reservoir pool level sensors. Two sensors are used for redundancy, in case one of the sensors fails or provides suspect readings. The pool level sensors will provide for real-time monitoring of lake levels at the site. For Hogchute Dam, we propose that a bubbler sensor be used as a primary means to take real-time pool level measurements. The backup sensor may be a pressure transducer.
- One tipping bucket rain gage at the dam site. The rain gage will provide supporting data that can validate or invalidate observations measured by the pool level sensors.
- One water level sensor located in the creek downstream of the dam and spillway. This sensor will be used for redundancy, to confirm or refute warnings issued solely as a function of measured reservoir levels upstream. Data measured from a water level sensor downstream of the dam will also allow the City to track historical flow patterns resulting from hydrology and operations.

Instruments will be powered by solar-charged batteries since no other power sources are available at the site.

Data will be transmitted via satellite telemetry from a base station at the dam to a secure database maintained by the City's selected vendor. In emergency situations, satellite communications ensure data are available to critical personnel when cellular systems are often overwhelmed or out of operation. The City's selected vendor will provide data transmittal services that follow industry standards for cyber security. The proposed system is designed to have low power consumption and is powered by batteries that are charged using a small solar panel. A Campbell Scientific datalogger offers a reliable, versatile platform from which to measure data measured by a wide variety of sensors and package those data for satellite transmission. Low power requirements and robust hardware will allow the site to operate for months at a time without maintenance.

The proposed base station uses traffic-style infrastructure capable of withstanding high winds. The batteries are housed within a stainless steel National Electrical Manufacturers Association (NEMA) rated enclosure that provides a watertight seal. The electronics are housed within a watertight plastic NEMA-rated enclosure that is housed within a ½-inch steel enclosure that prevents damage from firearms. Both enclosures accommodate a lock to prevent intrusion. The satellite antenna and sensors attached to the pole are mounted above a three-foot diameter disk that prevents vandals from removing site infrastructure. The solar panel is mounted on the plane of this disk to prevent theft and damage from firearms. All infrastructure is powder coated to prevent corrosion and to improve aesthetics. A photograph of a satellite telemetry station comparable to what would be installed at Hogchute Dam is provided in Figure 1.

Figure 1. Satellite telemetry for transmitting data from site to secured database (photo courtesy of OneRain Inc.)

Data Monitoring

After the data collected at the site are transmitted via satellite to a secure database, they will then be viewable in real time using a vendor-supported secure online interface. These interfaces can be configured to show real-time data feeds, as well as complimentary information such as meteorological forecasts from the NWS and up-to-date photographs at the site taken by automated cameras. (Note: The use of a remote camera at Hogchute Dam is likely not feasible for the foreseeable future because of the costs associated with transferring the picture files via satellite telemetry.) The interface will be accessible via PC, tablet, or smartphone to users authorized by the City. A screenshot image of an example interface is provided in Figure 2. In Figure 3, we show an example summary of current sensor readings, which is accessible through the online interface.

The City will designate an on-call representative who is responsible for monitoring conditions at Hogchute Dam using the configured online interface. We anticipate that the City will provide the on-call representative with a device dedicated for this task. The on-call representative must regularly monitor conditions at the site, as well as weather forecasts, to keep abreast of the situation and possible hydrologic risk.

The City's EWS vendor will configure an automated system to constantly monitor the data reported at Hogchute Dam and to issue alerts and warnings to the on-call representative and others when specified thresholds are met or exceeded. This automated system will be accessible through the online interface. An example alert and corresponding real-time data plots are shown in Figure 4 and Figure 5, respectively. Additional details regarding proposed thresholds and alerts are provided in the Warning Issuance chapter of this report.

Historical records of data collected at the site will be maintained on a server either hosted by the City's chosen vendor or owned by the City. Historical data records will be useful for future assessments of reservoir operations, analysis of historical meteorological and hydrological conditions, and for model calibration.

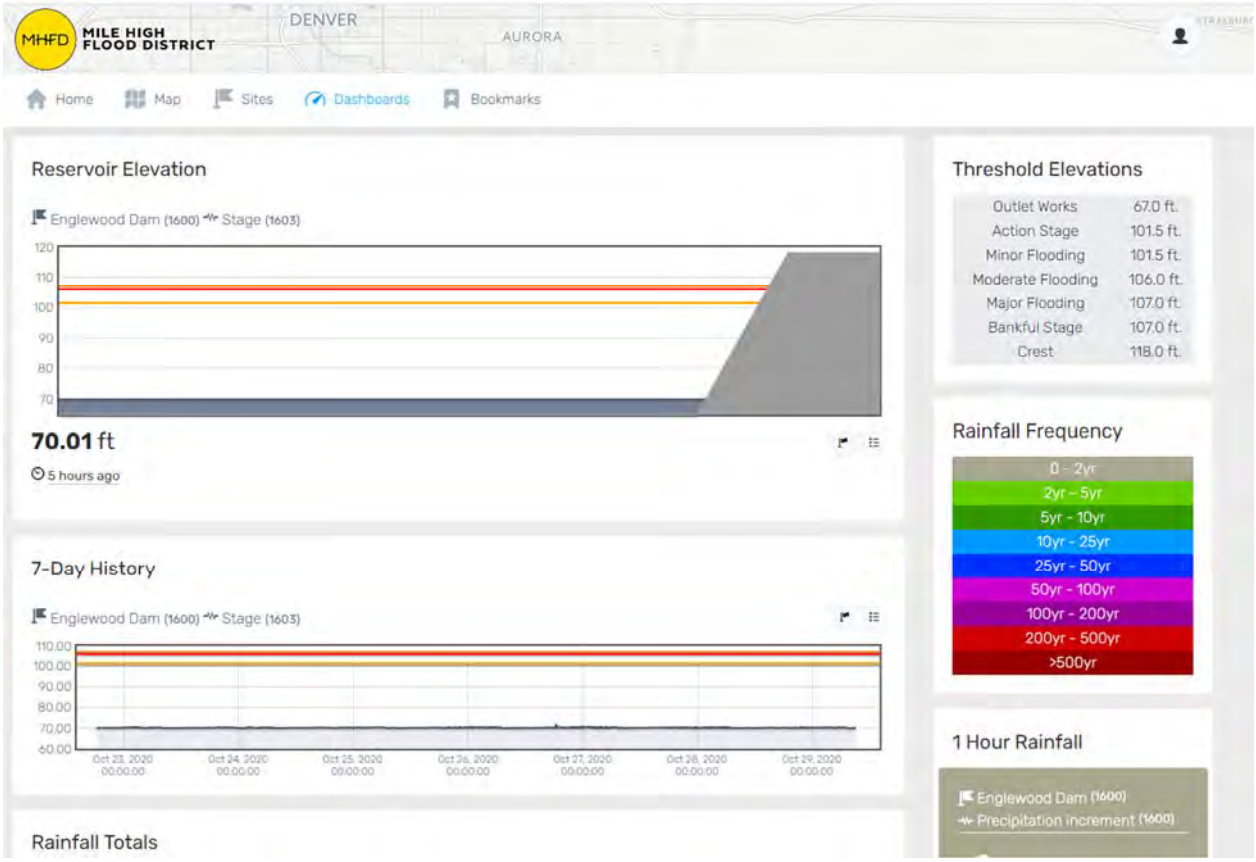


Figure 2. Example on-line interface (dashboard) for viewing and monitoring real-time conditions at the site (image courtesy of OneRain Inc.)

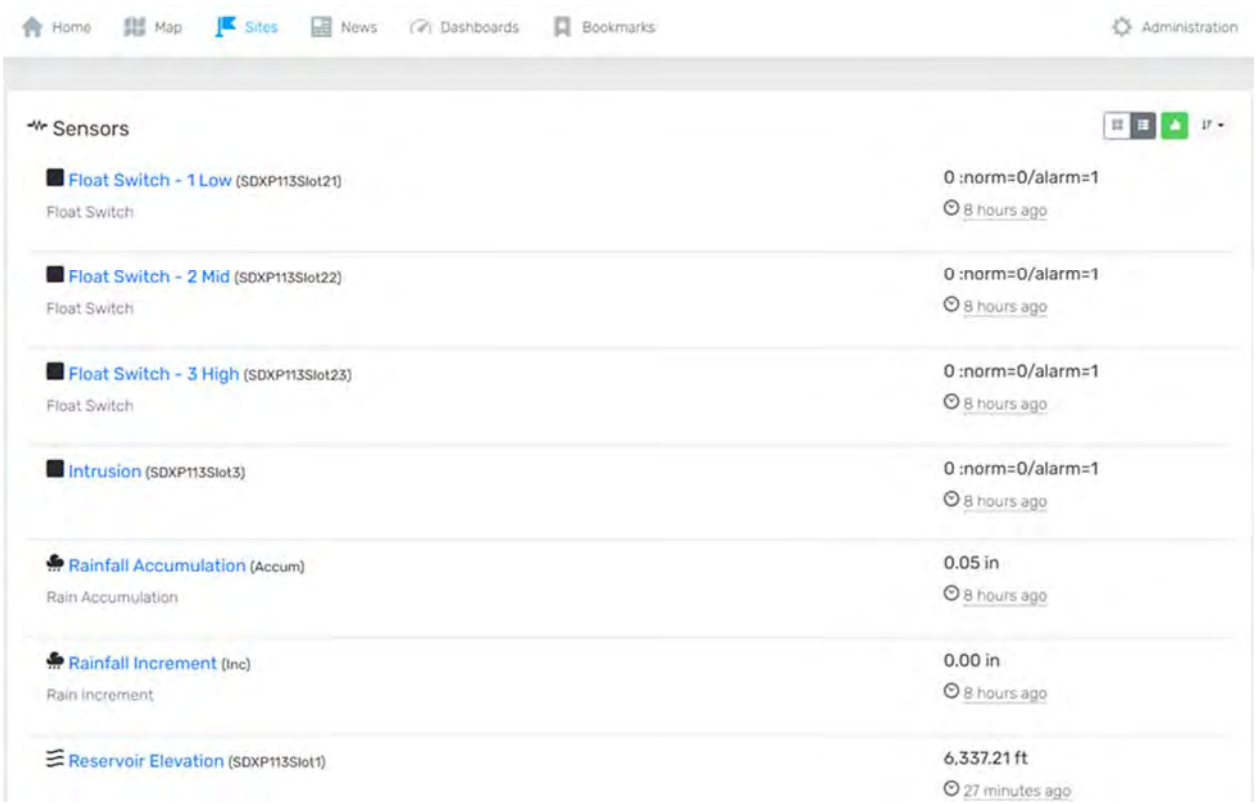


Figure 3. Example of current sensor reading summary viewed through the online interface (image courtesy of OneRain Inc.)

Figure 4. Example alarm notification message. The system issued an alarm when the water level reached 3 feet below the top of bank (image courtesy of OneRain Inc.)

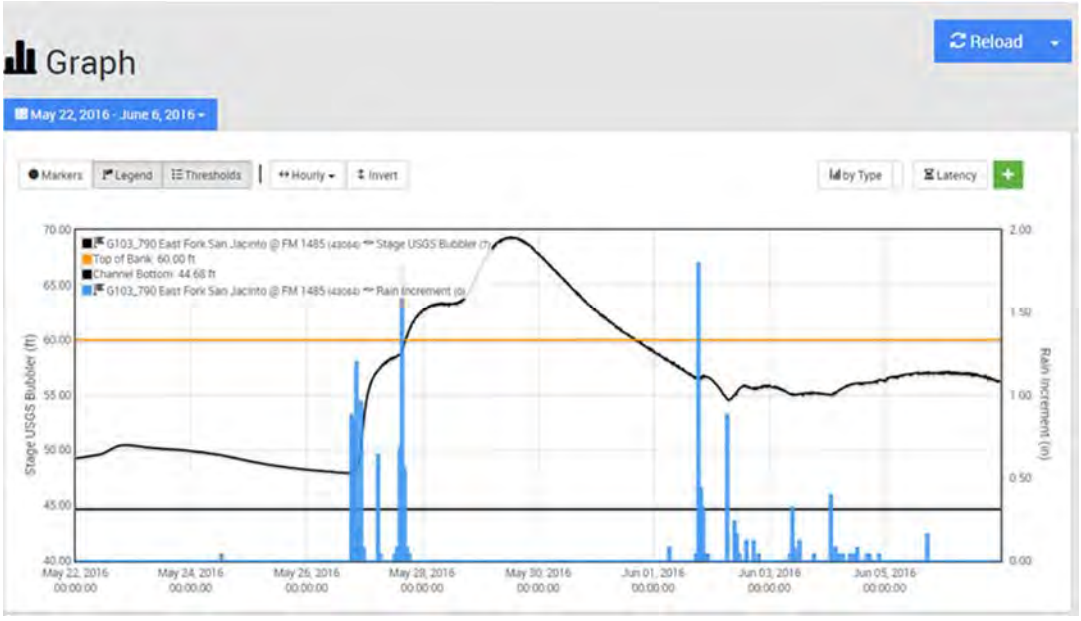


Figure 5. Example plots of real-time water level and incremental precipitation data, accessed after issuance of an alarm notification message (image courtesy of OneRain Inc.)

Warning Issuance

The EAP for Hogchute Dam documents event levels and corresponding responsibilities for situations when conditions experienced are not within the realm of normal operations at the dam. To date, assessment of whether conditions at the dam are normal has relied solely on personal observation. Because of the dam's remote location, personal observation is not always possible. Therefore, hazardous conditions could plausibly develop without anyone's knowledge.

The goal of the proposed EWSP at Hogchute Dam is to provide 24-7 remote monitoring of conditions at the dam so that hazardous conditions can be identified with sufficient time to activate the EAP and, if appropriate, evacuate those in harm's way downstream of the dam.

We propose five event levels to be identified in Hogchute Dam's EAP. Four of these event levels are described in the dam's existing EAP, and one is a proposed new event level to be added to the EAP in support of the EWSP:

- **Potential for Hazardous Conditions – Forecast-Based**: This is a proposed new event level to be added to Hogchute Dam's EAP. Triggering of this event level is intended to give stakeholders time to prepare for potentially hazardous hydrologic conditions. Quantitative precipitation forecasts (QPFs) issued by the NWS would be monitored, and this event level would be triggered if forecasted rainfall exceeds 10-yr annual exceedance interval depths. If required, forecasted air temperatures and other meteorological variables may also be monitored to ensure the forecasted precipitation is expected to fall as rain, not snow.
- **High Flow Below Dam – Non-Failure**: This event level is described in Hogchute Dam's existing EAP. It represents a situation where flows below the dam are higher than normal—because of hydrologic conditions, not because of a dam failure—and a heighted level of awareness is required.
- **Unusual Condition at Dam – Non-Failure (a non-emergency incident; slowly developing situation)**: This event level is described in Hogchute Dam's existing EAP. It represents an unusual, slowly developing situation at the dam. In general, these situations would not be the result of hydrologic conditions. Rather, these unusual conditions would represent slowly-developing changes to the dam's structure or appurtenant works. These conditions would be identified and monitored through regular dam visits and inspections. Therefore, we do not propose that monitoring of these types of situations be included as part of the EWSP.
- **Potential Dam Failure – Immediate Action Required:** This event level is described in Hogchute Dam's existing EAP. Under this event level, conditions are such that there is a real potential for dam failure, though a failure has not yet occurred and may be avoidable.
- **Evacuation Required – Dam Failure in Progress or Unavoidable (extremely urgent situation)**: This event level is described in Hogchute Dam's existing EAP. For this event level to be triggered, a dam failure is either in progress or imminent. In response, downstream evacuation is required.

Table 4 lists the EAP event levels and the proposed gage readings/thresholds that will trigger those event levels. The thresholds will be configured from within the vendor's online interface to automatically notify stakeholders when they are met or exceeded. These event level notifications will be sent via text message and email.

For *Potential for Hazardous Conditions – Forecast-Based* and *High Flow Below Dam – Non-Failure* event level warnings, the City's on-call representative will be required to confirm receipt of the message. If receipt is not confirmed within 10 minutes, additional City personnel will then be notified. After

message receipt confirmation, the City must view and analyze conditions from within the interface and either validate or invalidate the event level warning. Validated event level warnings will activate the dam's EAP, and responsibilities and actions described in the EAP will commence.

Unusual Condition at Dam – Non-Failure event levels would likely not be monitored as part of the flood EWSP. Unusual, slowly-developing, non-emergency conditions would likely be identified in-person during a visit to the dam.

For *Potential Dam Failure – Immediate Action Required* and *Evacuation Required – Dam Failure in Progress or Unavoidable* event level warnings, the situation may be too serious to allow for warning validation time. Therefore, warnings indicating these event levels will be sent immediately to the following key stakeholders, and responsibilities and actions described in the EAP will commence:

- City on-call representative.
- City Utilities Director.
- City Water Supply Supervisor.
- City Water Services Manager.
- Grand Junction Regional Communication Center.
- Mesa County Emergency Management Coordinator.
- Colorado DWR Dam Safety Engineer.
- NWS Operations Center.

Table 4. Event level thresholds for Hogchute Dam EWSP

Event Level (from EAP)	**Gage Readings[1]**	**Action**	**Significance**
Potential for Hazardous Conditions – Forecast Based	NWS QPFs contain forecasted precipitation that exceeds: • 1.2" (6 hr) • 2.5" (24 hr) • 3.4" (48 hr) • 3.9" (72 hr) (These are estimated 10-yr precipitation depths for the Hogchute Dam watershed.)	Follow EAP for Potential for Hazardous Conditions – Forecast Based.	NWS forecasts indicate that future precipitation in the area could result in hydrologic conditions capable of triggering other, more severe EAP event levels.

Event Level (from EAP)	Gage Readings[1]	Action	Significance
High Flow Below Dam – Non-Failure	10-yr flood flows: • Pool elevation 9895.5 ft. And, • 180-cfs flow/water level measured below dam. 25-yr flood flows: • Pool elevation 9896.1 ft. And, • 450-cfs flow/water level measured below dam. 50-yr flood flows: • Pool elevation 9896.6 ft. And, • 840-cfs flow/water level measured below dam. 100-yr flood flows: • Pool elevation 9897.2 ft. And, • 1320-cfs flow/water level measured below dam.	Follow EAP for High Flow Below Dam – Non Failure. EAP to be updated to correspond with increased actions based on increasing pool elevations.	Flows downstream of the dam are higher than normally experienced, though not of an unforeseen magnitude. No dam failure indicators, but hazard may escalate if conditions worsen.
Unusual Condition at Dam – Non-Failure	Unusual conditions at the dam would not be monitored as part of the flood EWSP. Rather, this EAP event level would likely be triggered by in-person observation.	Follow EAP for Unusual Condition at Dam – Non-Failure.	A regular dam visit identifies a potentially hazardous, slowly developing, unusual situation. This would likely not be related to a hydrologic event.

Event Level (from EAP)	Gage Readings[1]	Action	Significance
Potential Dam Failure – Immediate Action Required	Pool elevation within 2 ft of top of dam (9900 ft).	Follow EAP for Potential Dam Failure – Immediate Action Required.	Hydrological conditions are such that a continuation or worsening may result in an imminent dam failure. The dam and spillway are approaching, though have not exceeded, their design limitations.
Evacuation Required – Dam Failure in Progress or Unavoidable	Pool elevation equal to or exceeding that corresponding with a 10,000-year event (9900.40 ft). Or, Pool elevation is rapidly and inexplicably dropping.	Evacuation per EAP.	Hydrological conditions have caused the dam to meet or exceed its design limitations. Failure is in progress or unavoidable. Downstream evacuation required.

1. Thresholds listed are approximate and are subject to refinement following final design of the improved dam and spillway.

Emergency Notification Protocol

When an "Evacuation Required" event level is confirmed, the emergency notification protocol must be activated. The goal for this protocol is notify individuals in the 33 at-risk structures downstream of the dam that they must evacuate to an area safe from an impending or in-progress dam breach flood wave. Actions and responsibilities during an Evacuation Required event level will be described in Hogchute Dam's updated EAP. Additional details regarding the downstream evacuation procedure are provided here.

Prior to implementing the EWSP, the City will initiate contact with the owners and/or residents of all 33 affected downstream properties to obtain the information needed to develop an evacuation plan. The City will first attempt to reach the affected individuals by phone and mail. If needed information is not received in this manner, the City will visit the structures to speak with the owners and/or residents. Public meetings may also be organized, inviting the affected individuals to attend so they can provide the City with required information. This initial contact will be focused on obtaining the following information:

- Name of landowner.
- Names of all residents.
- Best-available phone number.
- Best-available email address.
- Landline availability.
- Cellphone signal quality.
- Special needs concerns (e.g., hard of hearing, blind, mobility issues, etc.).
- Residence details (e.g., year round, seasonal, etc.).

All information obtained will be compiled in Hogchute Dam's EAP. The EAP will also contain the closest available safe evacuation area for each structure.

Upon activation of the emergency notification protocol, the following process will be used to ensure that the at-risk residents downstream of the dam are aware of their need to evacuate:

- Emergency phone notifications (e.g., reverse 911) will be sent by the Grand Junction Regional Communication Center to all 33 identified at-risk structures. We anticipate that cell phone signals are poor throughout most of the at-risk area. Therefore, the City has agreed to work with the owners and/or residents of the 33 identified at-risk structures to ensure that functional landlines are installed to support emergency notification.
- Law enforcement and emergency response personnel will mobilize to visit and notify all at-risk properties of their need to evacuate. Priority will be placed on structures with known special-needs residents and structures closest to the dam.
- The NWS will issue a warning via National Oceanic and Atmospheric Administration (NOAA) Weather Radio.

The EAP will be updated to include a pre-scripted message to be used for evacuation orders.

Education

At program commencement, the selected EWSP vendor will provide training to City staff and other key stakeholders to provide instructions for using the implemented warning software and the customized user interface. Training should include, at minimum:

- How to access the database and online interface.
- How to check sensor status.
- How to view sensor data.
- How to generate maps and reports.
- How to select and export data.
- How to generate graphs.
- How to verify data after a warning is issued.
- How to obtain technical support.

Key to the success of the EWSP is that the affected individuals downstream of Hogchute Dam are also well informed of the program and kept up to date of their responsibilities should the emergency notification protocol be activated.

- To kick off the program, the City will send at-risk residents an informational letter and a follow up phone call. The purpose of this correspondence will be to introduce or to remind residents of the EWSP, describe its objectives, and describe how it will work. In addition to the written and phone correspondence, the City will organize and hold a local meeting(s) to provide further information and answer questions.
- The City will send yearly mailers (e.g., letters or pamphlets) to the affected residents reminding them of the EWSP and informing them of the upcoming annual system test. The yearly mailers will also contain information describing the closest available safe evacuation area for each structure.
- Following the annual system test, the City will solicit feedback from persons at risk that the test was successful (i.e., they received the test emergency notification message).
- On an as-needed basis, the City may hold additional public meetings or initiate further correspondence to provide affected residents with updated information, should significant changes be made to the EWSP.

Maintenance

The continued reliability and success of the EWSP will rely on maintenance of:

- The EWSP, itself, and the EAP to which it is tied. This includes ongoing assessment of stakeholders, roles, responsibilities, and the emergency notification protocol.
- The instruments and equipment used to collect and transmit data.

Maintenance of the EWSP will largely be accomplished through annual system tests. These tests will be completed once annually, preferably in the late spring or early summer to align with the start of the annual thunderstorm season. The test will consist of a drill staged to simulate the actual response required during an imminent dam failure (i.e., an Evacuation Required EAP event level). Therefore, the complete system, from event level notification to the emergency notification protocol will be tested. The drill will culminate with test emergency notification messages to individuals at risk. The City will coordinate with residents and officials on the ground to ensure messages were sent, as required.

The annual system test will be preceded by a planning meeting of operational stakeholders. After deciding on the date and specifics of the test, an informational mailer will be sent to the individuals at risk. Posts on social media may also prove useful for informing residents of the upcoming test. Following the annual system test, operational stakeholders will organize and hold a postmortem review meeting to assess the system's performance during the drill and propose needed improvements or changes. A proposed change will need required stakeholder buyoff before being implemented. If necessary, the EAP will be updated, as well. Changes to the system, depending on the nature and extent, may require additional mailings, phone calls, and/or public meetings.

The City will review annually the EWSP and EAP to confirm that all information and contacts remain current, updating the documents as needed. In addition, more frequent updates may be required if known staffing changes occur. The EAP will include a program maintenance log where significant changes to both the EWSP and EAP must be tracked.

Maintenance of instruments and IT resources is also important for the EWSP to operate as required. Regular preventive maintenance of instruments should include:

- Replacing damaged or missing parts or housings.
- Testing and calibrating the rain gage.
- Verifying that measured reservoir pool elevations match what is displayed via the online interface.
- Field checking other meteorological sensors for accuracy, re-calibrating or replacing instruments, as necessary.
- Testing and maintaining cables, connectors, antennas, grounding systems, solar panels, and other system components.
- Verifying EWS functionality, including data analysis and warning issuance.
- Verifying functionality of the satellite data transceiver.

We anticipate that maintenance of instruments, telemetry, databases, and other IT resources will be completed by the City's contracted vendor. All maintenance visits, including problems and solutions related to instrumentation, equipment, and IT resources must be documented in the EAP's maintenance log.

Cost

We coordinated with a vendor to estimate approximate costs for implementing an EWS at Hogchute Dam. Estimated costs break down as follows:

- Required EWS feasibility study: $35,000 (one-time cost).
- Stakeholder coordination and EAP update: $20,000 (one-time cost).
- Installation of instruments and setup of EWS: $70,000 (one-time cost).
- Annual service and data transmittal fees: $5,000 (annual cost).
- Annual maintenance of instruments completed by vendor: $6,000 (annual cost).
- Annual testing, education, and outreach: $2,000 (annual cost).
- Annual EWSP and EAP review: $2,000 (annual cost).

With a total estimated upfront cost of $125,000 and annual recurring costs estimated to be $15,000, the estimated present-day cost for setting up and operating the EWS for the next 30 years is about $840,000 (assuming an annual inflation rate of 3% over that time period).

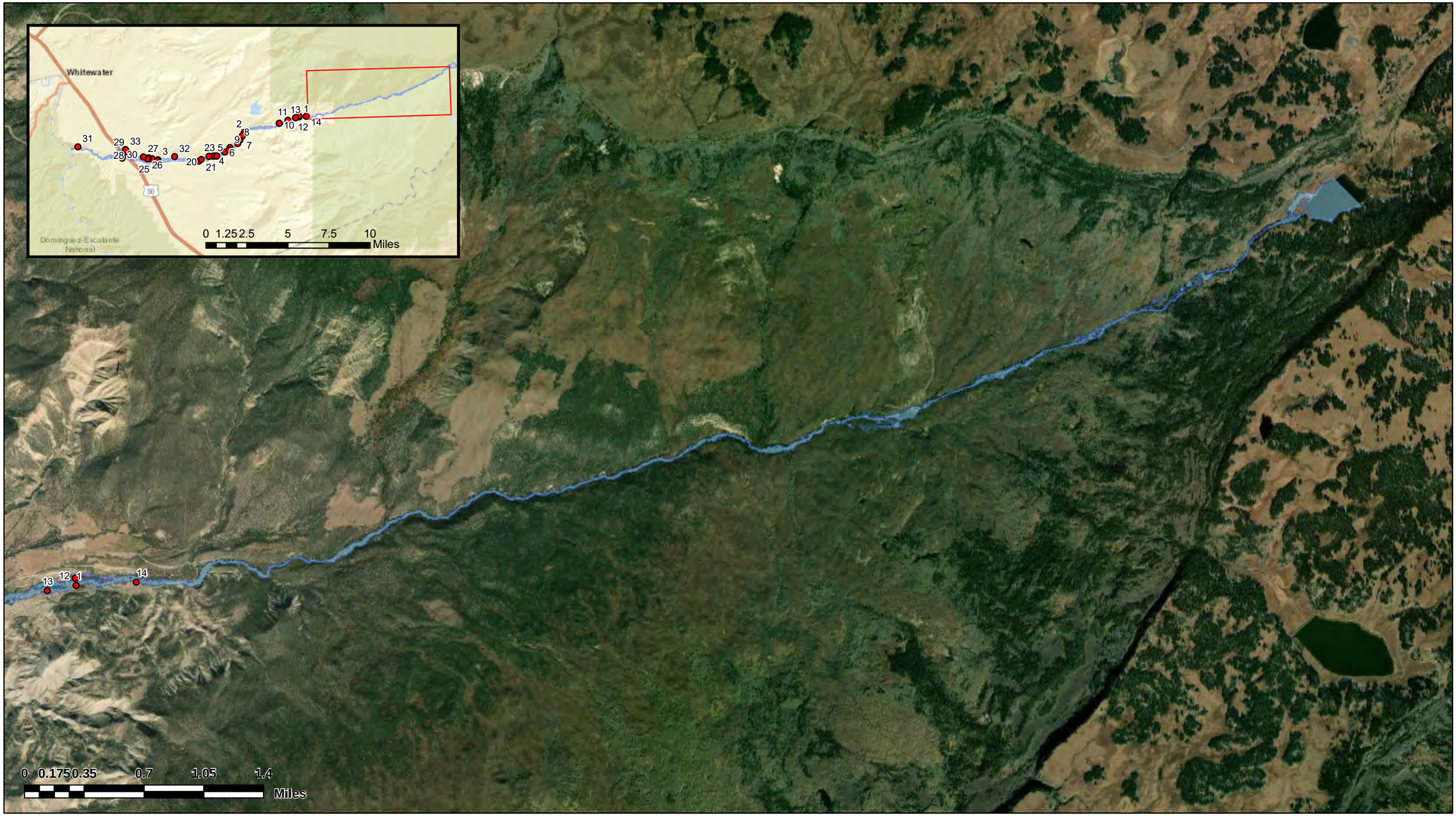
In Appendix C, we have included a budgetary cost estimate provided by OneRain Inc. that includes installation of instruments, setup of the EWS, initial training, and annual fees. Individual sensor costs are also provided. All cost estimates are provided for budgetary purposes only. Actual costs will depend on site conditions, conduit runs, and sensors chosen for implementation.

References

Ayres Associates Inc (Ayres) (2020). *Carson Lake Dam Hazard Classification Study Using the Reclamation Consequence Estimating Methodology*. Prepared for the City of Grand Junction, CO. March.

US Bureau of Reclamation (USBR) (2015). *RCEM – Reclamation Consequence Estimating Methodology (Interim): Guidelines for Estimating Life Loss for Dam Safety Risk Analysis*. US Department of the Interior. July.

Appendix A
Inundation Maps



Legend

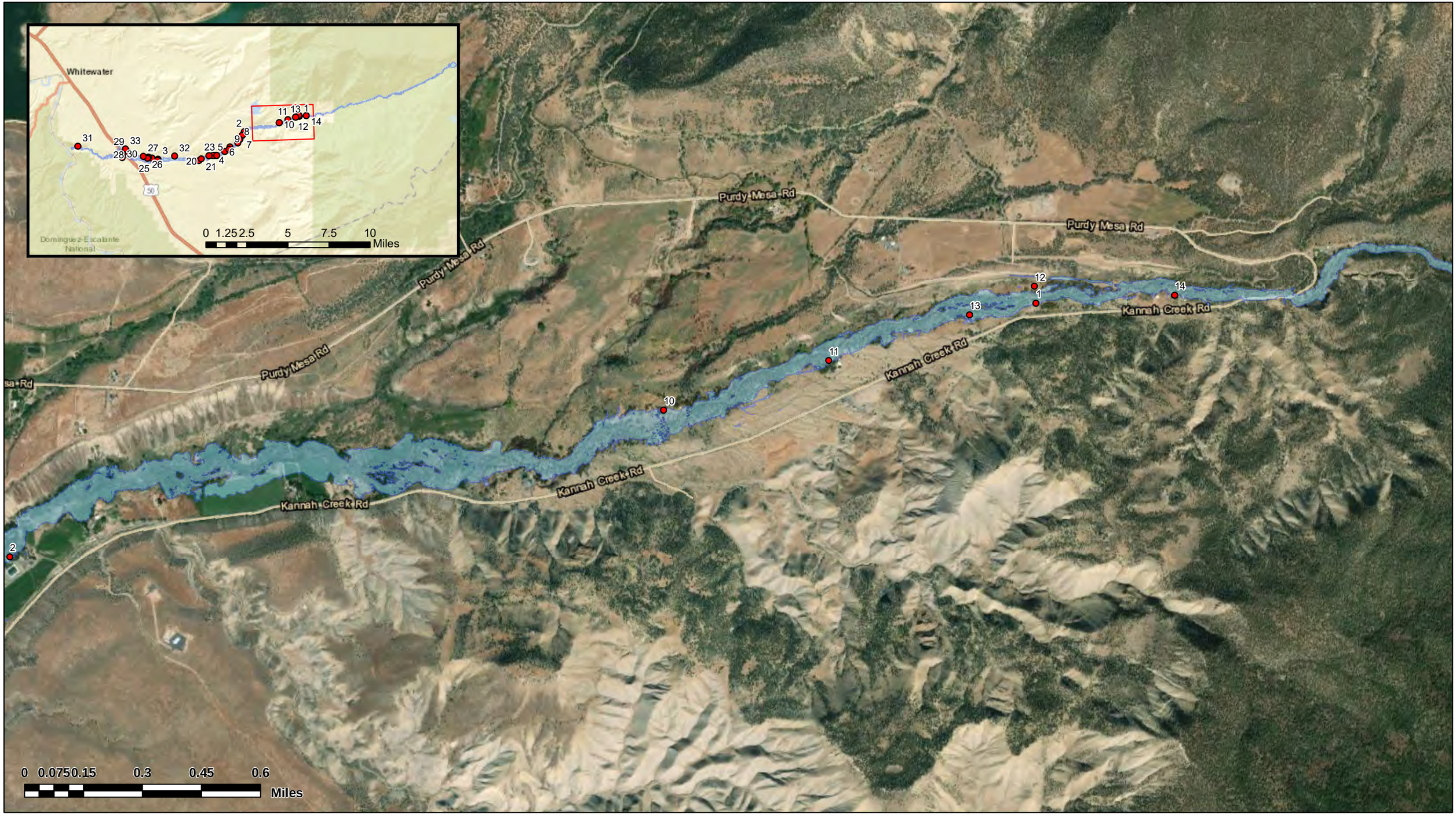
- House Locations
- Maximum Inundation

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 1 of 5
December 15, 2020



Legend

- House Locations
- Maximum Inundation

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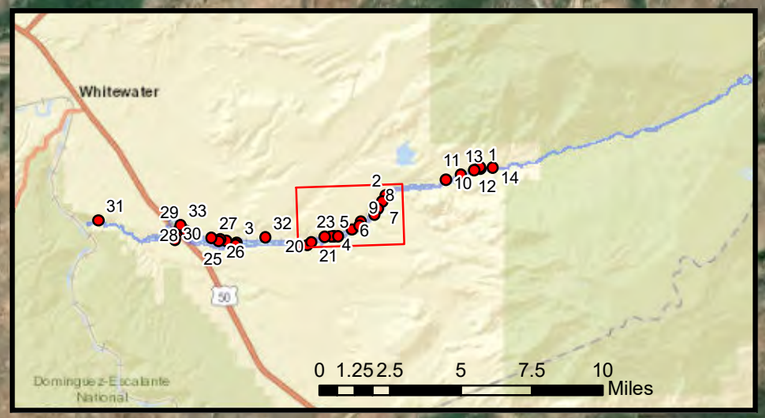
Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 2 of 5
December 15, 2020



Purdy Mesa Rd
Purdy Mesa Rd
Purdy Mesa Rd
Purdy Mesa Rd
Purple Sage Ct
Lands End Rd
Divide Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd
Kannah Creek Rd

0 0.075 0.15 0.3 0.45 0.6
Miles

Legend

- House Locations
- Maximum Inundation

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CITY OF Grand Junction COLORADO

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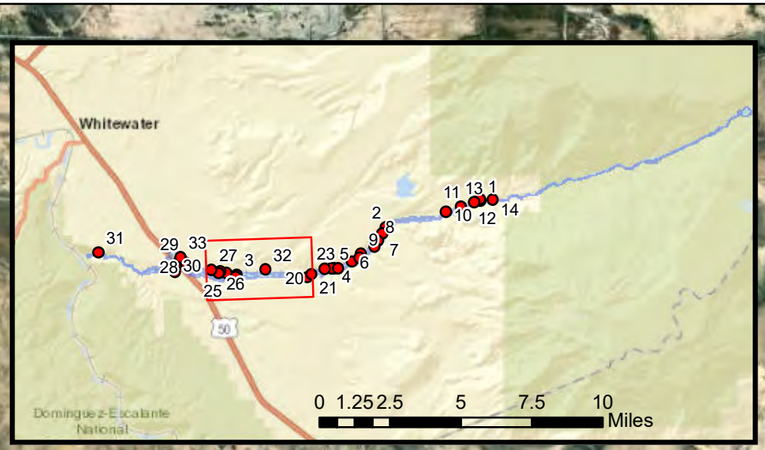
Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 3 of 5
December 15, 2020



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Legend

- House Locations
- Maximum Inundation

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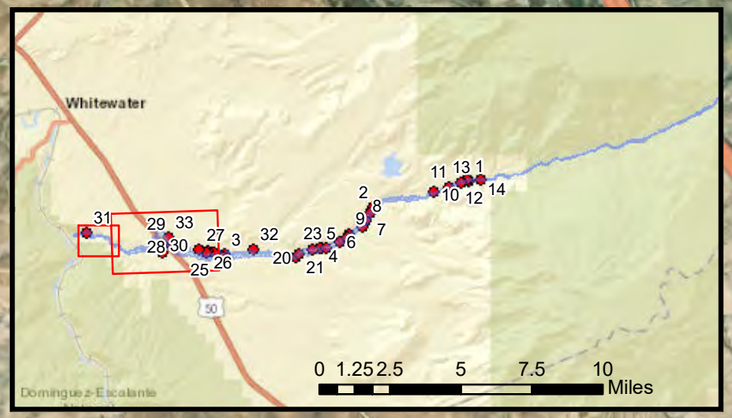
Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 4 of 5
December 15, 2020



Blair Rd
Old Kannah Creek Rd
Simbeck Rd
Pinyon Dr
33
Blair Rd
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16
17
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Kannah Creek Rd
Kannah Creek Rd
Whiting R
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Golden Eagle Ct
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Eagle Trail Ct
Red Tail Ct
Bean Ranch Rd
Eagle Trail Ct
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Miles

Legend

- House Locations
- Maximum Inundation

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CITY OF Grand Junction COLORADO

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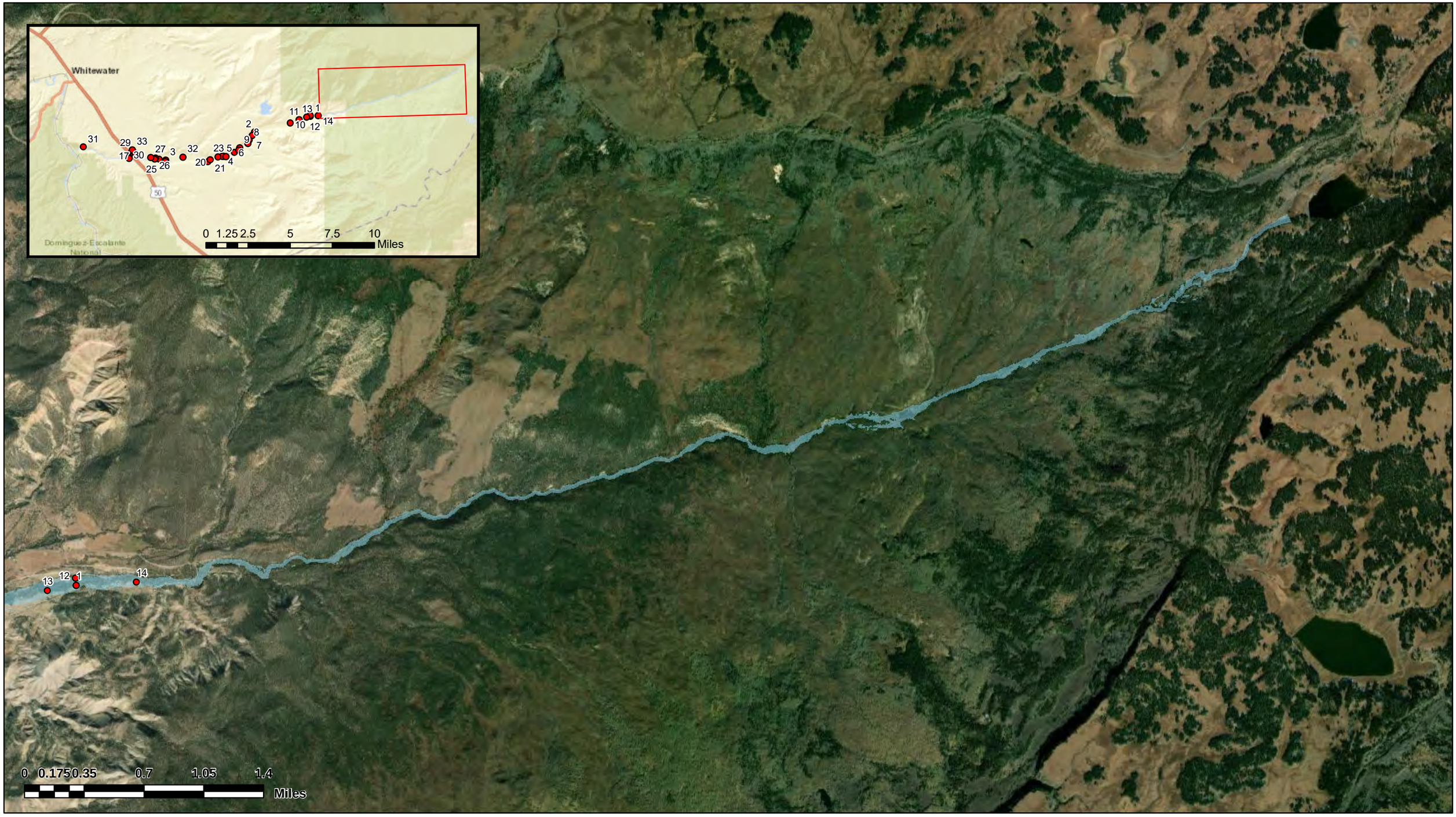
Dam Overtopping (Without Failure) Inundation
Mesa County, Colorado
Sheet 5 of 5
December 15, 2020



Legend

- House Locations
- Maximum Inundation

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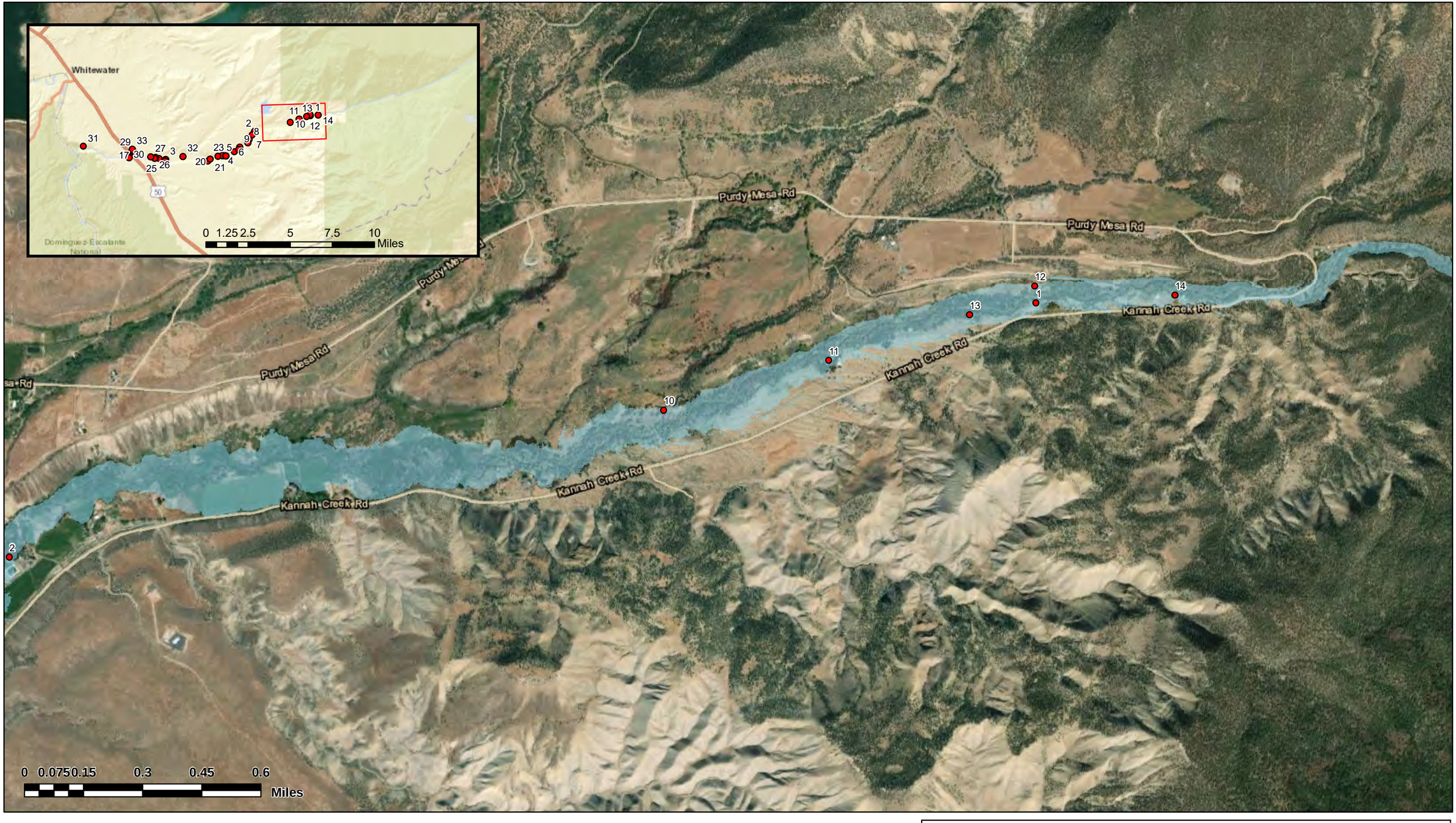
Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam

Mesa County, Colorado

Sheet 1 of 5

December 15, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Legend

- House Locations
- Maximum Inundation

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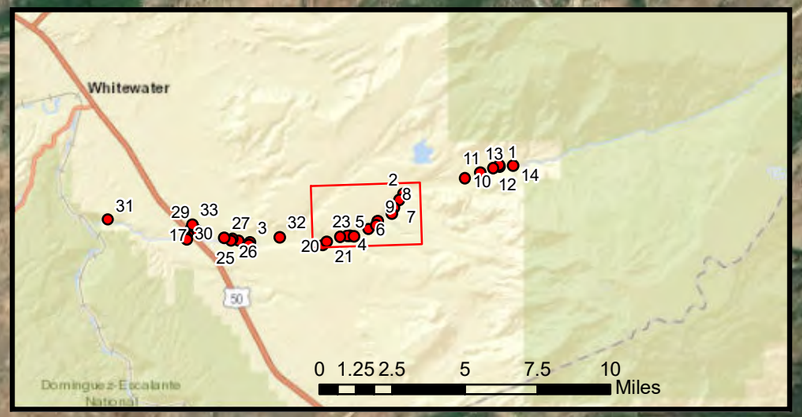
N

Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 2 of 5
December 15, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Legend

- House Locations
- Maximum Inundation

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Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 3 of 5
December 15, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.

Legend

- House Locations
- Maximum Inundation

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Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 4 of 5
December 15, 2020

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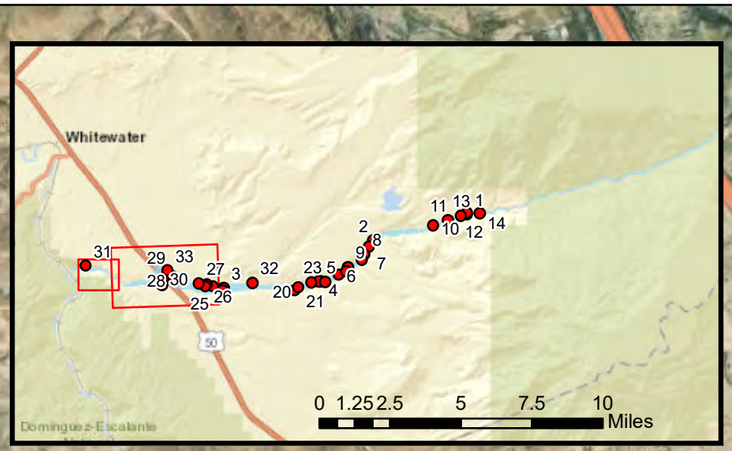

Old Kannah Creek Rd
Blair Rd
Crane
Pronghorn Dr
Kannah Creek Rd
Whiting R
Golden Eagle Ct
Red Tail Ct
Eagle Trail Ct
Bean Ranch Rd
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Miles

Legend

- House Locations
- Maximum Inundation

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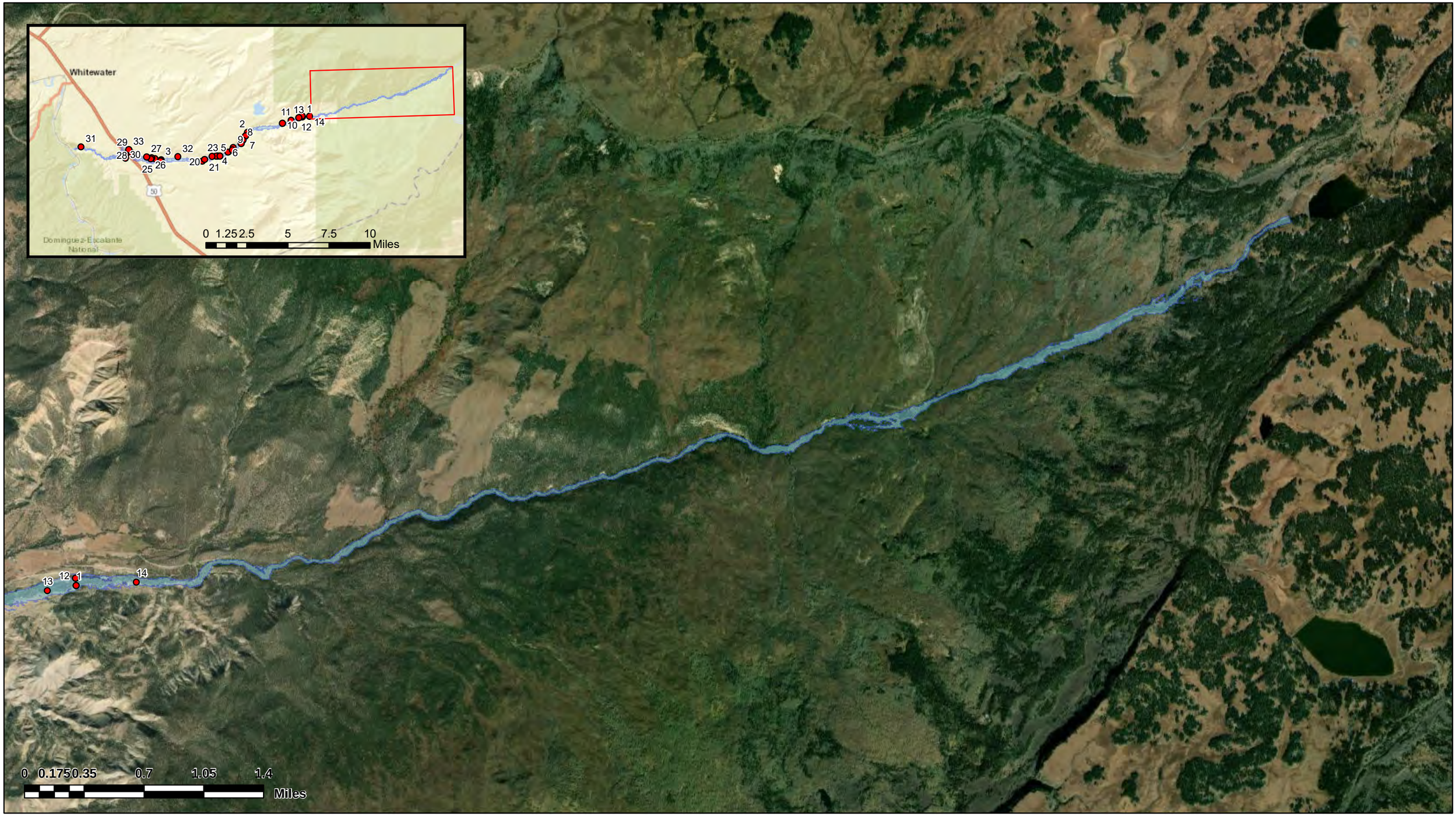
N

Maximum Flooding Depth, Overtopping Failure of Carson Reservoir Dam
Mesa County, Colorado
Sheet 5 of 5
December 15, 2020

Note: Maximum depth and maximum velocity are not necessarily simultaneous. Maximum depth x velocity (DV) is independent of maximum depth and maximum velocity.



Legend

- House Locations
- Maximum Inundation

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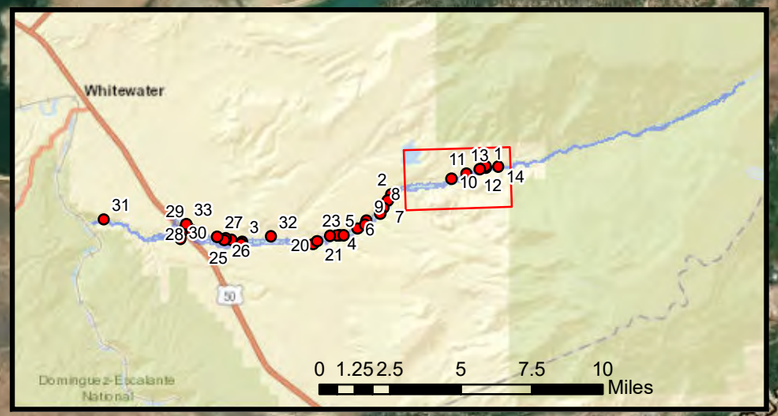
CITY OF Grand Junction COLORADO

N

Sunny-Day Failure Inundation
Mesa County, Colorado
Sheet 1 of 5
December 15, 2020



Purdy Mesa Rd

Kannah Creek Rd

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Legend

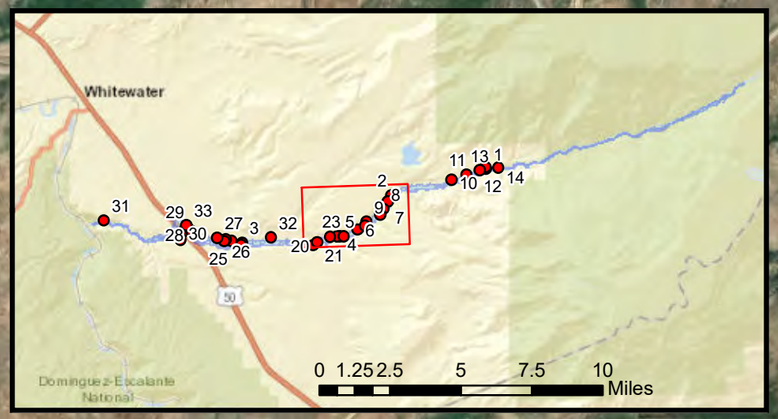
- House Locations
- Maximum Inundation

RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Sunny-Day Failure Inundation
Mesa County, Colorado
Sheet 2 of 5
December 15, 2020



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Kannah Creek Rd
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Miles

Legend

- House Locations
- Maximum Inundation

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

N

Sunny-Day Failure Inundation
Mesa County, Colorado
Sheet 3 of 5
December 15, 2020

Legend

- House Locations
- Maximum Inundation

AYRES RIVER ENGINEERING + WATER RESOURCES

CITY OF Grand Junction COLORADO

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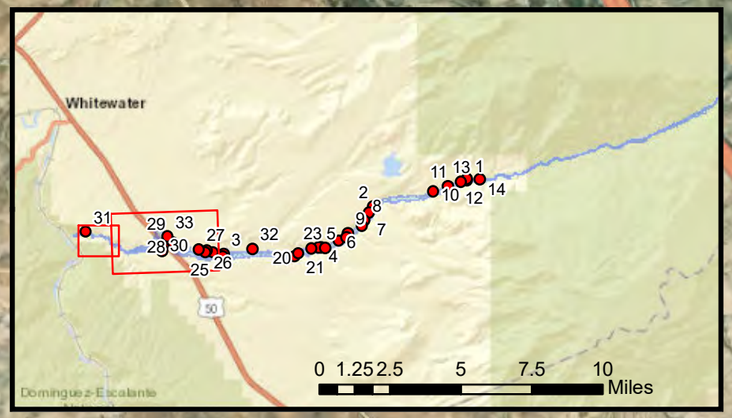
Sunny-Day Failure Inundation
Mesa County, Colorado
Sheet 4 of 5
December 15, 2020



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Old Kannah Creek Rd
Simpson Rd
Pinyon Dr
Blair Rd
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Eagle Trail Ct
Red Tail Ct
Eagle Trail Ct
Bean Ranch Rd
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Miles

Legend

- House Locations
- Maximum Inundation

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CITY OF Grand Junction COLORADO

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Sunny-Day Failure Inundation
Mesa County, Colorado
Sheet 5 of 5
December 15, 2020

Appendix B

Documentation of Stakeholder Coordination

MEMORANDUM

To:	Hogchute Dam Early Warning System Program (EWSP) Stakeholders		
From:	Hogchute Dam EWSP Design Team		
Date:	December 10, 2020	Project No.:	26-1144.00
Re:	List of stakeholder comments and responses		

Below is a list of stakeholders who were engaged in the development of the study. These stakeholders were given copies of the 90% draft report to review and comment on and were invited to the two stakeholder meetings. Meetings were held remotely on November 9 and December 2, 2020.

- Randi Kim – City of Grand Junction, Utilities Director.
- John Eklund – City of Grand Junction, Project Manager.
- Mark Ritterbush – City of Grand Junction, Water Services Manager.
- Slade Connell – City of Grand Junction, Water Supply Supervisor.
- Gus Hendricks – City of Grand Junction, Emergency Manager.
- Korey Kadrmas – State of Colorado, Dam Safety Branch.
- Jackie Blumberg – State of Colorado, Dam Safety Branch.
- Bobbie Lucero – State of Colorado, Division of Homeland Security and Emergency Management.
- Drew Petersen – State of Colorado, Division of Homeland Security and Emergency Management.
- Bob Morris – Colorado Parks and Wildlife.
- Andrew Martsolf – Mesa County, Emergency Management Coordinator.
- Matt Lewis – Mesa County, Sheriff.
- Aldis Strautins – National Weather Service, Warning Hydrologist.
- Jeff Colton – National Weather Service, Warning Meteorologist.
- Loren Paulson – US Forest Service, Realty Specialist.
- Adam Schneider – Ayres Associates.
- Dale Mathison – Ayres Associates.
- Chris Goodwin – Ayres Associates.
- Ellen Faulkner – Ayres Associates.

We provide a compilation of comments received regarding the 90% draft of the Hogchute Dam Early Warning System Program Feasibility Study on the following pages. Responses for each comment are also provided.

920.498.1200 | 3376 Packerland Drive | Ashwaubenon, WI 54115
www.AyresAssociates.com

No.	From	Comment	Response
1	Korey Kadrmas (DSB) during 11/9 stakeholder meeting	Korey Kadrmas noted that the presentation summarized the program at this stage, and that current technology is expected to offer adequate warning capability for protecting downstream residents. He also stated that the Emergency Action Plan (EAP) will need to be restructured to incorporate the EWSP and emphasized the importance of EAP stakeholder input and concurrence.	DSB has indicated that they will not be producing a new EAP template to be used for dams with EWSPs in place. Their thinking is that each dam's EWSP will be so unique that creating a special, general EAP template to be used at all EWSP dams is not feasible. Therefore, when the EAP for Hogchute Dam is updated to include the EWSP, it will be up to the City to decide how best to tailor the EAP for this need. Korey mentioned during the 12/2 stakeholder meeting that DSB will certainly be available to provide feedback and assistance during the EAP update process.
2	Jackie Blumberg (DSB) during 11/9 stakeholder meeting	Jackie Blumberg reiterated the importance of emergency management buy-in for the program and urged the City and stakeholders to carefully evaluate the physical components of the system considering the key functions and desired outcomes of the program.	Noted. Based on our understanding, DSB will not consider approving the EWSP until stakeholder buy-in is achieved. From the DSB's perspective, what would officialize stakeholder buy-in? During the12/2 meeting, Jackie stated that formal approval (e.g., signatures) of the EWSP is not needed. DSB would just like to see documentation that the operational stakeholders were given an opportunity to review the EWSP feasibility study and provide questions and comments. We will provide a list of participating stakeholders and this comment-response form as an attachment to the EWSP feasibility study.
3	Jackie Blumberg (DSB) during 11/9 stakeholder meeting	Jackie also reminded Ayres and the City to provide inundation shapefiles to Mesa County Emergency Government and the State Department of Emergency Management.	We will provide these shapefiles, as requested.

No.	From	Comment	Response
4	Loren Paulson (USFS) during 11/9 stakeholder meeting	Loren Paulson questioned whether the proposed installations raise NEPA issues, and also suggested including contacts from the Forest Service's recreation division and Colorado Parks and Wildlife because of potential risks to recreational visitors.	At this time, we envision that most to all the instrumentation will be installed within the footprint of the existing dam structure and appurtenant works, so we do not anticipate NEPA issues. However, when final instrument staging locations are selected, plans will be provided to USFS for approval to ensure that NEPA concerns are addressed. The updated EAP can include closure of the dam site when hazardous event levels are triggered. Closing of additional sites downstream can also be considered, if feasible. See response for comment number 18 below. During the 12/2 meeting, Andy Marsolf weighed in that trail closure signs are frequently ignored by the public, so closing trails in anticipation of a flood would likely have no impact. During the 12/2 meeting, Jackie Blumberg reiterated that we are trying to provide warning for hydrology-driven events. These events do not occur out of the blue and will have some natural, escalating warning associated with them. Therefore, the risk to downstream recreational trail users seems low.
5	Gus Hendricks (City of Grand Junction) during 11/9 stakeholder meeting	Gus Hendricks indicated he will work with Andy Martsolf to maintain inundation zone mapping and warning procedures. He asked that the program included pre-scripted messages for the emergency levels requiring downstream notification.	The updated EAP will include pre-scripted messages for downstream notifications. During the 12/2 meeting, Korey stated that pre-scripted messages should be included in the updated EAPs for dams with EWSPs.

No.	From	Comment	Response
6	Aldis Strautins (NWS) during 11/9 stakeholder meeting	Aldis Strautins pointed out that the quantitative precipitation forecast (QPF) proposed for use in the "forecast-based" emergency level does not differentiate between rain and snow. He asked whether the satellite-transmitted monitoring data could be accessed in real time by NWS. Also, he asked whether the operation of the outlet works without a significant pool level rise above the spillway crest could pose a hazard. Adam Schneider responded that the maximum capacity of the outlet works is approximately the 10-year flood, which is not expected to present a significant downstream flooding hazard. Aldis also suggested a downstream gage to verify outflows, and Korey Kadrmas noted that the Dam Safety Branch has made the same request to improve redundancy and verification. City staff agreed that a downstream gage makes sense.	See responses for comments 10, 12, and 13 below.
7	Aldis Strautins (NWS) via email on 11/13	Page 15: event level notifications will be sent via text message. - The NWS does not receive text messages into our operation center. Will there be another way to transmit these notifications?	Notifications can also be sent via email to designated NWS individuals or offices.
8	Aldis Strautins (NWS) via email on 11/13	Page 16: "The NWS warning meteorologist." Would like this changed to the NWS operations. Operations are open 24/7 and they can contact both the WCM and the Hydrologist. Will need to also update the EAP with current WCM and better wording for the operations center.	We will make this change as requested.
9	Aldis Strautins (NWS) via email on 11/13	What Satellite transmission system will be used. Will it use GOES?	The details regarding satellite transmission likely depend on the vendor that the City selects for their EWS. The vendor who has been assisting us with this EWS planning study, OneRain, uses Orbcomm satellites for data transmission. The system transmits in near real time with notifications and alarms being sent within 90 seconds.

No.	From	Comment	Response
10	Aldis Strautins (NWS) via email on 11/13	Will the NWS at Grand Junction be able to get the data into our Warning Computer system? (AWIPS - Advanced Weather Interactive Processing System)	Yes, we suspect this will be feasible. As an example, OneRain proposed that they could send data directly to the NWS via a SHEF feed, or the data could be sent to MADIS, which could then be pulled down for local display at NWS offices.
11	Aldis Strautins (NWS) via email on 11/13	So the EWS system will send a notification to the owner and other partners and then the dam owner in coordination with the emergency management will activate the EAP and notifications of residences below the dam? Will the early warning system send out text / email messages to the people/residences below the dam?	Your understanding is consistent with plans to date. And yes, the EWS can be configured to send out text/email messages directly to the people at risk below the dam. However, given spotty cell coverage in that area, the primary means to warn individuals downstream of the dam will be through landline phone calls (reverse 911).
12	Aldis Strautins (NWS) via email on 11/13	Consideration of a river gauge downstream of the dam should be discussed. Pool level only with rating calculations for flow downstream could work in many situations but I'm not sure it covers some possible scenarios where a downstream river gauge would help identify. What type of release can be done? Is there flooding only from a spilling situation or can it flood due to a max release through the gates?	We are currently modifying the plan to include either two float switches below the dam (a lower float switch would indicate higher than usual flows, and a higher float switch would indicate a serious problem) or a pressure transducer water level sensor in the stream below the dam. If the reservoir's gated outlet is operated fully open, it can release flows comparable in magnitude to a 10-yr flood event. However, the City does not release flows from the reservoir that even approach this magnitude. During the 12/2 meeting, Korey stated that the implementation of a water level sensor downstream of the dam will also afford an opportunity to assess seasonal patterns at the site.

13	Aldis Strautins (NWS) via email on 11/13	Page 15: How will the City's EWS vendor get the QPF data from the NWS. Will they pull it from our data services or are they expecting us to push a product to them? Is the QPF in the form of rain or snow. 3 inches of rain is much different than 3 inches equivalent liquid of snow. I heard something about a SHEF product, I could use some more details to help my understanding.	This aspect of the EWS will require close coordination and planning between the EWS vendor and the NWS. To date, OneRain is not issuing warnings based on NWS forecasts for any of their clients, so this would be a new application for them. We are not sure if other vendors are currently issuing warnings in this manner either. In my own (Adam Schneider's) past project work for the USACE, the NWS would pass off forecasts to the USACE using SHEF files. The SHEF files would then be decoded by USACE software routines, and the forecasts would then be posted to a USACE database for use by their models. In discussions with OneRain, they believe a similar workflow would work for the Hogchute EWS application with some development effort required on their end. However, for Hogchute, we anticipate that the EWS vendor would pull the SHEF files from NWS data services. We do not foresee a workflow where the NWS would be required to regularly send forecast products to the City or EWS vendor. The distinction between snowfall and rain in the QPF is a good point. Perhaps both QPFs and forecasted freezing levels or temperatures could be incorporated into the warning logic? During the 12/2 meeting, Aldis mentioned that the warning logic could be refined, if needed, using not only QPF and air temperature forecasts, but dew points, as well. At this point, it is unclear what level of refinement is needed, however. Aldis stated warnings based on QPFs and air temperature may be enough for a starting point. During 12/2 meeting, Korey mentioned that seasonality also will play a role in the distinguishing of snow vs. rain. Clearly during summer months, when flood risk is highest, snow is not much of a

No.	From	Comment	Response
			concern. Thus, a high level of refinement in parsing out snow vs. rain may not be necessary.
14	Aldis Strautins (NWS) via email on 11/13	NWR - NOAA Weather Radio can be used to send out warnings. In addition to the code red or reverse 911.	Noted. The updated EAP can include warnings being sent via NOAA Weather Radio. Furthermore, persons at risk can be equipped with weather radios to use as a backup to the reverse 911 system.
15	Loren Paulson (USFS) via email on 11/30	It would be interesting to see/include a potential flood map of the section of Kannah Creek below the dam and above the intake.	Flood maps currently do not show this area because no habitable structures are located along this segment of the river. However, we can add a panel(s) to the maps to include this area.
16	Loren Paulson (USFS) via email on 11/30	Where will the instrumentation be located and how will it be protected from the public?	Exact locations are to be determined, but we envision a primary monitoring station near the top of dam and a secondary monitoring station below the dam. The primary station will be connected to the reservoir pool level sensors, the precipitation gauge, and will house the data logger and satellite telemetry equipment. The secondary station will be connected to float switches and/or a pressure transducer monitoring water levels in the creek below the dam. The data logger in the secondary station will connect to the primary station for satellite data transmittal. If vandals are determined to destroy equipment, there is little that can be done to stop them, especially in such a remote location. However, the stations will be armored to make vandalism difficult. Both primary and secondary stations will be enclosed in bullet-resistant enclosures, and intrusion sensors will be installed in each.
17	Loren Paulson (USFS) via email on 11/30	What happens to the instrumentation during the winter?	Instruments will remain operational throughout the year. They are designed for operation in all weather conditions.

No.	From	Comment	Response
18	Loren Paulson (USFS) via email on 11/30	A recognition that there may be people around the dam and along the creek/trails in the basin. The only trail that actually crosses the creek is the Farmers Trail (near Sheep Creek). That said, I am not sure how you could or be expected to develop a warning system to address that use.	The updated EAP will ensure that public access to the dam is closed when hazardous conditions are identified (i.e. Potential Dam Failure or Failure in Progress/Unavoidable event levels). Furthermore, the EAP can also include closure of other access points below the dam (such as the Farmers Trail) if closure is determined to be feasible. During the 12/2 meeting, the stakeholder group generally concluded that warning recreational users downstream of the dam likely would not be necessary or useful in the event of a hazardous hydrologic situation. The group identified two reasons supporting this conclusion. First, warnings (e.g. trail closure signs) are routinely ignored. Second, hydrologic events take place with some advanced warning, and recreational trail users would not be expected during the poor weather conditions associated with and leading up to these events.
20	Korey Kadrmas (DSB) during the 12/2 meeting	DSB is gaining comfort with pressure transducers to measure water levels. They are OK with using either bubblers or pressure transducers in the reservoir.	Currently the plan is to use a bubbler as the primary water level sensor in the reservoir and a pressure transducer as the backup sensor. (Our research indicates that pressure transducers are not as costly as bubblers.) In the creek below the dam, a pressure transducer is proposed for measuring water levels.
21	Jackie Blumberg (DSB) during the 12/2 meeting	Make sure precipitation totals in Table 10 are from MetPortal	Confirmed. The precipitation totals in Table 10 were extracted from MetPortal.
22	Jackie Blumberg (DSB) during the 12/2 meeting	Jackie would like to see that multiple City officials be notified in the event of warnings.	We have modified the plan such that in the case of a Potential Dam Failure or Dam Failure in Progress or Unavoidable event level warning, multiple City staff are notified.

No.	From	Comment	Response
23	Korey Kadrmas (DSB) during the 12/2 meeting	Include list of stakeholders involved in the process of reviewing the feasibility study document and/or have attended the two meetings.	A list of stakeholders involved in the review process will be included as an attachment to the report.
24	Jackie Blumberg (DSB) during the 12/2 meeting	Include instrumentation sheet in the planset	Plans and specifications for EWSP instrumentation will be included in the design submittal.

Appendix C
Supplemental Cost Information

The following budgetary cost estimate was provided by OneRain Inc. on November 13, 2020. Note that actual costs may vary depending on site conditions, conduit runs, and instruments selected.

	Description	Qty	Total
1.	**Contrail® Server 20** [1] 12-Month managed, and hosted application service subscription includes: - Contrail Base Station - Contrail Inventory *plus* Maintenance - Contrail Insight - Input sensor data stream based on 20 unique sensor IDs	1 12 mo.	**$4,500**
2.	**Contrail Server, Implementation & Training**[2] One-time cost includes: - Generation/setup of one (1) map - Setup and initial customization of website - Webinar On-line Training Session[2] (End User and Administrator)	1	**$3,500**
3.	**StormLink® Monitoring Station** Satellite configuration - Bullet Resistant Enclosure - FTS Bubbler - Pressure Transducer - Rain Gauge - Data logger - Enclosure with Intrusion sensor - Solar panel, battery, and regulator - Integration, Testing & Prep - Includes Shipping	1	**$29,000**
4.	**StormLink® Monitoring Station – Second Site** - Bullet Resistant Enclosure - (2) Float Switches - Data logger w/spread spectrum to connect to Master - Enclosure with Intrusion sensor - Solar panel, battery, and regulator - Integration, Testing & Prep - Includes Shipping		**$15,400**
5.	**Equipment Installation (Estimate only)** - Two-day installation by Two OneRain Field Technicians - Includes travel, hotel and meals	1	**$12,700**
6.	**StormLink Satellite Data Services Fees**[3] **(1 Site)** - StormLink Satellite Services Transmission Base Usage Fee up to 10 Kilobytes (kB): $39 per month per site. - Minimum Contract Term: 12 months	1 12 mo.	**$468**
	First Year Total Hardware + Data Services		**$65,568**

After First Year		
Annual Recurring Data Services Fees	**Qty**	**Total**
Contrail Web annual subscription[1]	1 12 mo.	$4,500
StormLink Satellite Data Services (1) site[3] Transmission Base Usage Fee up to 10 Kilobytes (kB)	1 12 mo.	$468
Total Annual Fees		**$4,968**

NOTES

[1] ***Contrail Server 20*** *Contrail Server is a managed application service operated by OneRain and provided on a fixed annual subscription fee basis. Contrail Server operates Contrail software exclusively and resides in a secure industrial data center. OneRain manages and administers server OS updates, and performs all Contrail software application updates and new releases.*

[2] ***Webinar-based training*** *requires that trainees have a computer with an Internet connection and a web browser to attend. For audio, a speaker phone or computer's audio system (VoIP) may be used. A group of participants may be assembled around a single computer (or LCD projector and screen connected to a computer).*

[3]***Satellite Data Services Important Note****: The minimum annual StormLink IDP Usage Fee is $468 per site for up to 10 Kilobytes (kB) of data per site per month. Usage above this allotment is $2.00 per kB per site per month and must be paid by customer. OneRain will invoice for the minimum annual data fee at the beginning of each contract period and the overages will be billed throughout the year based on data usage.*

Budgetary individual sensor pricing, as provided by OneRain Inc on October 30, 2020:

Bubbler pricing - $3,600 each

100 ft, 0-10m range pressure transducer - $1,964 each

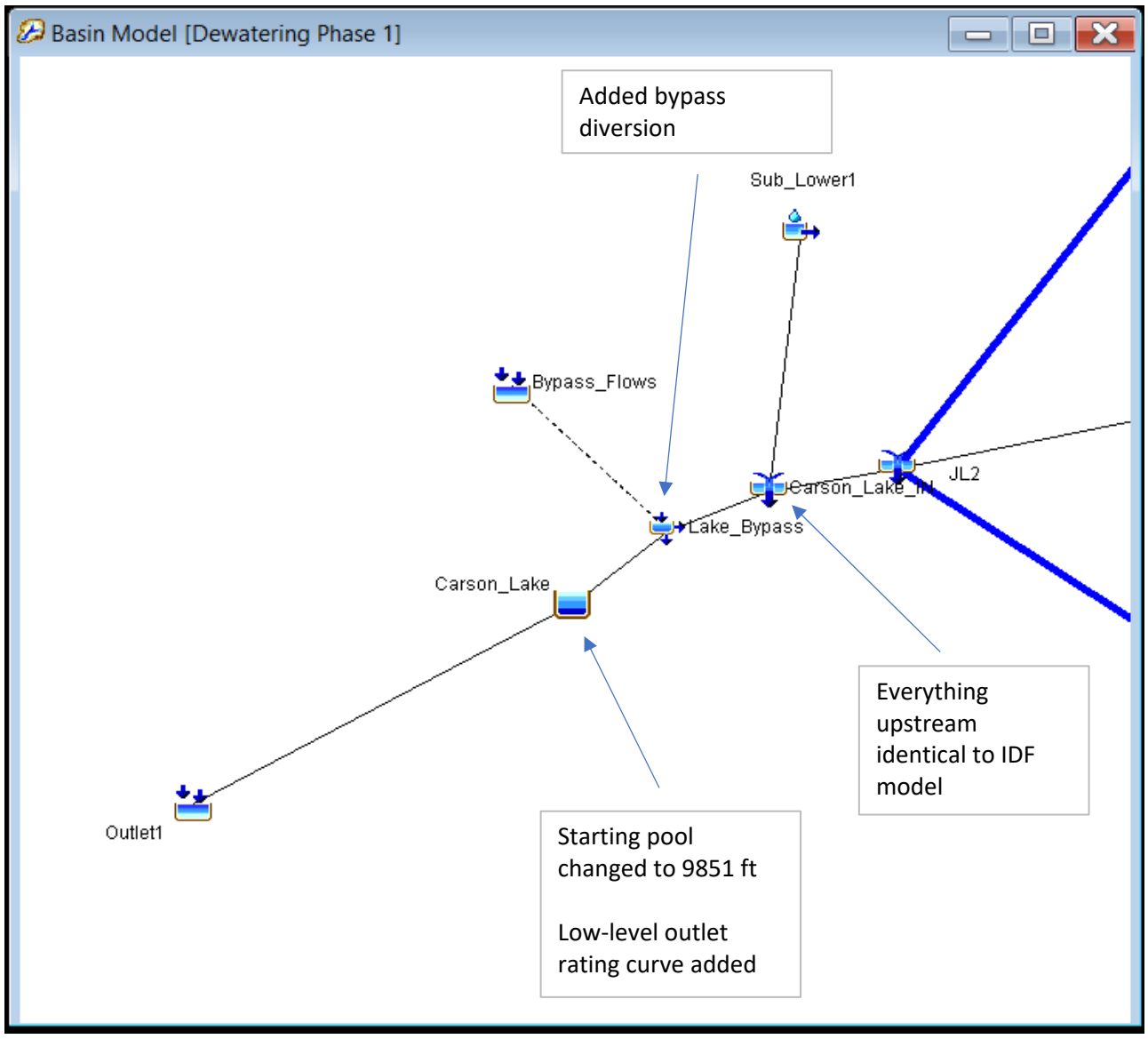
0.01-inch tipping bucket - $1,554 each

Float switch - $269 each

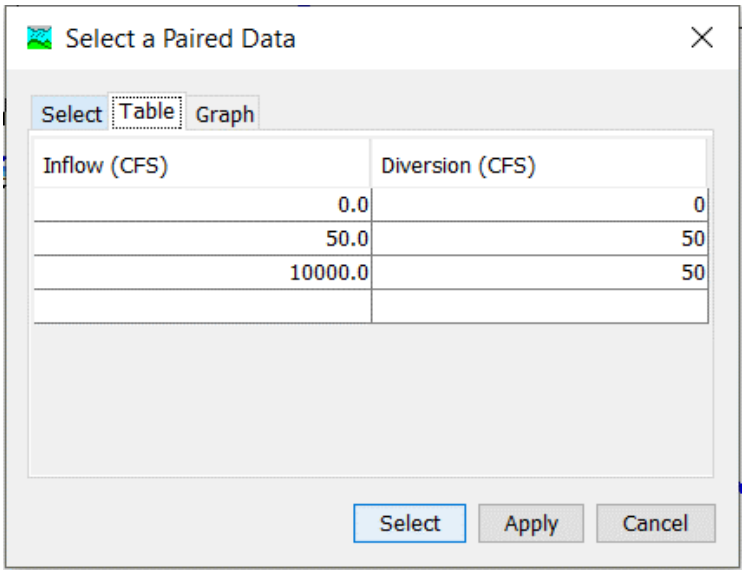
Appendix M
Dewatering Calculations

Description on HEC-HMS Model Setup for Phase 1 Dewatering

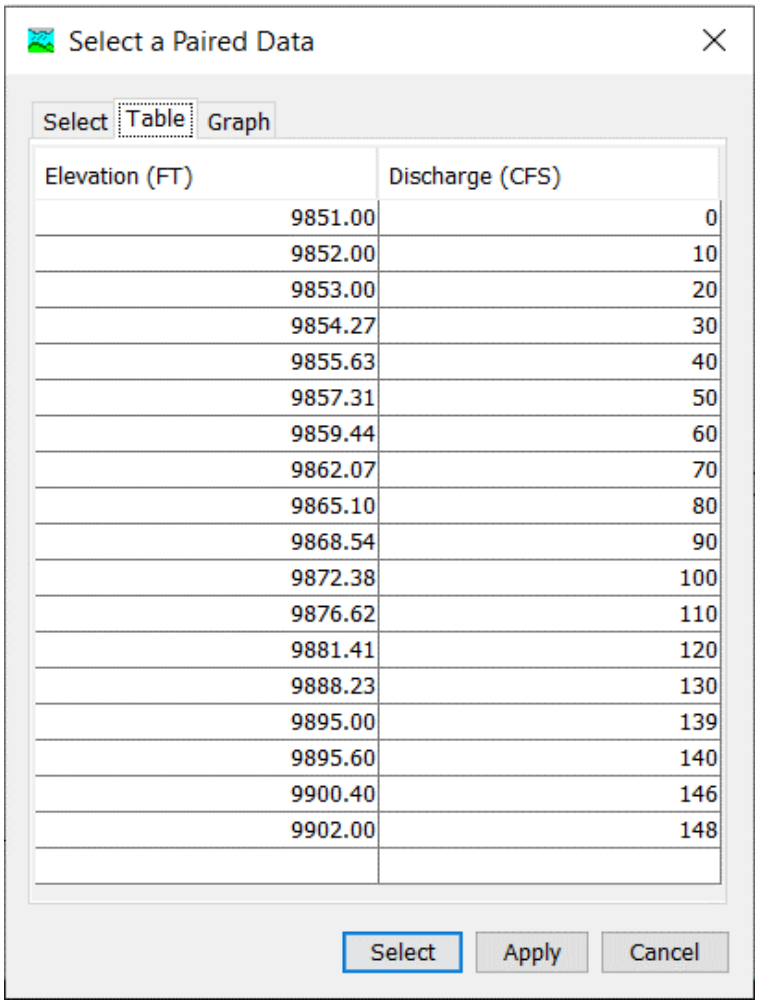
Basin model setup for Phase 1 dewatering configuration (HEC-HMS Basin Model "Dewatering Phase 1")



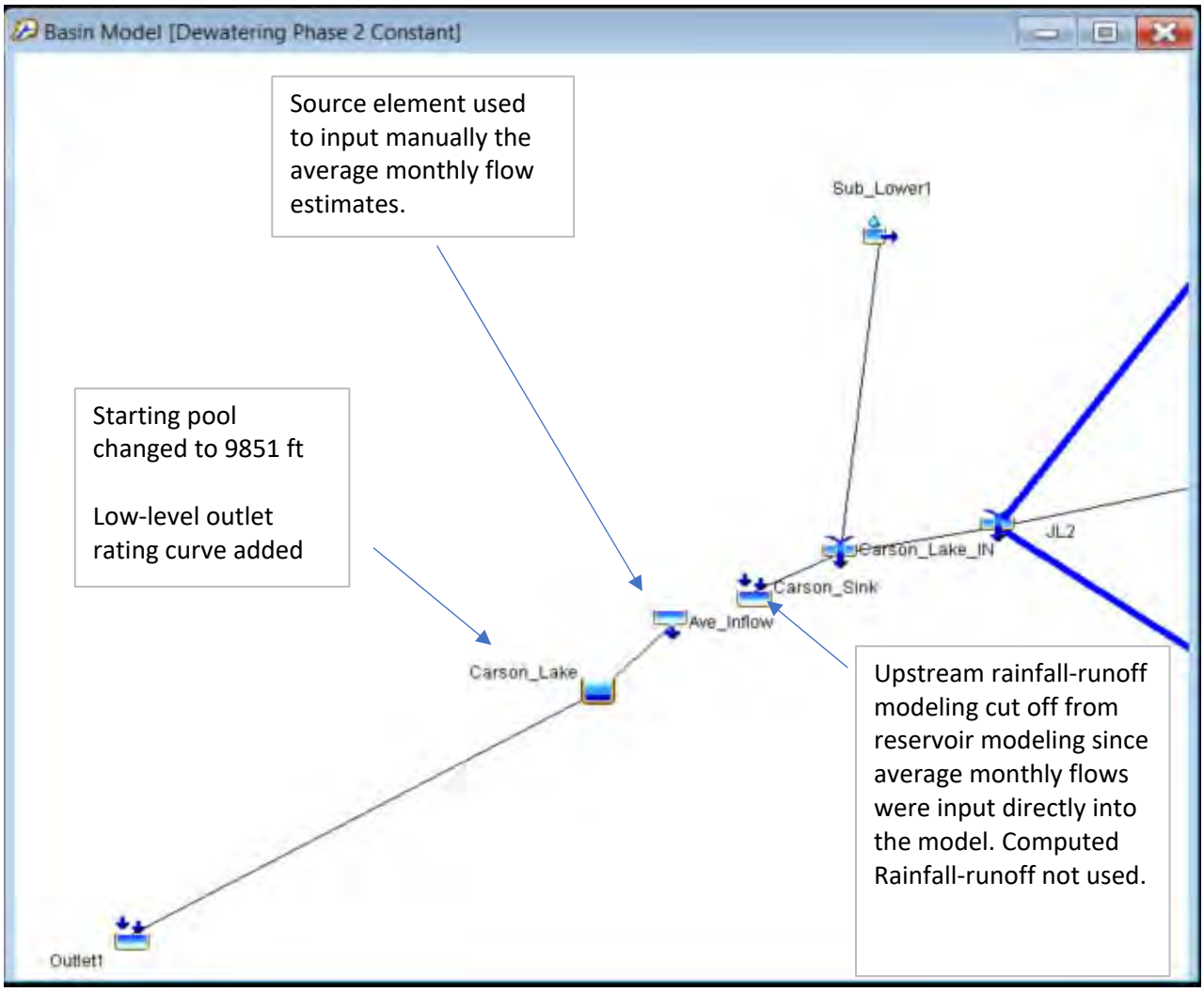
Inflow-diversion function for "Lake_Bypass"



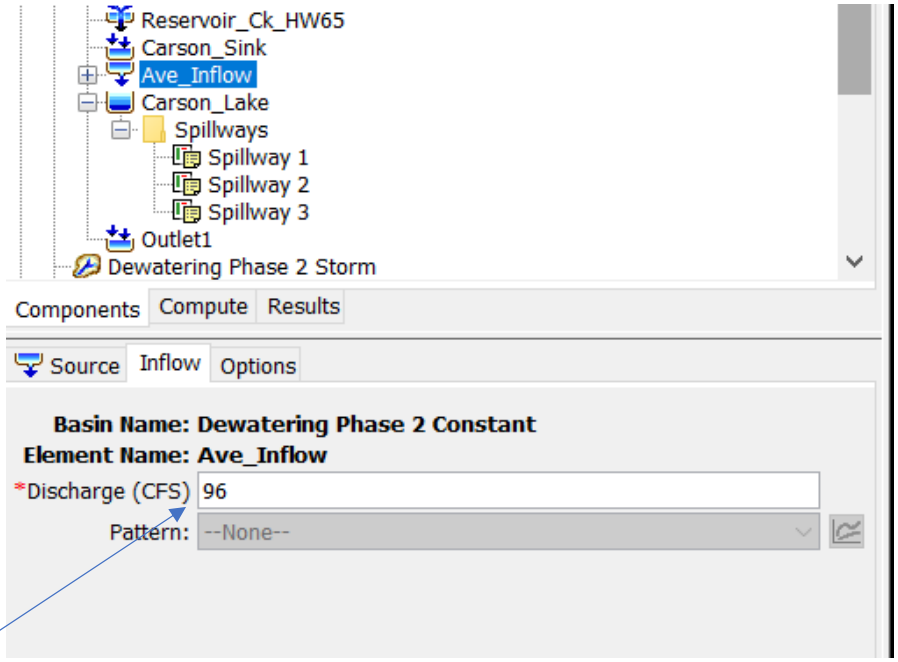
Elevation-discharge curve for 30" relief outlet. Approximated using rating curve for proposed outlet works.



Description on HEC-HMS Model Setup for Phase 2 Dewatering
(Average Monthly Flow Simulations)
HEC-HMS Basin Model "Dewatering Phase 2 Constant"

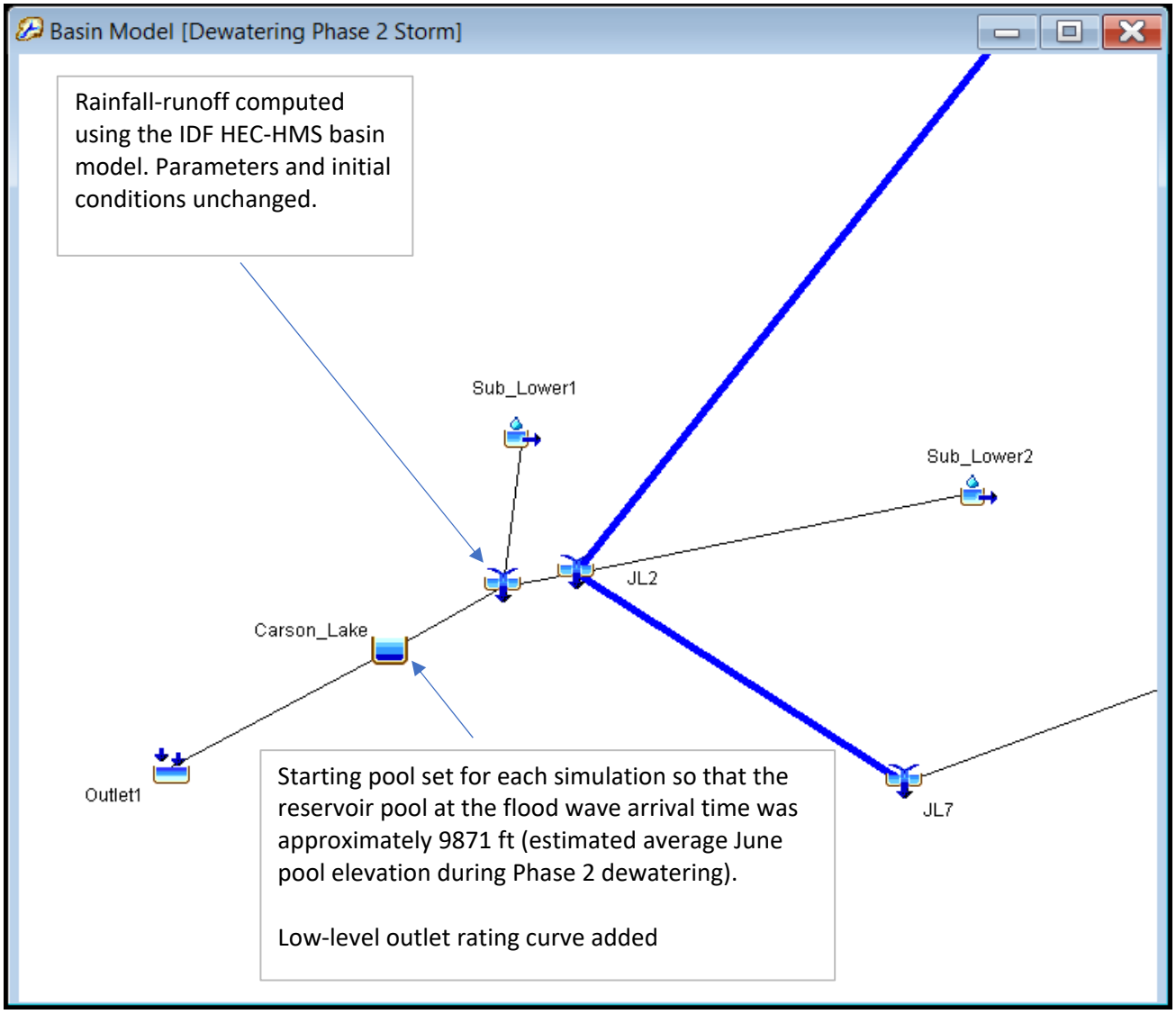


"Ave_Inflow" source element setup in HEC-HMS.

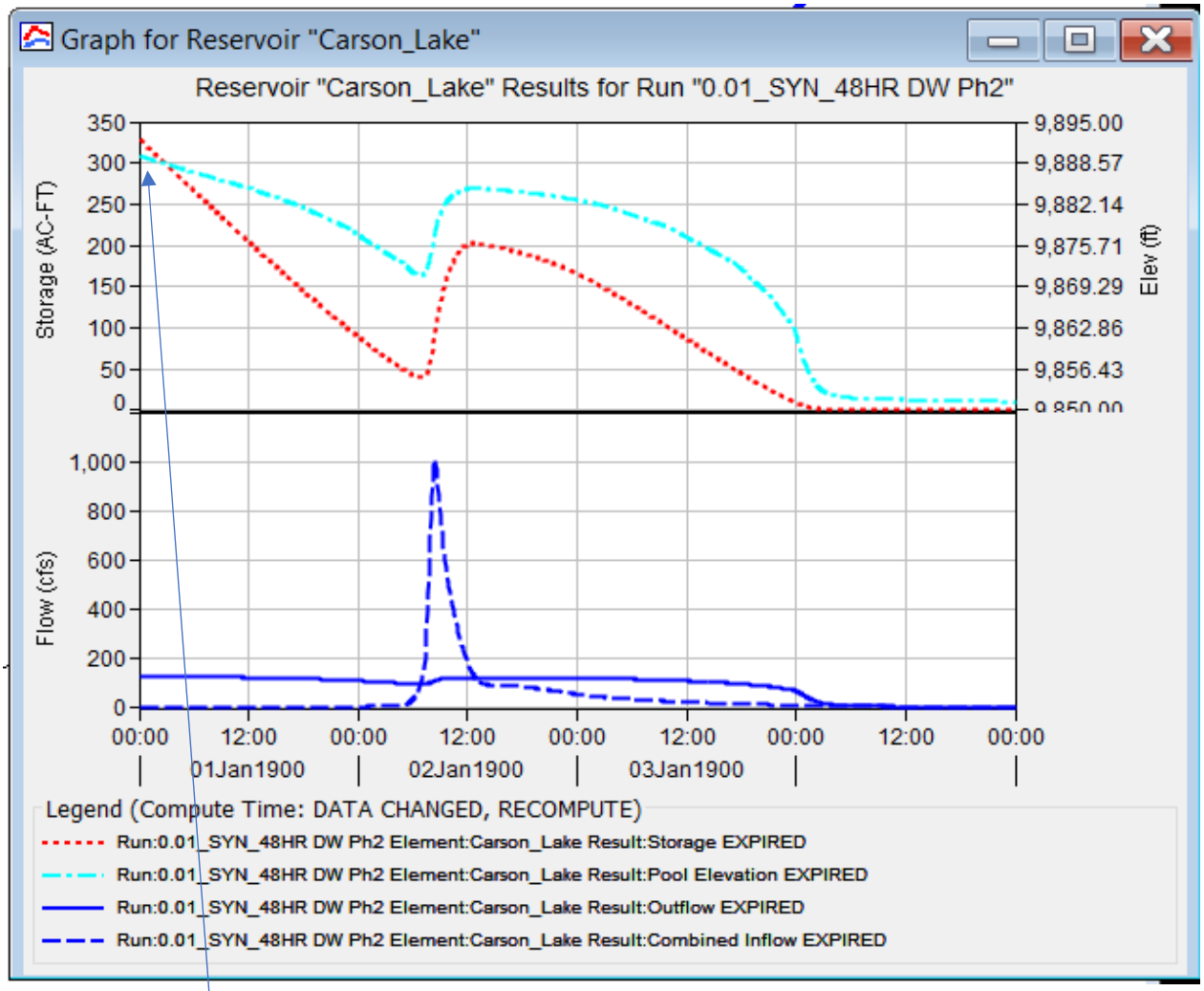


For this run, a constant flow of 96 cfs was input into the model (the estimate for June).

Description on HEC-HMS Model Setup for Phase 2 Dewatering
(Storm Flow Simulations)
HEC-HMS Basin Model "Dewatering Phase 2 Storm"



Example results for the 100-yr, 48-hr simulation during Phase 2 Dewatering



Because the reservoir's low-level outlet was modeled for these simulations, this outlet drew down the reservoir prior to the arrival of the incoming flood wave, as indicated by the downsloping magenta line, which decreases from an initial value of 9889 ft to 9871 ft before the flood wave arrives on day 2 of the simulation. We set a unique initial pool elevation (9889 ft for this example) for each simulation using trial and error to ensure that the pool elevation was approximately 9871 ft (estimated average lake elevation for June) when the flood hydrograph arrived at the reservoir. This was mostly a concern for the simulated 48-hr events.

2-hr Design Storm Precipitation Input for Hogchute Dewatering Simulations
100-yr and 10-yr hyetographs from MetPortal (scaled up by 7 percent)
2-yr hyetograph is the 10-yr hyetograph multiplied by scale factor of 0.925 (to produce a peak inflow to Hogchute consistent with USGS regression equation result)

/COLORADO/CARSON DAM/PRECIP-INC/31DEC1899/5...

File Edit View Help

Incremental precipitation (inches)

Ordinate	Date / Time	100-yr 0.01_SYNTHETI...	10-yr 0.1_SYNTHETIC...	2-yr RUN:0.5_SYN_2HR
Units		INCHES	INCHES	IN
Type		PER-CUM	PER-CUM	PER-CUM
1	31 Dec 99, 24:00	0.0	0.0	
2	01 Jan 00, 00:05	0.0354105	0.0224819	0.02080
3	01 Jan 00, 00:10	0.0522726	0.0331875	0.03070
4	01 Jan 00, 00:15	0.0743467	0.0472022	0.04366
5	01 Jan 00, 00:20	0.13796	0.0875916	0.08102
6	01 Jan 00, 00:25	0.31118	0.19757	0.18275
7	01 Jan 00, 00:30	0.17015	0.10803	0.09993
8	01 Jan 00, 00:35	0.13183	0.0836987	0.07742
9	01 Jan 00, 00:40	0.0996399	0.0632606	0.05852
10	01 Jan 00, 00:45	0.0766461	0.0486620	0.04501
11	01 Jan 00, 00:50	0.0732736	0.0465209	0.04303
12	01 Jan 00, 00:55	0.0657623	0.0417520	0.03862
13	01 Jan 00, 01:00	0.0588642	0.0373724	0.03457
14	01 Jan 00, 01:05	0.0462942	0.0293919	0.02719
15	01 Jan 00, 01:10	0.0406224	0.0257909	0.02386
16	01 Jan 00, 01:15	0.0306584	0.0194648	0.01800
17	01 Jan 00, 01:20	0.0260597	0.0165451	0.01530
18	01 Jan 00, 01:25	0.0223807	0.0142093	0.01314
19	01 Jan 00, 01:30	0.0191615	0.0121655	0.01125
20	01 Jan 00, 01:35	0.0157891	0.0100244	0.00927
21	01 Jan 00, 01:40	0.0130298	0.0082725	0.00765
22	01 Jan 00, 01:45	0.0107304	0.0068127	0.00630
23	01 Jan 00, 01:50	0.0088909	0.0056448	0.00522
24	01 Jan 00, 01:55	0.0073580	0.0046716	0.00432
25	01 Jan 00, 02:00	0.0045988	0.0029197	0.00270
26	01 Jan 00, 02:05	0.0038323	0.0024331	0.00225
27	01 Jan 00, 02:10	0.0030658	0.0019465	0.00180
28	01 Jan 00, 02:15	0.0030658	0.0019465	0.00180
29	01 Jan 00, 02:20	0.0030658	0.0019465	0.00180
30	01 Jan 00, 02:25	0.0030658	0.0019465	0.00180
31	01 Jan 00, 02:30	0.0030658	0.0019465	0.00180

48-hr Design Storm Precipitation Input for Hogchute Dewatering Simulations
100-yr and 10-yr hyetographs from MetPortal (no scaling)
2-yr hyetograph is the 10-yr hyetograph multiplied by scale factor of 0.73 (to produce a peak inflow to Hogchute consistent with USGS regression equation result)

Incremental precipitation (inches)

Ordinate	Date / Time	CA... 0.01_... (100-yr)	0.1_...C_... (10-yr)	...PER6 RU... 48... (2-yr)
1	31 Dec 99, 24:00	0.0	0.0	
2	01 Jan 00, 01:00	0.0300878	0.0204342	0.01492
3	01 Jan 00, 02:00	0.0305893	0.0207747	0.01517
4	01 Jan 00, 03:00	0.0310907	0.0211153	0.01541
5	01 Jan 00, 04:00	0.0315922	0.0214559	0.01566
6	01 Jan 00, 05:00	0.0320936	0.0217964	0.01591
7	01 Jan 00, 06:00	0.0325951	0.0221370	0.01616
8	01 Jan 00, 07:00	0.0330966	0.0224776	0.01641
9	01 Jan 00, 08:00	0.0335980	0.0228181	0.01666
10	01 Jan 00, 09:00	0.0340995	0.0231587	0.01691
11	01 Jan 00, 10:00	0.0346010	0.0234993	0.01715
12	01 Jan 00, 11:00	0.0356039	0.0241804	0.01765
13	01 Jan 00, 12:00	0.0366068	0.0248616	0.01815
14	01 Jan 00, 13:00	0.0376097	0.0255427	0.01865
15	01 Jan 00, 14:00	0.0421229	0.0286078	0.02088
16	01 Jan 00, 15:00	0.0456332	0.0309918	0.02262
17	01 Jan 00, 16:00	0.0501463	0.0340569	0.02486
18	01 Jan 00, 17:00	0.0546595	0.0371220	0.02710
19	01 Jan 00, 18:00	0.0601756	0.0408683	0.02983
20	01 Jan 00, 19:00	0.0661931	0.0449551	0.03282
21	01 Jan 00, 20:00	0.0742166	0.0504042	0.03680
22	01 Jan 00, 21:00	0.0827414	0.0561939	0.04102
23	01 Jan 00, 22:00	0.0902634	0.0613025	0.04475
24	01 Jan 00, 23:00	0.0987883	0.0670921	0.04898
25	01 Jan 00, 24:00	0.10781	0.0732224	0.05345
26	02 Jan 00, 01:00	0.11584	0.0786715	0.05743
27	02 Jan 00, 02:00	0.12587	0.0854829	0.06240
28	02 Jan 00, 03:00	0.13690	0.0929754	0.06787
29	02 Jan 00, 04:00	0.14843	0.10081	0.07359
30	02 Jan 00, 05:00	0.16498	0.11205	0.08179
31	02 Jan 00, 06:00	0.18554	0.12601	0.09199
32	02 Jan 00, 07:00	0.22566	0.15326	0.11188
33	02 Jan 00, 08:00	0.47639	0.32354	0.23618
34	02 Jan 00, 09:00	0.28583	0.19412	0.14171
35	02 Jan 00, 10:00	0.24572	0.16688	0.12182
36	02 Jan 00, 11:00	0.20560	0.13963	0.10193
37	02 Jan 00, 12:00	0.18053	0.12260	0.08950
38	02 Jan 00, 13:00	0.15997	0.10864	0.07931
39	02 Jan 00, 14:00	0.15395	0.10455	0.07632
40	02 Jan 00, 15:00	0.14242	0.0967217	0.07061
41	02 Jan 00, 16:00	0.13138	0.0892291	0.06514
42	02 Jan 00, 17:00	0.12085	0.0820772	0.05992
43	02 Jan 00, 18:00	0.11082	0.0752658	0.05494
44	02 Jan 00, 19:00	0.10180	0.0691355	0.05047
45	02 Jan 00, 20:00	0.0932722	0.0633459	0.04624
46	02 Jan 00, 21:00	0.0847473	0.0575562	0.04202
47	02 Jan 00, 22:00	0.0772253	0.0524477	0.03829
48	02 Jan 00, 23:00	0.0707063	0.0480203	0.03505
49	02 Jan 00, 24:00	0.0641873	0.0435929	0.03182
50	03 Jan 00, 01:00	0.0561639	0.0381438	0.02784
51	03 Jan 00, 02:00	0.0476390	0.0323541	0.02362
52	03 Jan 00, 03:00	0.0416215	0.0282672	0.02064
53	03 Jan 00, 04:00	0.0376097	0.0255427	0.01865
54	03 Jan 00, 05:00	0.0346010	0.0234993	0.01715
55	03 Jan 00, 06:00	0.0330966	0.0224776	0.01641

Suggested citation:
Capesius, J.P., and Stephens, V.C., 2009, Regional regression equations for estimation of natural streamflow statistics in Colorado: U.S. Geological Survey Scientific Investigations Report 2009–5136, 46 p.

Peak Streamflow Equations for Northwest Hydrologic Region

Generalized least-squares (GLS) regression, 90 stations

Approximate range of predictor variables

A: 1–5,250 square miles; A_{7500}: 0–100 percent, and P: 8–49 inches

$Q_2 = 10^{-1.15} A^{0.75} A_{7500}^{-0.41} P^{2.15}$	√	$SEP = 113$,	pseudo$R^2 = 72$,	$SME = 108$,
$Q_5 = 10^{-0.49} A^{0.76} A_{7500}^{-0.54} P^{2.03}$	√	$SEP = 88$,	pseudo$R^2 = 75$,	$SME = 84$,
$Q_{10} = 10^{-0.10} A^{0.75} A_{7500}^{-0.63} P^{1.93}$	√	$SEP = 79$,	pseudo$R^2 = 77$,	$SME = 75$,
$Q_{25} = 10^{0.35} A^{0.75} A_{7500}^{-0.72} P^{1.82}$	√	$SEP = 74$,	pseudo$R^2 = 79$,	$SME = 70$,
$Q_{50} = 10^{0.65} A^{0.74} A_{7500}^{-0.77} P^{1.74}$	√	$SEP = 74$,	pseudo$R^2 = 79$,	$SME = 69$,
$Q_{100} = 10^{0.93} A^{0.74} A_{7500}^{-0.81} P^{1.65}$	√	$SEP = 75$,	pseudo$R^2 = 79$,	$SME = 70$,
$Q_{200} = 10^{1.20} A^{0.73} A_{7500}^{-0.85} P^{1.56}$		$SEP = 76$,	pseudo$R^2 = 79$,	$SME = 70$, and
$Q_{500} = 10^{1.53} A^{0.72} A_{7500}^{-0.88} P^{1.44}$		$SEP = 79$,	pseudo$R^2 = 79$,	$SME = 72$.

Minimum Streamflow for Northwest Hydrologic Region

Generalized least-squares (GLS) regression, 52, 47, and 42 stations

Kannah Creek above Carson Dam

A	11.4	sq mi	(drainage area)
A_{7500}	100	%	(percentage of A above 7,500 ft)
P	38.17	in	(mean annual precipitation)

	Peak (cfs)
Q_2	167
Q_{10}	306
Q_{25}	381
Q_{50}	441
Q_{100}	503

Note: StreamStats uses the Mountain Hydrologic Region Equations for Hogchute. However, the valid slopes for the Mountain Region equations are listed as 7.6 to 60.2 percent. With an average basin slope of 5 percent, Hogchute lies below this range. Thus, we used the Northwest Hydrologic Region Equations for Hogchute. This is a reasonable approach given that Hogchute Dam is located in proximity to the border separating the two hydrologic regions.

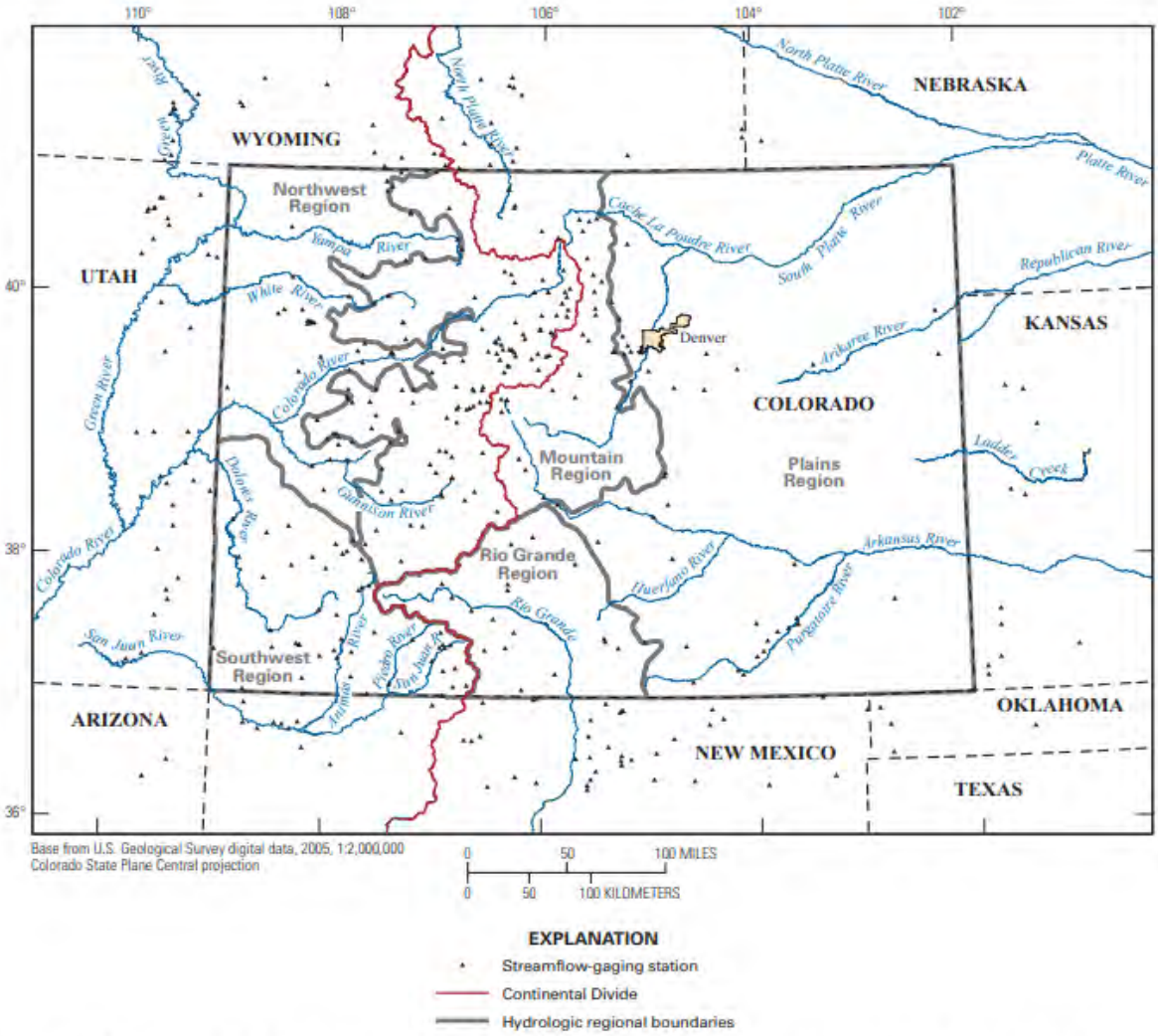


Figure 1. Boundaries of hydrologic regions and locations of streamflow gaging stations in Colorado and adjacent States.

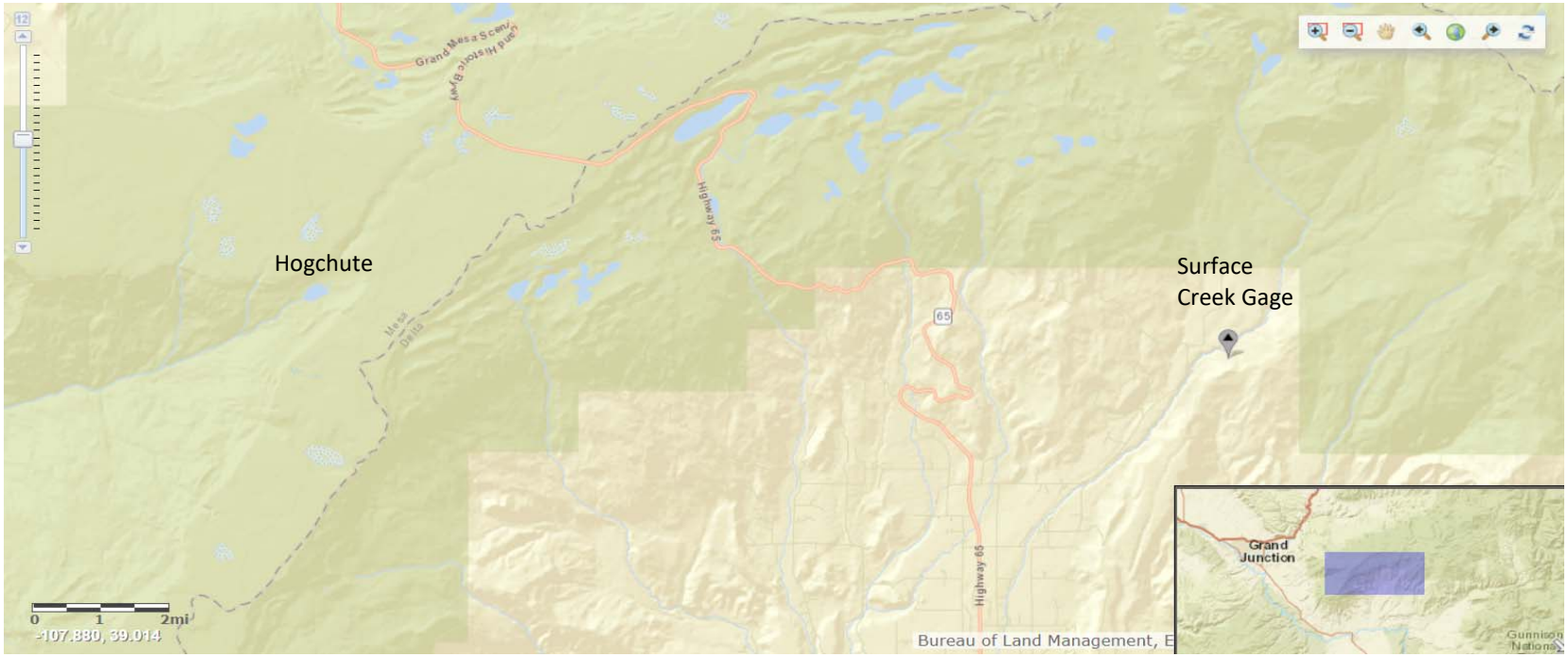
Day of month	USGS 09143000 Surface Creek near Cedaredge, CO (27.4 Square Mile Drainage Area) Mean of daily mean values for each day for water years of record in ft^3/s (Calculation Period 1938-10-01 to 2020-09-30)											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	5	4.9	5.6	13	72	145	96	55	45	24	12	5.6
2	5	4.9	5.6	13	75	150	93	55	45	23	11	5.6
3	5.1	5	5.6	13	82	150	90	55	44	23	10	5.6
4	5.1	5	5.6	13	90	151	84	54	44	21	9.8	5.5
5	5.1	5	5.7	14	93	152	81	56	43	21	9.6	5.5
6	5.1	5.1	5.7	15	97	153	81	55	44	21	9.4	5.7
7	5	5	5.8	16	99	154	81	55	42	20	9	5.6
8	5	5.1	5.9	18	102	153	84	53	39	19	8.3	5.5
9	5	5	6	19	102	148	83	53	38	19	8.2	5.4
10	5	5	6.1	20	105	144	81	52	38	18	8	5.3
11	5.1	5	6.1	22	110	143	80	53	37	18	7.4	5.4
12	5.1	5	6.2	23	115	142	78	53	36	17	7.1	5.5
13	5	5.1	6.7	25	117	142	76	54	34	19	7.2	5.4
14	5	5.1	6.7	27	118	143	73	53	33	18	6.9	5.4
15	5	5.2	6.9	29	121	145	71	51	33	17	6.9	5.3
16	5	5.1	7	31	126	141	69	51	32	16	6.7	5.5
17	5	5.1	7.2	35	129	135	68	52	32	16	6.6	5.2
18	5	5.1	7.3	39	128	129	66	51	32	15	6.6	5.2
19	5	5.1	7.4	41	131	124	65	52	32	16	6.6	5.3
20	5	5.1	7	44	137	120	65	52	30	16	6.6	5.3
21	5	5.1	7.1	48	145	119	64	51	29	17	6.7	5.2
22	4.9	5.1	7.3	51	147	117	62	51	27	17	6.4	5.2
23	4.9	5.1	7.5	54	147	114	62	49	28	16	6.4	5.2
24	5	5.1	7.7	57	146	113	60	48	26	16	6.4	5.2
25	5	5.1	8.1	58	146	110	60	49	25	16	6.2	5.2
26	5	5.2	8.4	60	146	107	60	50	25	17	6.1	5.2
27	5	5.1	8.7	62	148	104	59	51	25	17	6.1	5.2
28	4.9	5.2	8.8	64	147	100	59	50	25	17	6	5.2
29	4.9	4.7	9.3	64	145	98	59	50	25	17	5.9	5.3
30	4.9		9.3	65	145	98	57	49	25	16	5.9	5.2
31	4.9		10		143		57	47		14		5.2
AVE	5	5	7	35	121	131	72	52	34	18	8	5

Gage Values		
Average (of daily averages) =	41.2	cfs
Min (of daily averages) =	4.7	cfs
Max (of daily averages) =	154	cfs
Drainage Area Gage	27.4	square miles
Drainage Area Hogchute Dam	11.0	square miles
Average Elevation Gage Watershed	9875	ft (from StreamStats)
Average Elevation Hogchute Dam Watershed	10607	ft (from StreamStats)
Mean Annual Precip. Gage Watershed	38.89	in (from StreamStats)
Mean Annual Precip.Hogchute Dam Watershed	38.17	in (from StreamStats)

Hogchute Dam Transformation		
Average (of daily averages) =	16.5	cfs
Min (of daily averages) =	4.7	cfs
Max (of daily averages) =	154.0	cfs

Jan	2	cfs
Feb	2	cfs
Mar	3	cfs
Apr	14	cfs
May	48	cfs
Jun	52	cfs
Jul	28	cfs
Aug	20	cfs
Sep	13	cfs
Oct	7	cfs
Nov	3	cfs
Dec	2	cfs

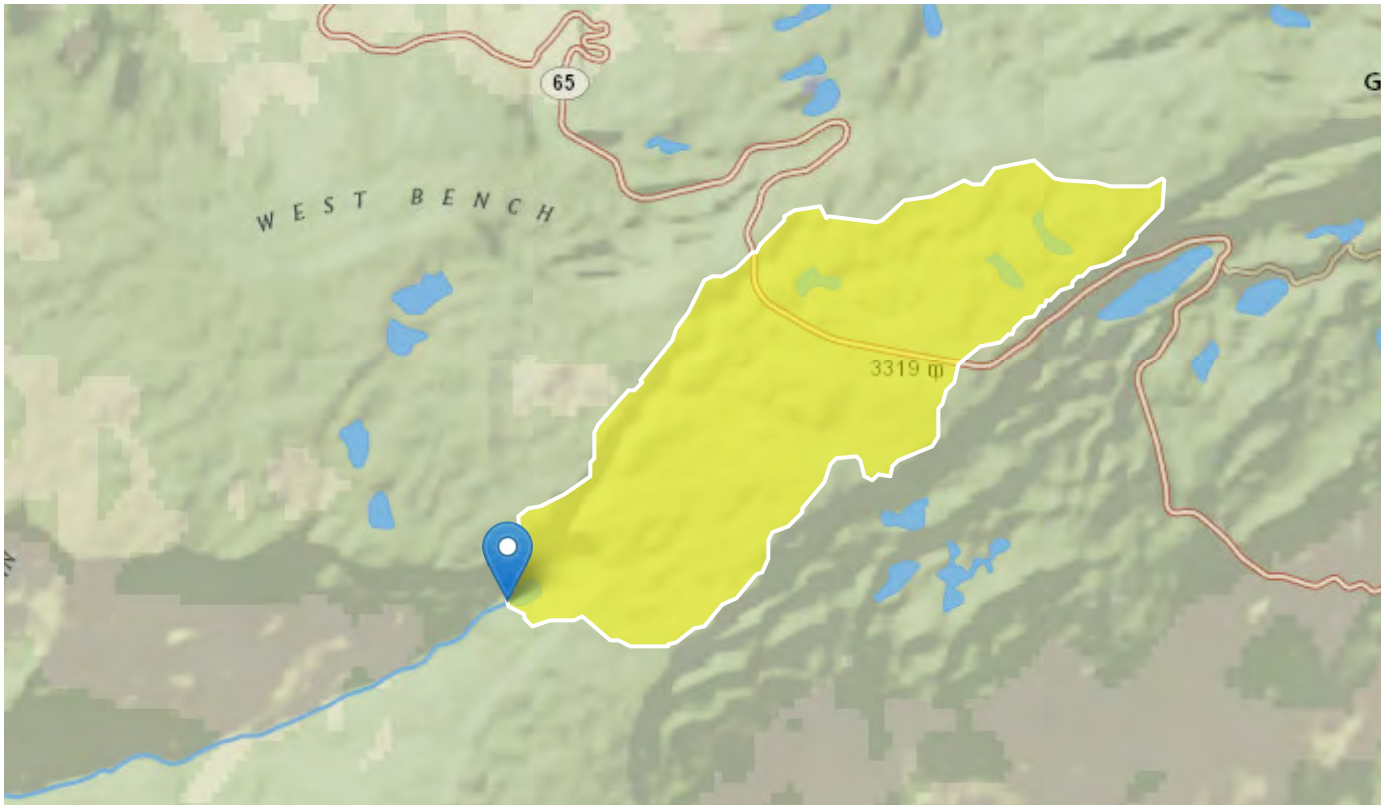
Average monthly flows at Hogchute estimated by adjusting the Surface Creek gage averages by contributing area and average annual precipitation.


Hogchute
Surface
Creek Gage
Grand Mesa Scenic & Historic Byway
Highway 65
Mesa Dells
65
Grand Junction
0 1 2mi
-107.880, 39.014
Bureau of Land Management, E

StreamStats report for Hogchute Dam

StreamStats Report

Region ID: CO
Workspace ID: CO20210202194142489000
Clicked Point (Latitude, Longitude): 38.99541, -108.10974
Time: 2021-02-02 13:42:01 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	11.4	square miles
PRECIP	Mean Annual Precipitation	38.17	inches
BSLDEM10M	Mean basin slope computed from 10 m DEM	5	percent
ELEV	Mean Basin Elevation	10607	feet

Monthly Flow Statistics Parameters[Mountain Region Mean Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	11.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.17	inches	18	47

Monthly Flow Statistics Flow Report[Mountain Region Mean Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
January Mean Flow	3.22	ft^3/s	50
February Mean Flow	2.93	ft^3/s	51
March Mean Flow	3.14	ft^3/s	49
April Mean Flow	7.06	ft^3/s	44
May Mean Flow	46.9	ft^3/s	46
June Mean Flow	95.7	ft^3/s	46
July Mean Flow	41.2	ft^3/s	76
August Mean Flow	15.5	ft^3/s	80
September Mean Flow	8.97	ft^3/s	59
October Mean Flow	7.36	ft^3/s	45
November Mean Flow	5.14	ft^3/s	46
December Mean Flow	3.64	ft^3/s	47

Monthly Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Peak-Flow Statistics Parameters[Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	11.4	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	5	percent	7.6	60.2

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
PRECIP	Mean Annual Precipitation	38.17	inches	18	47

Peak-Flow Statistics Disclaimers[Mountain Region Peak Flow]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors

Peak-Flow Statistics Flow Report[Mountain Region Peak Flow]

Statistic	Value	Unit
50_percent_AEP_flood	164	ft^3/s
20_percent_AEP_flood	220	ft^3/s
10_percent_AEP_flood	261	ft^3/s
4_percent_AEP_flood	286	ft^3/s
2_percent_AEP_flood	330	ft^3/s
1_percent_AEP_flood	368	ft^3/s
0_5_percent_AEP_flood	395	ft^3/s
0_2_percent_AEP_flood	427	ft^3/s

Peak-Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Flood-Volume Statistics Parameters[Mountain Region Max Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	11.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.17	inches	18	47

Flood-Volume Statistics Flow Report[Mountain Region Max Flow]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp

Statistic	Value	Unit	SEp
7 Day 2 Year Maximum	147	ft^3/s	46
7 Day 10 Year Maximum	216	ft^3/s	35
7 Day 50 Year Maximum	276	ft^3/s	31

Flood-Volume Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Low-Flow Statistics Parameters[Mountain Region Min Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	11.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.17	inches	18	47
ELEV	Mean Basin Elevation	10607	feet	8600	12000

Low-Flow Statistics Flow Report[Mountain Region Min Flow]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
7 Day 2 Year Low Flow	1.5	ft^3/s	89
7 Day 10 Year Low Flow	1.13	ft^3/s	153
7 Day 50 Year Low Flow	0.891	ft^3/s	126

Low-Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Flow-Duration Statistics Parameters[Mountain Region Flow Duration]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	11.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.17	inches	18	47

Flow-Duration Statistics Flow Report[Mountain Region Flow Duration]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
10 Percent Duration	64.9	ft^3/s	45
25 Percent Duration	15.5	ft^3/s	55
50 Percent Duration	5.04	ft^3/s	55
75 Percent Duration	2.7	ft^3/s	64
90 Percent Duration	1.65	ft^3/s	85

Flow-Duration Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Annual Flow Statistics Parameters[Mountain Region Mean Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	11.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.17	inches	18	47

Annual Flow Statistics Flow Report[Mountain Region Mean Flow]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
Mean Annual Flow	21.1	ft^3/s	33

Annual Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations

Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

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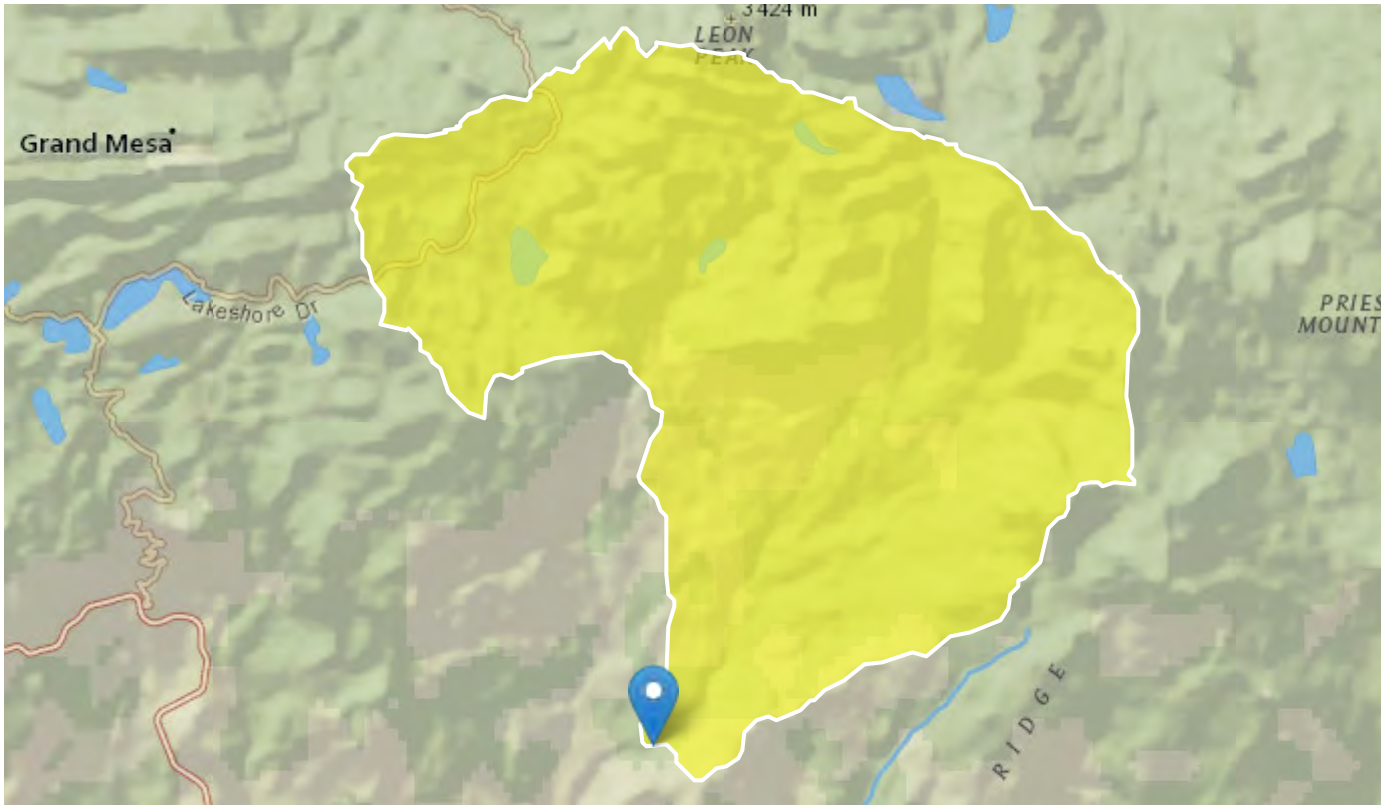
Application Version: 4.4.0

StreamStats report for nearby Surface Creek stream gage

StreamStats Report

Region ID: CO
Workspace ID: CO20210205152914670000
Clicked Point (Latitude, Longitude): 38.98463, -107.85431
Time: 2021-02-05 09:29:32 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	27.4	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	17	percent
PRECIP	Mean Annual Precipitation	38.88	inches
ELEV	Mean Basin Elevation	9875	feet

General Disclaimers

Upstream regulation was checked for this watershed.

Peak-Flow Statistics Parameters[Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	27.4	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	17	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	38.88	inches	18	47

Peak-Flow Statistics Flow Report[Mountain Region Peak Flow]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
50_percent_AEP_flood	416	ft^3/s	49
20_percent_AEP_flood	543	ft^3/s	44
10_percent_AEP_flood	628	ft^3/s	41
4_percent_AEP_flood	691	ft^3/s	40
2_percent_AEP_flood	795	ft^3/s	39
1_percent_AEP_flood	864	ft^3/s	36
0_5_percent_AEP_flood	915	ft^3/s	36
0_2_percent_AEP_flood	NaN	ft^3/s	33

Peak-Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Flood-Volume Statistics Parameters[Mountain Region Max Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	27.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.88	inches	18	47

Flood-Volume Statistics Flow Report[Mountain Region Max Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
7 Day 2 Year Maximum	313	ft^3/s	46
7 Day 10 Year Maximum	470	ft^3/s	35
7 Day 50 Year Maximum	600	ft^3/s	31

Flood-Volume Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Low-Flow Statistics Parameters[Mountain Region Min Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	27.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.88	inches	18	47
ELEV	Mean Basin Elevation	9875	feet	8600	12000

Low-Flow Statistics Flow Report[Mountain Region Min Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
7 Day 2 Year Low Flow	2.73	ft^3/s	89
7 Day 10 Year Low Flow	2.01	ft^3/s	153
7 Day 50 Year Low Flow	1.99	ft^3/s	126

Low-Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Flow-Duration Statistics Parameters[Mountain Region Flow Duration]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	27.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.88	inches	18	47

Flow-Duration Statistics Flow Report[Mountain Region Flow Duration]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
10 Percent Duration	147	ft^3/s	45
25 Percent Duration	37.4	ft^3/s	55
50 Percent Duration	12.2	ft^3/s	55
75 Percent Duration	6.73	ft^3/s	64
90 Percent Duration	4.46	ft^3/s	85

Flow-Duration Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Annual Flow Statistics Parameters[Mountain Region Mean Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	27.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.88	inches	18	47

Annual Flow Statistics Flow Report[Mountain Region Mean Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
Mean Annual Flow	47.2	ft^3/s	33

Annual Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Monthly Flow Statistics Parameters[Mountain Region Mean Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	27.4	square miles	1	1060
PRECIP	Mean Annual Precipitation	38.88	inches	18	47

Monthly Flow Statistics Flow Report[Mountain Region Mean Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
January Mean Flow	8.11	ft^3/s	50
February Mean Flow	7.37	ft^3/s	51
March Mean Flow	8.16	ft^3/s	49
April Mean Flow	19.8	ft^3/s	44
May Mean Flow	110	ft^3/s	46
June Mean Flow	202	ft^3/s	46
July Mean Flow	89.7	ft^3/s	76
August Mean Flow	33.8	ft^3/s	80
September Mean Flow	20.8	ft^3/s	59
October Mean Flow	17.9	ft^3/s	45
November Mean Flow	12.6	ft^3/s	46
December Mean Flow	9	ft^3/s	47

Monthly Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

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