Hale Reservoir Dam Design Report

Cross Creek Metropolitan District

Final Dam Design Report

Hale Dam and Reservoir

Fountain, Colorado

March 2014

AG File No. 12-130

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CERTIFICATION

I hereby affirm that this Design Report was prepared under my responsible charge, for the owners thereof, and to my knowledge is accurate and adheres to the applicable standards and rules provided by the State of Colorado, Department of Natural Resources, Division of Water Resources, Office of the State Engineer.

Craig Michael Ullmann
Registered Professional Engineer
State of Colorado P.E. No.: 38551

TABLE OF CONTENTS

LIST OF FIGURES

APPENDICES

INTRODUCTION

This report summarizes the final design for the Hale Reservoir Dam renovation. The proposed dam and reservoir are located in the Southeast ¼ of the Southeast ¼ of Section 29, Township 15 South, Range 65 West (Figure 2). The reservoir is located within the Cross Creek Park owned by the Metro District, which is approximately two miles northeast of Fountain, Colorado.

The final design was prepared by Applegate Group, Inc. (AG) for Cross Creek Metropolitan District in accordance with Work Order No. 2012-1, approved in October 2012. AG submitted the *Hazard Classification Report* in February 2013, and received Dam Safety concurrence with the minor low hazard dam classification in April 2013. AG submitted the *Hydrology Report* to Dam Safety in May 2013. The emergency spillway for Hale Reservoir Dam was increased from 30 feet to 45 feet to address Dam Safety comments on the May 2013 Hydrology Report, and a revised Hydrology Report is provided with this submittal.

The existing minor low hazard Hale Reservoir Dam will be completely removed, and a new embankment dam will be constructed. The jurisdictional height of the dam will be 15.2 feet. The design includes a principal standpipe spillway at an elevation of 5,622 feet that will be open at all times in order to maintain the normal high water level of 5,622 feet. The emergency spillway will be at an elevation of 5,627 feet, and the crest of the dam will be at 5,630 feet.

Other than flood control, the proposed dam and associated reservoir will provide non-potable water supply for park irrigation, and also provide recreational benefits including fishing, bird watching, and hiking. Water used for irrigation purposes will be removed from the reservoir using a pump station that will be constructed at a future date. The gated outlet in the dam will only be used for emergency drawdown purposes.

Applegate Group has worked with Colorado State Engineer's Office Dam Safety Division throughout the design process. AG submitted a 60% design package to Dam Safety in May 2013, and met with Dam Safety in July 2013 to go over the design. Dam Safety comments were addressed between 60% and final design completed for this design report. This final design report also includes a set of construction specifications, construction drawings, and supporting Reports previously prepared. An opinion of probable cost is also included.

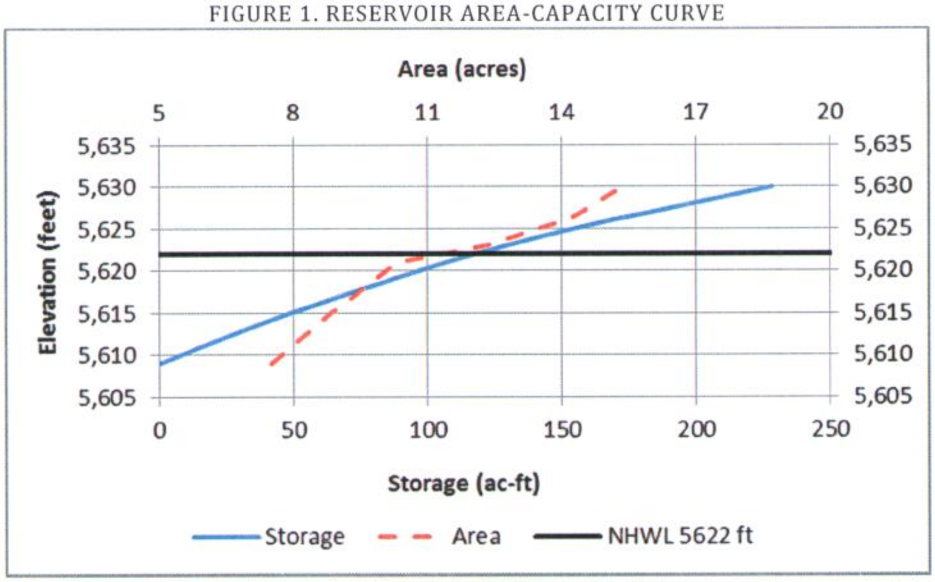
PROJECT COMPONENTS

This report includes the final design for the Hale Reservoir dam replacement project. Primary design component considerations are as follows:

a. The replaced dam will be classified as a minor low hazard dam, and the design was based on the associated *Rules and Regulations for Dam Safety and Dam Construction* (January 2007) for low hazard dams.
b. The existing dam embankment will be completely removed and replaced. The cross section of the dam includes an estimated bedrock surface, a keyway tie-into competent bedrock, a 25-foot crest width, and 4H:1V side slopes with riprap on the upstream slope.

c. The primary spillway is an open standpipe that will maintain the normal high water level at an elevation of 5,622. The standpipe outlet tower wyes into the main outlet pipe downstream of the inclined slide gate.
d. A 45-foot emergency spillway will include a concrete weir with a riprap rundown channel.
e. The new dam and spillway configuration will route the 100-year storm with residual freeboard of 1.36 feet, and the 50-year inflow design flood (IDF) with a residual freeboard of 1.89 feet.
f. The outlet works includes a 2-foot diameter pipe with an inclined slide gate. The outlet will be used infrequently primarily for maintenance purposes, and is capable of drawing down the reservoir 5 feet from the emergency spillway in 23 hours.
g. The new dam will have a reservoir area-capacity curve as shown in Figure 1 and Table 1.

FIGURE 1. RESERVOIR AREA-CAPACITY CURVE



$$x = 0.208\,y^2 + 6.3177\,y + 2.2148$$

Where: y = depth (ft) or (Elevation – 5609)
x = Capacity (ac-ft)

TABLE 1. RESERVOIR CAPACITY TABLE

	ELEVATION (ft)	AREA (ac.)	STORAGE (ac-ft)
Dead Storage	5,609	7.5	0.0
	5,610	7.7	7.6
	5,611	8.0	15.5
	5,612	8.2	23.6
	5,613	8.4	31.9
	5,614	8.6	40.4
	5,615	8.9	49.2
	5,616	9.1	58.1
	5,617	9.4	67.4
	5,618	9.6	76.9
	5,618.74	9.8	84.0
Active Storage	5,619	9.8	86.6
	5,620	10.1	96.5
	5,621	10.3	106.7
	5,622	11.4	117.6
Flood Storage	5,623	12.3	129.4
	5,624	12.9	142.1
	5,625	13.5	155.3
	5,626	14.2	169.1
	5,627	14.4	183.4
Free-board	5,628	14.7	198.0
	5,629	15.0	212.9
	5,630	15.4	228.1

FIGURE 2. VICINITY MAP

SITE REQUIREMENTS

Cesare, Inc. completed a supplemental Geotechnical Study for the proposed reconstruction of the Hale Reservoir Dam in April 2013 during the 30% design effort; this report is included in Appendix A. Exploratory borings were conducted to obtain data on subsurface conditions beneath the existing dam and proposed borrow areas. Soil boring depths varied from 29 to 34 feet below grade. Dam embankment material generally consists of 4 to 23 feet of low plasticity clay material, underlain by up to 8 feet of weathered claystone and Pierre Shale bedrock. Dam foundation and existing soil properties were determined from site borings completed by Cesare, Inc. in April 2013: two along the proposed embankment (one of which was co-located along the outlet works), two along the outlet works, and one north of the reservoir where a non-potable system will be installed. In order to extend the bedrock surface, additional geotechnical data were reviewed from previous reports: Entech Engineering, Inc. January 2009 (5 borings) and Kleinfelder September 2002 (7 borings).

Cesare, Inc. installed two piezometers in April 2013: 1 located on top of the existing dam crest and coincident with the proposed outlet pipe, and the second located at the downstream end of the proposed embankment along the outlet pipe. Groundwater elevations during the geotechnical investigation ranged from 5,616 to 5,620 near the proposed embankment. Depth to water was subsequently measured monthly after the geotechnical investigation, with a resulting water level of approximately 5,621 near the proposed embankment measured in August 2013.

The resulting design considerations came from Cesare's report:

1. Embankment height will allow for up to 1 inch of settlement when calculating residual freeboard with the inflow design flood (IDF). The initial estimate for potential settling was 7 inches, but Cesare reduced settlement potential to 1 inch after discussion with Applegate Group regarding the proposed compacted clay keyway tied into bedrock.
2. The main outlet pipe will be encased in concrete to minimize uneven settling that could impact the outlet gradient, pipe joint integrity, and general performance.
3. Concrete mixtures used on site will consider severe levels of water soluble sulfates in site soils. Concrete mix will be Type II/III with fly ash to address soluble sulfates.
4. The embankment construction contractor will be notified of the moisture and density requirements in the bidding process, and constructability issues will also be addressed during the bidding process. The native clay material to be used in the embankment will be processed to within +/- 2% of optimal moisture content and compacted to a minimum of 95% of standard Proctor density according to ASTM D698. The soil will be placed in 12-inch maximum loose lifts. Note that Cesare's April 2013 geotechnical report suggested placing fill material in 6-inch maximum loose lifts within 2 percent optimal moisture content and compacted to at least 98% of standard Proctor density according to ASTM D698. The design specification for 12-inch loose lifts, +/- 2% of optimal moisture content, and 95% density was selected based on constructability of embankment material (i.e., low to high plasticity clay), input from the State Engineer's Office during the design process, and low hydraulic pressure on the dam associated with a low dam height.
5. Desiccation cracking associated with fat and wet clays should be addressed, potentially by mixing in some of the existing embankment sandy material with the native clays available for borrow material.

Additional site requirements that impact construction of Hale Reservoir are:

- Shallow depth to water – depth to water varies from near ground surface to approximately five feet below ground surface. Dewatering of the reservoir and dam embankment work areas will be necessary to facilitate construction activities. Dewatering will be achieved by pumping water out of open excavated areas, but soil dewatering prior to excavation will not be necessary.
- Non-jurisdictional wetlands – wetlands exist in the area around the Hale Reservoir construction site; however, the wetlands were determined to be non-jurisdictional by the Albuquerque District of the Army Corps of Engineers (Appendix B). Although the wetlands are non-jurisdictional, Cross Creek Metropolitan District is voluntarily mitigating the onsite wetlands as part of the Cross Creek Restoration Plan. Wetland mitigation will include salvaging key wetland plants and topsoil in areas to be disturbed, and relocating the wetlands to the fringes of the planned reservoir. Wetland restoration will be overseen by Kiowa Engineering, who will coordinate directly with Applegate Group during construction oversight.

HYDROLOGY & HAZARD CLASSIFICATION

Applegate Group prepared a *Hazard Classification Report* for the proposed Hale Reservoir Dam in February 2013, and received Dam Safety concurrence with the minor low hazard dam classification in April 2013. The Hydrology and Hazard Classification Reports are included in Appendix C.

Applegate Group submitted a *Hydrology Report* to Dam Safety in May 2013. Hydrologic analyses were completed using the Mesa Ridge Master Plan hydrologic model completed by Kiowa Engineering (2013), which is based on the NRCS Curve Number method. Precipitation input to the hydrologic model was based on the National Weather Service NOAA, Atlas #2 Precipitation-Frequency Atlas of the Western United States Volume III-Colorado. The new dam will provide flood control with a spillway capable of attenuating and routing the 100-year storm, in addition to safely passing and routing the 50-year event, as required by the SEO Rules and Regulations. Peak flows and total storm volumes are shown in Table 1, for more information see the Hydrology Report in Appendix C.

TABLE 2. SUMMARY OF HALE RESERVOIR DAM INFLOW HYRDOLOGY

Event	Cumulative 24 Hour Precip (inches)	Storm Volume (acre-feet)	Peak Inflow (cfs)
50-yr IDF	3.9	257	1,241
100-yr	4.4	295	1,540

SPILLWAY & OUTLET WORKS DESIGN

OUTLET WORKS

The outlet will consist of a 2-foot diameter C905 PVC outlet pipe, and a 30-inch diameter inclined slide gate in order to completely cover the slightly oval opening. The outlet slide gate will be used infrequently, and was designed to meet the 5 feet in 5 day drawdown requirement of the SEO. The actual time to achieve 5 feet of drawdown assuming a starting water surface elevation at the spillway elevation of 5,627 feet will be 23 hours (Figure 3). The outlet will operate under inlet control for reservoir levels below 5,626 feet, and under pipe control above 5,626 ft. The maximum pressure on the pipe would be approximately 8 psi, based on a head of approximately 17.5 feet (maximum water surface elevation of 5630 feet and outlet pipe invert elevation of 5612.5 feet). The outlet pipe will easily accommodate the minimal pressurized flow conditions, because the minimum C905 pressure rating is 100 psi. Additionally, the pipe will be encased in concrete. The resulting stage-discharge curve for the outlet works is shown in Figure 4. Pipe friction losses were calculated using Hazen-Williams and the following loss coefficients:

- PVC pipe loss coefficient of 140
- Entrance loss coefficient of 0.5
- Exit loss coefficient of 1.0
- Wye loss coefficient of 0.6
- Orifice coefficient of 0.62
- Trash rack losses of 0.024 feet (based on an equation for trash rack losses from USBR Design of Small Dams, Chapter 10 Equation 11)

The inlet will be flush with the upstream face of the dam, and then connect to the outlet pipe via a 60 degree elbow. The outlet pipe will be encased in 9-inch thick reinforced concrete through the dam. The side slope of the concrete encasement will be 10H:1V in order to enable the contractor to compact the embankment material around the encasement. A filter diaphragm will be installed around the concrete encasement to intercept seepage flows through the dam. The filter diaphragm will be 11 feet in width and 6.75 feet tall. The location and dimension of the diaphragm were determined using FEMA's Technical Manual: Conduits through Embankment Dams (September 2005).

A hinged trash rack will be installed at the inlet with 2-inch diameter steel pipe cross bars spaced 8 inches apart to provide a maximum velocity of 1.0 feet per second. The trash rack will be hinged on the top end, allowing it to be flipped up by divers if maintenance is needed.

The inclined slide gate will be operated by a hand crank operator on the dam crest. The gate stem will be encased within a 2-inch diameter carrier pipe filled with food grade oil. The carrier pipe will be encased in a concrete beam that runs up the face of the dam, with 1.5 feet below grade and 1.5 feet above grade. The concrete encasement will also carry a 4-inch diameter air vent pipe from the 60 degree bend in the inlet up to the crest of the dam.

The outlet pipe will be extended from the toe of the dam approximately 240 feet to a discharge point downstream of the existing gas line. The purpose of the extended outlet pipe is to convey discharge over the existing gas line downstream of the dam, protecting the gas line and preventing the potential need to lower the gas line if an open channel was constructed over the gas line. The outlet pipe will not be encased in concrete for the 240 foot extension, but will be placed in a trench with compacted fill to finish the trench and cover the pipe.

The outlet pipe will discharge into a Type VI USBR impact basin sized according to the maximum outlet plus standpipe spillway flows of 86 cfs exiting the pipe. Hand railings will be installed for safety and a small grate on the end of the pipe will limit pedestrian access to the outlet pipe. Twelve inch thick 6-inch median diameter riprap with 6 inches of Type II bedding will be placed in the channel that leads to the culvert under C&S Road to prevent erosion. The 6-inch median diameter riprap was based on the Army Corps of Engineer's EM 1110-2-1601 (Hydraulic Design of Flood Control Channels).

FIGURE 3. HALE RESERVOIR DRAWDOWN CURVE

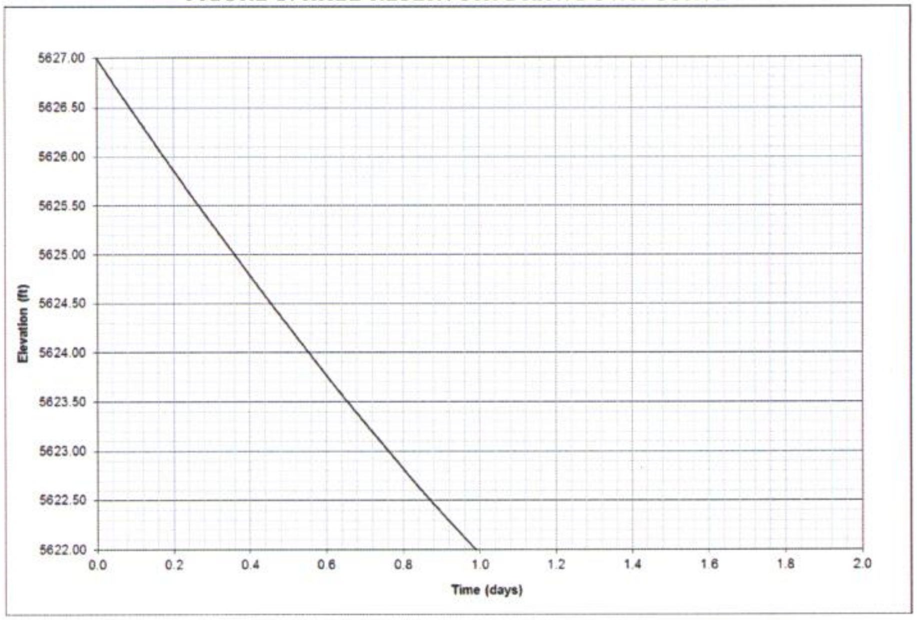
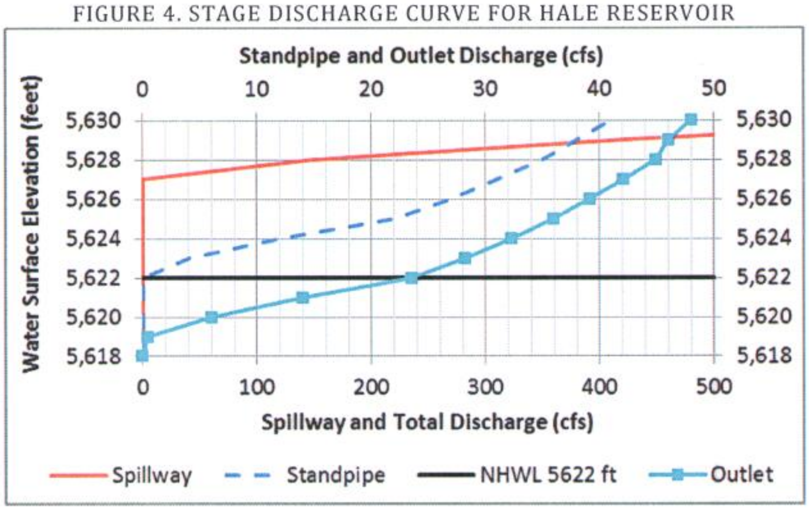


FIGURE 4. STAGE DISCHARGE CURVE FOR HALE RESERVOIR



SPILLWAY DESIGN

PRIMARY SPILLWAY

The primary spillway is an uncontrolled 2-foot diameter standpipe that will maintain the normal high water level at an elevation of 5,622. With a dam crest at an elevation of 5,630 feet, there will be 8 feet of freeboard above the primary spillway. The standpipe outlet tower will wye into the main outlet pipe downstream of the outlet inlet and 60 degree elbow to the outlet pipe. A 4-foot diameter trash rack will be mounted to a 12-inch thick and 5-foot square concrete pad on the upstream face of the dam. The trash rack will have #5 rebar with 4-inch maximum spacing, with a maximum flow through velocity of 1.4 feet per second. Weir flow is the controlling flow regime for reservoir water surface elevation between 5,622 and 5,623 feet, and orifice flow was the controlling flow regime from elevation 5,623 to 5,630. The stage-discharge curve for the standpipe primary spillway is shown in Figure 4.

Emergency Spillway

The emergency spillway design was evaluated using the 50-year IDF from the Mesa Ridge Master Plan hydrologic model developed by Kiowa Engineering in 2013. The spillway crest will be set at an elevation of 5,627 feet, and will be located on the right abutment. The dam crest elevation will be 5,630, resulting in 3 feet of freeboard above the emergency spillway.

The concrete crest of the emergency spillway controls the discharge passing through the spillway. The spillway discharge coefficient was set at 3.3 based on the USBR Design of Small Dams (Section 9.12, page 369), which indicates a discharge coefficient of 3.3 to be appropriate for weirs with height (1 feet high weir above riprap for the proposed spillway) at least one-fifth the head over the weir (3 feet maximum for the proposed Hale Reservoir dam).

The emergency spillway stage discharge table and curve is shown in Figure 4 and Table 3. Total flow from the emergency spillway, standpipe spillway, and outlet for the 50-year and 100-year flows will be 256 cfs and 402 cfs, respectively.

TABLE 3. EMERGENCY SPILLWAY STAGE DISCHARGE

Reservoir Stage (ft)	Spillway Discharge (cfs)
5630	772
5629	420
5628	149
5627	0

The 50-year IDF and 100-year storm were modeled with the Mesa Ridge Master Plan Hydrologic Model developed by Kiowa Engineering in 2013. The primary standpipe spillway was conservatively assumed to be clogged, resulting in all discharge occurring through the emergency spillway. The spillway configuration results in approximately 1.89 feet of residual freeboard during the 50-year IDF and 1.36 feet of residual freeboard during the 100-year event (Table 4).

TABLE 4. SUMMARY OF HALE RESERVOIR DAM HYDROLOGY

Event	Peak Inflow (cfs)	Peak Discharge (cfs)	Residual Freeboard (ft)
50-yr IDF	1,241	178	1.89
100-yr	1,540	322	1.36

Emergency Spillway Channel

Water passing over the emergency spillway crest will enter a riprap lined outlet channel that curves around from the right abutment at the dam crest approximately 500 feet to the outlet channel that conveys flow underneath C&S Road through an existing 12' x 6' box culvert. The spillway channel has a 3-foot high berm on either side. The height of the berm was based on superelevation calculations around the two bends in the spillway channel. Superelevation calculations were based on Equation 4.34 referenced in US Federal Highway Administration (FHWA) Hydraulic Design Series (HDS) No. 4, Introduction to Highway Hydraulics. Berm height was determined as normal depth for the 100-year event discharge through the spillway plus one-half of the superelevation height plus six (6) inches of freeboard. Berm freeboard height was also verified with the equation for spillway channel freeboard from the Bureau of Reclamation's Design of Small Dams.

A one-dimensional HEC-RAS hydraulic model was developed to determine the hydraulics of the spillway channel for the 50-yr, 100-yr, and IDF flows. Riprap was sized based on the Federal Highway Administration's Hydraulic Engineering Circular No. 23, *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition, Volume 2,* Design Guideline 4.

Mannings roughness coefficient was determined for the spillway channel based on the Rice et al. (1998) equation for steep slope riprap revetment.

$$n = 0.0292(D_{50}S)^{0.147}$$

where D_{50} = *median stone diameter (mm)*
S = slope of bed *(ft/ft)*

The resulting riprap sizing and Mannings roughness coefficients are summarized in Table 5.

TABLE 5. RIPRAP SIZING AND MANNING'S ROUGHNESS VALUES

Station	D50 (in)	Riprap Thickness (in)	S (ft/ft)	n
0+52 to 1+50	18	36	0.0367	0.0442
1+50 to 3+80	6	12	0.0367	0.0376
3+80 to 4+50	6	12	0.0336	0.0371
4+50 to 7+20	12	24	0.0336	0.0411

The channel will operate in supercritical flow conditions with a maximum Froude number *(Fr)* of 1.26 at the 50-year IDF and 1.32 at the 100-year flow. Maximum flow velocity was modeled to be 5.7 feet per second (fps) for the 50-year IDF and 7.1 fps for the 100-year flow. Because the emergency spillway will operate under supercritical conditions, discharge for the emergency spillway was calculated using the weir flow equation without consideration of backwater effects. A hydraulic jump will occur at Station 7+47. The Froude number will change from a 1.09 to 0.69 which will initiate a hydraulic jump that will likely consist of standing waves and minor turbulence. A 24-inch thick layer of grouted riprap with 12-inch median riprap diameter will be installed from Station 7+20 to the concrete apron of the upstream end of the concrete box culvert at Station 7+64. The grouted riprap will provide an additional erosion countermeasure at the location of the hydraulic jump.

A concrete cutoff wall will be installed downstream of the existing gas line. The cutoff wall will provide additional countermeasure against potential headcutting of the emergency spillway channel.

STRUCTURAL ANALYSIS

Concrete rebar reinforcement was designed based on the US Army Corps of Engineers Strength Design for Reinforced-Concrete Hydraulic Structures, EM 1110-2-2104. The area of steel reinforcement for temperature and shrinkage stresses is equal or greater than the Army Corps' minimum of 0.0028 times the gross cross-sectional area, half in each face, with a maximum area equivalent to No. 9 bars at 12 inches in each face. This analysis was completed for the following concrete components:

- Outlet pipe concrete encasement
- Inlet concrete encasement
- Spillway control wall
- Staff gage concrete caissons
- Slide gate control concrete house on the dam crest
- Concrete Type VI Impact Basin

The reinforcement for the Type VI Impact Basin conforms to standard designs prepared by the USBR and depicted in Design of Small Canal Structures.

DAM EMBANKMENT

The embankment includes removal of the existing embankment, excavation of a keyway trench and fill placement using a portion of the existing embankment material and the primary borrow area of the excavated reservoir site. There will be approximately 285,000 cubic yards of excavated material available to be used in the new embankment that will require approximately 33,000 cubic yards of fill material.

KEYWAY

The embankment will be tied into bedrock with a keyway approximately 10 feet wide that penetrates a minimum of 2 feet into competent shale bedrock. As noted on the construction drawings, the embankment is founded on Pierre Shale. It is critical to not expose the bedrock foundations for any more than one work shift. All exposed foundation will be covered with a minimum of two loose lifts of fill prior to ending a shift to prevent air slaking of the bedrock. The engineer will determine if excessive time has passed on the exposed foundation or if it has been protected adequately with cover. Compaction control, moisture control, lift thickness, and adequate bonding between lifts are all part of performance criteria used to evaluate the placement. Testing shall be done on each lift and construction observation activities should be conducted under the supervision of a Professional Engineer. The keyway will be backfilled with native clay material processed to within +/- 2% of optimal moisture content and compacted to a minimum of 95% of Standard Proctor density according to ASTM D698. The soil will be placed in 12-inch maximum loose lifts.

EMBANKMENT

The dam crest will be constructed with a crest width of 25 feet at an elevation of 5,630 feet. This will provide for 8 feet of freeboard above the normal high water line. The upstream and downstream slopes will be constructed to 4H:1V side slopes to meet stability and seepage design requirements. This will also allow for simplified placement of riprap slope protection on the upstream slope and mowing operations on the downstream slope. Fill placement for the embankment will be performed in the same manner as the keyway backfill.

The reservoir capacity curve (Figure 1) was based on survey information collected by Clark Land Surveying, Inc. (Clark) under contract with Applegate Group. Clark completed a bathymetric survey of the existing reservoir, and also completed a general topographic survey of the Cross Creek Park in the vicinity of the proposed reservoir. An as-built survey will be required to establish the final reservoir capacity.

WAVE RUN-UP AND SLOPE PROTECTION

Riprap and bedding is included along the upstream face of the dam to protect against wave erosion. The longest fetch across the reservoir would result from a wind blowing out of the north, based on climatic wind data for Colorado Springs obtained from the National Climatic Data Center, National Oceanic and Atmospheric Administration (NOAA). Wind data from the Colorado Springs station was used to determine appropriate wind speeds that would be likely to occur. Monthly peak gust wind velocities, represented by 5-second winds, were reviewed for the period of record available. A summary of this data is shown in the table below; additional data is included in Appendix D.

TABLE 6. SUMMARY OF WIND DATA

Station	Period of Record	5-Second Max Velocity (mph) (mph)	Mean Wind Velocity (mph) (mph)	Prevailing Wind Direction (compass point)
Colorado Springs	1930-1996	70	10	North

A peak 5-second wind speed of 70 mph was selected for design. The wave run-up was analyzed according to USBR ACER TM No. 2. According to this analysis the minimum freeboard should be 1.6 feet and the significant wave height is 1.3 feet. The required riprap size was calculated according to USBR Design Standards No. 13 Embankment Dams, Chapter 7 Riprap Slope Protection. This analysis results in a required riprap size of 4.6 inches. Although the calculations indicate a 9-inch thick layer of 5-inch D_{50} rock will suffice, the design includes a larger rock size to accommodate for and prohibit various urban behaviors. The design includes an 18-inch thick layer of 9-inch D_{50} rock placed atop a 9-inch thick layer of Type II bedding from elevation 5609 to 5622 feet. To prevent vandalism of the riprap, 12-inch diameter D_{50} rock riprap will be used from elevation 5622 to 5630 feet. The 12-inch diameter riprap will be placed in a 24-inch thick layer, and will be placed atop a 12-inch thick layer of Type II bedding.

SEEPAGE DESIGN

A toe drain will be installed at the toe of the dam, and will discharge into the wingwall of the impact basin at an elevation above the normal water level in the downstream channel. The drain system was designed using a minimum diameter of 6 inches for maintenance purposes. The drain pipe used in construction will be a dual wall HDPE with AASHTO Class 2 perforation pattern. This pattern consists of small slits located between the exterior corrugations. The pipe will be surrounded by a two stage filter. An ASTM C33 Fine Sand Aggregate is filter compatible with the embankment material (Appendix F) but too fine to be placed near the pipe. Therefore, an envelope of ASTM No. 67 aggregate will surround the pipe and meets the filter compatibility limits with the fine sand and the slotted drain pipe. Two cleanouts will be installed, one on the midpoint of the southern toe drain, and the second on the midpoint of the northern toe drain. The cleanouts will be 6-inch diameter schedule 40 steel pipe painted black, and will stick up 3 feet above ground surface. No measuring devices have been incorporated into the toe drain design. Flow measurements for the two drains that terminate at the impact basin walls can be measured using a bucket underneath the protruding pipes.

DRAWINGS AND SPECIFICATIONS

A complete set of drawings and construction specifications are included with this submittal under separate covers. The complete drawings are intended to provide a comprehensive documentation of the design including construction details regarding all major and minor project components. Final construction details regarding the outlet and spillway channels and instrumentation have been included in this final design.

DAM INSTRUMENTATION PLAN

A staff gage will be installed on the upstream face of the dam in two sections. The upper portion will extend from the 5,622 to 5,630 feet and will be directly mounted to the concrete beam for the gate stem and air vent. The lower staff gage will extend from the minimum reservoir elevation of 5,609 feet to 5,623 feet. Both gages will display the reservoir elevation in increments of 0.1 ft and numeric labels every foot with the zero point coinciding with the outlet gate invert.

RIVER DIVERSION DURING CONSTRUCTION

The two drainages that drain to Hale Reservoir will be bermed and diverted into temporary bypass channels flowing into the existing stormwater pond just west of Hale Reservoir. The southern portion of the existing berm for the stormwater ponds will be removed in order to facilitate construction of the right abutment of the new Hale Dam. A new berm will be constructed upstream of the removed berm, and a temporary 12-inch diameter culvert will be installed at the downstream end of the new berm to discharge minor inflows downstream of Hale Reservoir dam. The new berm will be constructed upstream far enough to allow excavation of the keyway. An overflow ditch will be cut in the east berm of the stormwater pond to allow flows in excess of the stormwater storage and culvert capacity to be stored in Hale Reservoir below the proposed outlet works elevation in the over excavation area. The temporary bypass channel will be capable of passing the 5-year storm event while utilizing proposed Hale Reservoir storage below the proposed outlet works. The 5-year storm event was modeled with the Mesa Ridge Master Plan Hydrologic Model, and the maximum water surface elevation in Hale Reservoir was determined to be 5615.03 feet during the 5-year storm event (approximately 3.7 feet below the invert of the outlet works).

After the dam foundation and keyway are complete and the outlet works are constructed, the stormwater pond and associated bypass channels will be removed. At that point, the right abutment of the dam can be constructed and the new outlet works will be used to pass flows up to the 5-year event.

OPINION OF PROBABLE COST

Per Rule 6.1.2.1 and Rule 5.6, an opinion of probable cost for completion of the renovation at Hale Reservoir Dam is included with this Report. This estimate is based on the final complete drawings and construction specifications dated October 2013. The opinion of probable cost is provided in Table 7.

The cost opinion includes an estimated construction schedule based mainly on summer and fall construction 2014. The estimated costs reflect the best available construction cost data at the time of the report preparation and are in no way binding or indicative of actual construction costs, which will be bid by the selected contractor.

TABLE 7. OPINION OF PROBABLE COST

Applegate Group, Inc.

Hale Reservoir

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Job No.:	12-130
By:	SAS
Date:	3/26/2014
Client:	Cross Creek Metro District

Item	Item Description	Units	Quantity	Unit Cost	Total Cost
	Administration				
1a	Mobilization	LS	1	$ 85,000	$ 85,000
1b	Bonds and Permits	LS	1	$ 10,000	$ 10,000
	Site Preparation				
2a	Dewatering and Control	LS	1	$ 85,000	$ 85,000
2b	Removal of Existing Dam and Appurtenances	CY	7,119	$ 5	$ 35,595
2c	Clearing and Grubbing	AC	2	$ 5,000	$ 10,000
2d	Erosion and Sediment Control	LS	1	$ 10,000	$ 10,000
2e	Construction Bypass Channels	CY	2,000	$ 5	$ 9,700
2f	Furnish and Place Temporary 30" CMP Culvert	LF	310	$ 25	$ 7,800
	Earthwork				
3a	Stripping and Stockpiling Topsoil	CY	1,500	$ 4	$ 6,000
3b	Excavation & Processing	CY	282,769	$ 5	$ 1,413,845
3c	Dam Placement	CY	56,828	$ 5	$ 284,140
3d	Foundation Preparation	CY	3,000	$ 9	$ 27,000
3e	Furnish and Place 6" D50 Riprap	TN	1,700	$ 35	$ 59,500
3f	Furnish and Place 9" D50 Riprap	TN	4,120	$ 35	$ 144,200
3g	Furnish and Place 12" D50 Riprap	TN	6,490	$ 35	$ 227,150
3h	Furnish and Place 18" D50 Riprap	TN	1,240	$ 35	$ 43,407
3i	Furnish and Place Concrete Grout for Grouted Riprap	CY	41	$ 200	$ 8,100
3j	Furnish and Place Type II Granular Bedding	TN	7,570	$ 25	$ 189,250
	Dam Structures and Outlet Works				
4a	Furnish and Place 24" C905 Encased Outlet Conduit Pipe	LF	123	$ 620	$ 76,260
4b	Furnish and Place 24" C905 Encased Standpipe Overflow	LF	15	$ 620	$ 9,300
4c	Furnish and Place 24" C905 Outlet Conduit Pipe	LF	260	$ 100	$ 26,000
4d	Furnish and Place Structural Concrete with Rebar	CY	35	$ 650	$ 22,750
4e	Furnish and Place Baffled Outlet	CY	15	$ 1,000	$ 15,000
4f	Furnish and Place Fittings for Standpipe	EA	2	$ 2,000	$ 4,000
4g	Furnish and Place Outlet Intake Fitting	LS	1	$ 2,000	$ 2,000
4h	Furnish and Place Overflow Standpipe Trashrack	LS	1	$ 1,500	$ 1,500
4i	Furnish and Place Inlet Trashrack	LS	1	$ 2,500	$ 2,500
4j	Furnish and Place Baffle Structure Grate	LS	1	$ 1,500	$ 1,500
4k	Furnish and Place Outlet Slide Gate, 30-inch Diameter	LS	1	$ 7,500	$ 7,500
4l	Furnish and Place Staff Gages	LS	1	$ 5,000	$ 5,000
4m	Furnish and Place Filter Diaphragm	CY	10	$ 100	$ 1,000
4n	Furnish and Place Toe Drain	LF	700	$ 35	$ 24,500
4o	Furnish and Place Precast Toe Drain Manhole	LS	1	$ 2,000	$ 2,000
4p	Furnish and Place 8" C900 Toe Drain Outlet Pipe	LF	260	$ 30	$ 7,800
	Site Reclamation				
5a	Seeding	AC	3	$ 1,000	$ 3,000
5b	Haul off old debris 3 mi to Fountain Landfill	LS	1	$ 5,000	$ 5,000
5c	Place topsoil	CY	1500	$ 4	$ 6,000
	Optional Bid Items				
1	Traffic Control	LS	1	$ 2,000	$ 2,000
				Sub Total	**$ 2,880,297**
	Contingency				
1	Design and Contractor Contingency	%		0%	$ -
2	Construction Contingency	%		10%	$ 288,029.70
		Engineering and Construction Base Bid			**$ 3,168,327**
	Stripping and Stockpiling Wetland Topsoil	CY	4,800	$ 4	$ 19,200
	Wetlands Topsoil Placement	CY	4,800	$ 4	$ 19,200
	6" Aluminum Gated Irrigation Pipe (Wetland Flow Spreader)	LF	300	$ 20	$ 6,000
	Invasive Plant Control	LS	1	$ 3,000	$ 3,000
	Harvesting and Planting	LS	1	$ 2,500	$ 2,500
	Construction Observation	%		5%	$ 159,000
	Testing	%		5%	$ 159,000
	Survey	%		2%	$ 64,000
				Grand Total	**$ 3,600,227**