FINAL REPORT

BEAVER PARK DAM
DAM ID: 200102
ALTERNATIVES REPORT

RIO GRANDE COUNTY, COLORADO
WATER DIVISION 3
WATER DISTRICT 10

FEASIBILITY STUDY APPROVAL
Pursuant to Colorado Revised Statutes 37-60-121 &122, and in accordance with policies adopted by the Board, the CWCB staff has determined this Feasibility Study meets all applicable requirements for approval.

Signed

9/18/2012
Date

Prepared for

Colorado Department of Natural Resources,
Division of Parks and Wildlife
Denver, CO 80216

March 5, 2012

URS

URS Corporation
8181 East Tufts Avenue
Denver, CO 80237

Project No. 22242295

COLORADO REGISTERED PROFESSIONAL ENGINEER
GREGORY GENE GUNZ
37737
7MAR12

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Beaver Park Dam is a large, high hazard dam located near South fork, Colorado. The dam is owned and operated by the Colorado Parks and Wildlife (CPW), located near South Fork, Colorado. In the spring of 2010, a sinkhole was observed on the downstream left abutment of the dam, near a location of prior seepage concerns. An assessment of Beaver Park Dam began in 2010, when the Colorado State Engineer's Office (SEO) observed the sinkhole in the downstream left abutment rock and moraine area and imposed a restriction on the reservoir.

A risk analysis performed in 2010 (URS) identified potential failure modes (PFMs) that were judged to be significant contributors to risks under normal operating conditions, including PFMs that involved internal erosion through the moraine to the left of a rock knob in the left abutment. The largest contributor to risk was judged to be piping through the left abutment moraine, exiting through a bedrock window located immediately downstream of the left abutment of the dam, designated as PFM 1.

CPW contracted URS Corporation (URS) to evaluate alternatives that met the following objectives:

a) Reduce risks associated with seepage and piping through the left abutment under static (normal operating) conditions (PFM 1),

b) accommodate a 3-foot increase in the normal maximum pool level,

c) address primary spillway concerns identified by the SEO,

d) improve the reliability of the outlet works operation, and

e) removal of the current SEO reservoir restriction.

The alternatives developed included consideration of rehabilitation of the existing dam and construction of a new replacement dam downstream of the existing dam. In the development of the dam rehabilitation alternatives different rehabilitation alternative elements (or design components) were required to achieve the different objectives. In fact, for a complete rehabilitation alternative that addresses all five objectives, five different elements are required. The different rehabilitation alternative elements (elements) were developed separately, in part so that CPW could decide whether it wanted to consider rehabilitation to address only some of the elements, for example, rehabilitation that did not include the 3-foot increase in normal maximum pool or the improved reliability of the outlet works. Ultimately, CPW decided in a design review workshop, held on September 19, 2011, that the rehabilitation alternative should include all five elements to achieve all five objectives.

With the understanding that the reservoir is currently under an SEO imposed restriction that limits the reservoir to a maximum elevation of 20 feet below the primary spillway crest for "unresolved concerns with the 2010 sinkhole and spillway channel erosion." Given identified concerns and the subsequent magnitude of the imposed restriction, a "do nothing" alternative is not recommended. The CPW would have to consult with the SEO on operating the reservoir at the restricted level indefinitely. The risks associated with the dam were identified in the Risk Analysis Report completed in 2010.

For reduction of left abutment seepage and piping risks, Alternative 1A was judged to be preferred over Alternative 1B, principally because the opinion of probable construction cost for Alternative 1B was estimated to be 3 times higher than that for Alternative 1A.

To address all of the desired project objectives given above, other elements of rehabilitation must be added to Alternative 1A. The resulting total alternative would include the following five elements:

- Element 1A - Left Abutment Downstream Filter and Drain System (Alternative 1A)
- Element 2 - Outlet Works Rehabilitation and Gate Reconfiguration
- Element 3 - Primary Spillway Concrete Chute
- Element 4A - Crest Modifications - HDPE Liner
- Element 5 - 1914 Landslide Filter and Drain System

This configuration was then compared to the new RCC dam alternative (Alternative 2), which would also address all desired project objectives.

When this was done, Alternative 1A was found to be the preferred alternative; because Alternative 2 has an opinion of probable construction cost about 3 times that of Alternative 1A. In addition, we believe that the cost of the Alternative 2 would likely be even higher than that used in the comparison, because of unaccounted for construction variables that would be required to address subsurface conditions in the new left abutment, which at this time are not well defined.

The elements proposed in Alternative 1A were judged to provide a reduction in the annual probability of failure of PFM 1 from ***1.5×10^{-2} to 6.0×10^{-5}.*** Flood surcharge storage and the potential 3-foot raise in the maximum water surface elevation were considered in estimating this risk reduction.

The opinion of probable cost for Alternative 1A, separated into the five elements of rehabilitation, is summarized in the following table.

Alternative 1A Opinion of Probable Cost Summary

Element	*Element Description*	*Opinion of Probable Cost*
1A	Left Abutment Downstream Filter and Drain System	$ 6,392,100
2	Outlet Works Rehabilitation and Gate Reconfiguration	$ 3,892,000
3	Primary Spillway Concrete Chute	$ 1,704,300
4A	Crest Modifications - HDPE Liner	$ 1,237,600
5	1914 Landslide Filter and Drain System	$ 725,000
	Alternative 1A Opinion of Probable Cost	***$ 13,951,000***

Alternative 1A will require SEO approval prior to construction. The proposed three-foot raise will require SEO approval in accordance with Rule 6 of the 2007 State of Colorado Rules and Regulations for Dam Safety and Dam Construction. The proposed 3-foot raise of the primary spillway invert will require submittal of a Design Report, which is to include a stability analyses to verify that the dam is stable at the proposed reservoir pool elevation and spillway hydraulic analysis verifying that the spillway can safely pass the Inflow Design Flood (IDF) identified in the Hydrology Report. The above submittals are to be coordinated with the SEO to avoid delays in the construction of the alternative

Based on the results of the evaluations of alternatives, URS recommends the following:

- Continue detailed observations of the dam until rehabilitation is completed, to identify changes that could indicate increasing risks, and take appropriate actions if changed conditions are observed.
- Proceed with the engineering design and construction of the preferred rehabilitation alternative to improve the facility's safety and reliability.
- Construct Elements 1A and 2 concurrently. An additional $1,000,000 would need to be included in the opinion of probable cost for Element 1A to account for dewatering costs and reconstruction of the existing outlet conduit, if it were constructed separately from Element 2. All dewatering costs were included in Element 2 and the dewatering is required for both Elements 1A and 2.
- Obtain all required permits for the construction of all of the elements associated with Alternative 1A. This includes permits from U.S. Army Corps of Engineers (USACE) and U.S. Forest Service (USFS).
- Perform a field investigation of the 1914 landslide area to confirm the dimensions of the filter drain system proposed for that area.
- Perform geotechnical investigations as necessary to confirm assumptions related to Alternative 1A.
- Perform an underwater inspection of the upstream conduit intake structure and conduit to identify dimensions and condition and to identify potential design and construction challenges associated with Element 2.
- Install piezometers in the area between the left abutment crest dike and the 1914 landslide.

1.1 INTRODUCTION

Beaver Park Dam is a large, high hazard dam located near South Fork, Colorado, as shown on Figure 1-1. The dam is owned and operated by CPW. In the spring of 2010, a sinkhole was observed by the Colorado State Engineer's Office (SEO) on the downstream left abutment of the dam, near a location of prior seepage concerns. After discovery of the sinkhole, the SEO imposed a restriction on the reservoir elevation to 20 feet below the spillway crest elevation. Shortly thereafter, the CPW engaged URS to evaluate the potential causes for the sinkhole and develop recommendations for further actions. A key component of the assessment was a risk analysis, which resulted in the following conclusions:

1) The estimated risk of seepage and piping failure under static loading (normal operations) exceeded U.S. Department of the Interior, Bureau of Reclamation (Reclamation) guidelines (Reclamation, 2003) for requiring expedited risk reduction action.
2) The estimated risk of seepage and piping failure under the reservoir restriction imposed by SEO was below Reclamation's guidelines for expedited risk reduction action, but was within the range justifying long-term risk reduction measures.

The risk analysis was limited to static potential failure modes. Potential failure modes related to hydrologic events resulting from elevated reservoir levels or seismic loading conditions were not considered.

Based on the results of the assessment of the existing conditions, CPW engaged URS to complete an alternatives study, the results of which are presented in this report. This report presents alternatives that address the dam safety concerns associated with normal reservoir operations including a 3-foot increase in normal reservoir storage above the current primary spillway crest. Also presented in this report are opinions of probable costs and conceptual construction schedules for the developed alternatives. The overall objective of the proposed alternatives is to improve the safety and reliability of the facility.

1.2 SCOPE

CPW engaged URS Corporation (URS) to develop and evaluate alternatives related to the rehabilitation of the existing dam or the construction of a new dam. The alternatives needed to meet the following objectives:

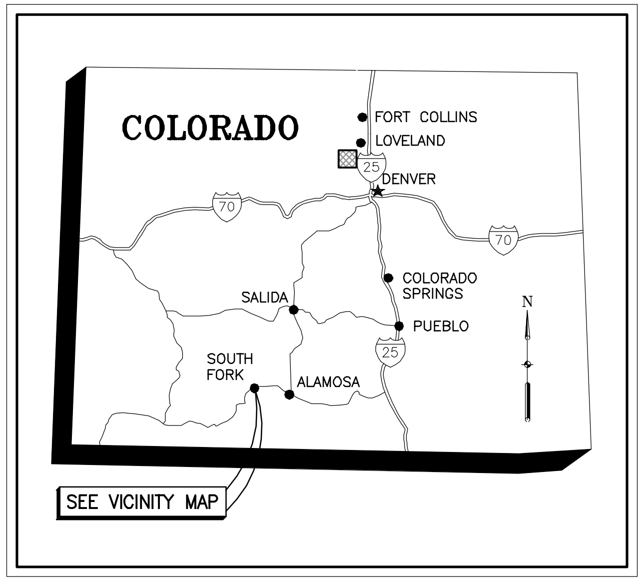
1) Reduce risks associated with seepage and piping through the left abutment under static (normal operating) conditions.
2) Accommodate a 3-foot increase in the normal maximum pool level.
3) Address primary spillway concerns identified by the SEO.
4) Improve the general operations and reliability of the outlet works.
5) Removal of the current SEO reservoir restriction.

1.3 LIMITATIONS

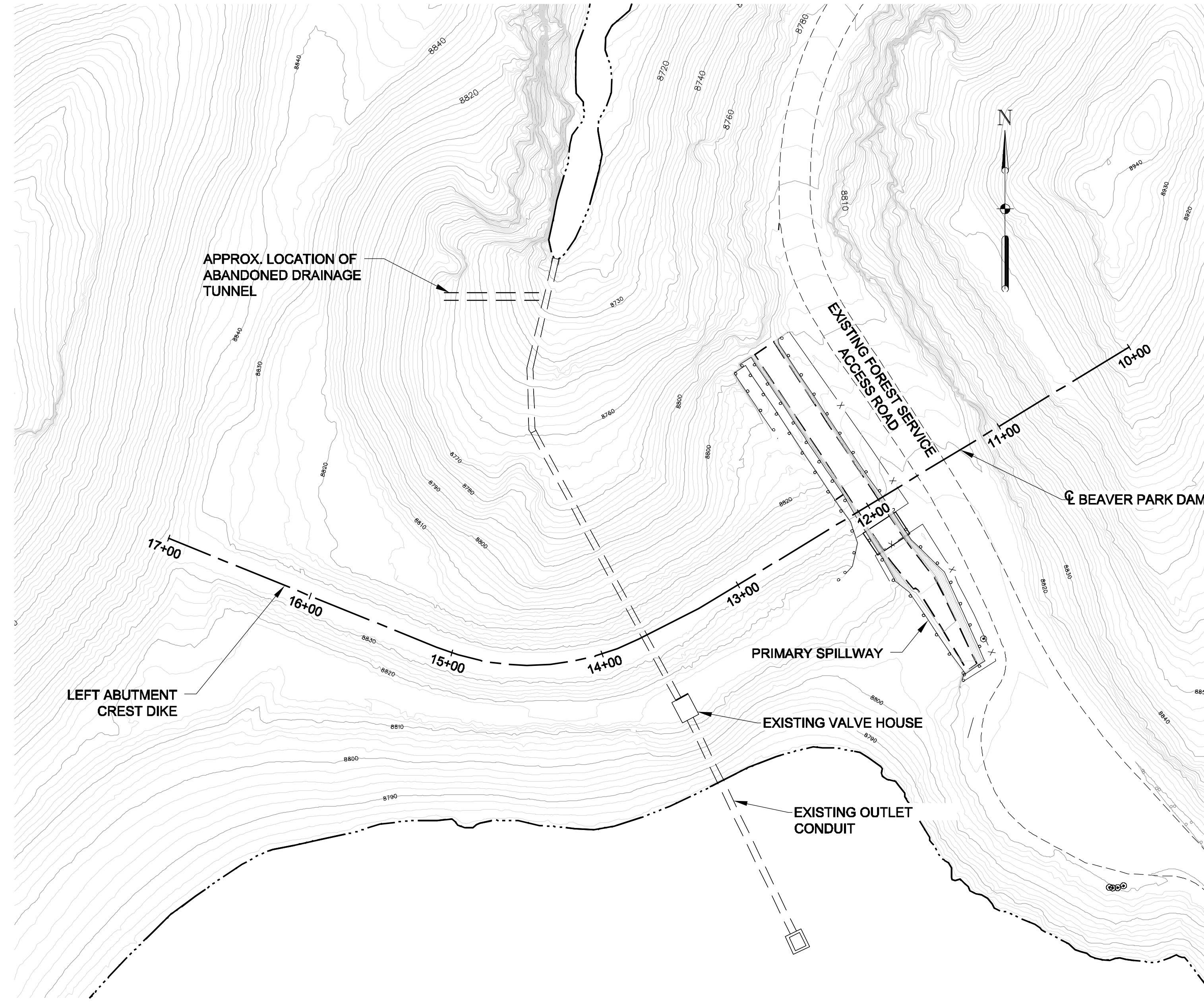
URS represents that its services were performed within the limits prescribed by the client in a manner consistent with the level of care and skill exercised within consistent with the current standard of professional engineering practice of other similar engineering professionals in Colorado. No other representation to the client or the SEO is expressed or implied, and no warranty or guarantee is included or intended. URS does not guarantee the performance of the project in any respect; only that the engineering work and judgments rendered meet the standard of care of the profession.

Professional judgments are presented in this report. These are based partly on evaluation of technical information gathered and partly on our general experience with similar projects.

URS

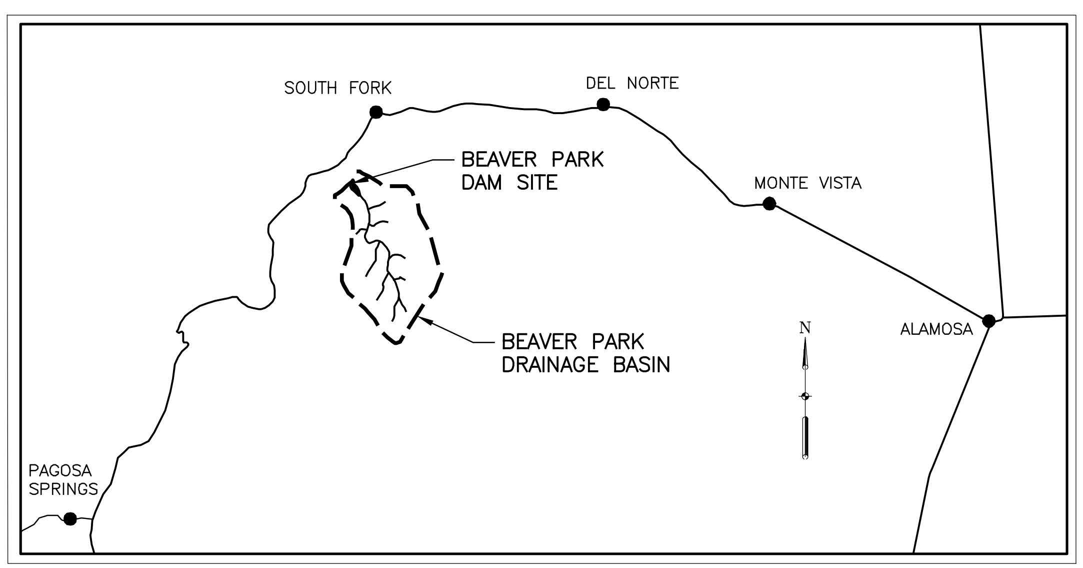


STATE MAP
NOT TO SCALE



LOCATION MAP

0 50 100 200
SCALE IN FEET



VICINITY MAP
NOT TO SCALE

Job No. :	22242295	BEAVER PARK DAM GENERAL PLAN
Prepared By :	PPP	
Date :	2/20/12	

FIG. 1-1

2.1 GENERAL

Beaver Park Dam was originally constructed between 1912 and 1914 as a concrete-faced rockfill structure. The dam was originally 87 feet high. A low level outlet tunnel was constructed along the left abutment, with a gate chamber located at the upstream dam toe. A masonry overflow spillway channel was constructed over the dam crest near the maximum section. Photographs of the pertinent features of Beaver Park Dam are included in Appendix E.

2.2 HISTORY OF MODIFICATIONS

In 1914, during first filling, when the reservoir was about two-thirds full, fine till soil reportedly began to wash out at a location a short distance downstream of the dam on the left abutment. The reservoir was drawn down and cribbing was constructed on the downstream slope adjacent to the left abutment. When the reservoir filling resumed, a major landslide reportedly occurred on the moraine slope some 700 feet north of the dam.

Between 1915 and 1916, grouting and reservoir puddle lining operations were performed. Unfortunately, no design or construction documentation is available to substantiate this activity.

Between 1916 and 1917, a drainage tunnel was excavated at the base of the downstream left abutment to address significant seepage noted during first filling attempts. The exact location and extent of the tunnel is not known, although a general location can be inferred from available information.

Little information is known for the time period of 1917 through 1937.

In 1938, following the reported appearance of a sinkhole above the presumed location of the drainage tunnel constructed between 1916 and 1917, the drainage tunnel was backfilled with hand placed rock.

Design drawings from 1947 show an upstream extension of the outlet tunnel of about 146 feet and construction of a new trash rack structure for the outlet works intake. The extended outlet conduit was backfilled using rockfill material at a slope of 2.5H: 1V (horizontal: vertical) from the newly constructed trash rack structure to the existing upstream slope of the dam. The outlet was also extended to accommodate the placement of fill described below.

Between 1949 and 1951 design drawings document the construction of a 10-ft raise of the dam with placement of selected impervious fill upstream and dumped rockfill downstream. The spillway was removed from the crest of the dam as part of this work. To abandon the existing spillway, a reinforced concrete seepage wall was placed across the old spillway entrance, and dumped rockfill was placed in the chute of the spillway. A new side-channel ogee spillway was constructed at the right abutment. The gate tower and controls were raised 10 feet to match the dam raise, and repairs were made in the outlet tunnel. This work included the placement of compacted fill on the upstream dam face and upstream of the left abutment rock knob and moraine.

Modifications were made to the outlet tunnel and emergency spillway between 1966 and 1969. Repairs included installation of vent tubes and filling of a cavity in the floor of the outlet just downstream of the gates.

A 20-ft long, 3/8-inch thick steel plate was placed in the invert and 2 feet up the sides of the outlet tunnel immediately downstream of the gate. Spillway crest modifications included lowering the crest by approximately 3 feet, where it remains today.

Modification to the outlet tunnel continued between 1970 and 1971. Work included installation of a 48-inch diameter steel liner in the tunnel. The liner was grouted in place using concrete.

Repairs were made to the outlet works shaft in 1976. The work included repairs to the intermediate landings in the shaft and placement of concrete lining in the shaft between landings one and two (counting from the bottom of the shaft).

In 1977, modifications to the downstream portion of the outlet conduit were performed. Modifications included extending the 48-inch diameter steel conduit downstream 133 feet. The extension was the required to accommodate a request by the SEO to place fill over deteriorated cribbing on the left abutment.

Between 1987 and 1988 modifications to raise the dam and excavate an auxiliary spillway to the right of the existing side-channel spillway on the right abutment were proposed to be completed in two phases however, only Phase I was completed. Phase I included, a downstream crest raise of 17.5 feet which extended the crest along the surface of the left abutment area and is referred to as the crest dike. Phase I also included excavation of the right side of the primary spillway, extending approximately 65 feet along the dam centerline. Figure 2-1 provides a cross section of the evolution of the dam cross section. The distance from the centerline of the primary spillway to the right extent of the auxiliary spillway is 80 feet. Additional rockfill was placed over the slope that was formed during the 1977 modifications that covered the cribbing and drainage tunnel portal.

In 2009, repairs were made to the steel lining in the outlet tunnel on the downstream side of the gate chamber. These repairs were made to redesign and replace a collapsed section of the steel liner.

In 2010, the SEO observed a sinkhole in the downstream left abutment area and imposed a 20-foot restriction on the reservoir elevation, as noted earlier in this report.

2.3 RECOGNIZED CONCERNS

The SEO and representatives from CPW performs a dam safety inspection of the dam and appurtenant structures annually. The major concerns observed during the inspections consist of headcutting of the primary spillway, sinkhole development at the left abutment and seepage occurring at the abutment.

The reliability of the outlet works is also a recognized concern, as the results of the risk analysis are dependent on the reliability of the outlet works to safely operate in the event the reservoir would have to be evacuated in during an emergency.

2.4 TOPOGRAPHIC SURVEY

A survey of the dam and reservoir area was completed by URS in June and July 2011. The survey included spot elevations of pertinent features near the dam as well as topographic information for the dam and reservoir area. The topographic information was developed using Lidar information obtained aerially.

The vertical control for the survey was derived from GPS observations, using GEOID09, based on a North American Vertical Datum of 1988 (NAVD88) elevation of 8397.1 feet on an NGS control disk LENICH (Brass Cap set in boulder). No differential levels were performed. The horizontal control for the survey was modified Colorado State Plane South Zone (0503) NAD 83(2007) coordinates. These coordinates are for the exclusive use of the Beaver Park Reservoir, and is considered project control only. Coordinates for the control points on this project were brought to ground using the inverse of the Combined Scale factor (1/0.999534293). To convert project coordinates to State Plane, multiply the Project Northing and Easting values by the Combined Scale Factor (0.999534293), then add 1,000,000.00 to the resultant Northing value, and 2,000,000.00 to the resultant Easting value. All elevations presented in this report are NAVD 88 unless noted otherwise.

From the original construction of Beaver Park Dam to the most recent modification, several datums have been referenced. The elevations shown on the various construction drawings were reviewed and correlated to the NAVD 88 datum used for the new topographic survey. The correlation was made by comparing a survey of the pertinent features in NAVD 88 with the elevations of the various features shown on the historical drawings. Table 2-1 shows a summary of the elevation and datum information as related to the above mentioned periods of time of the various modifications. The correlations shown in Table 2-1 are our best interpretations of the information presented in the historical documents. There are inherent uncertainties in these interpretations and the resulting correlations, and these uncertainties should be considered in using the information presented in the table.

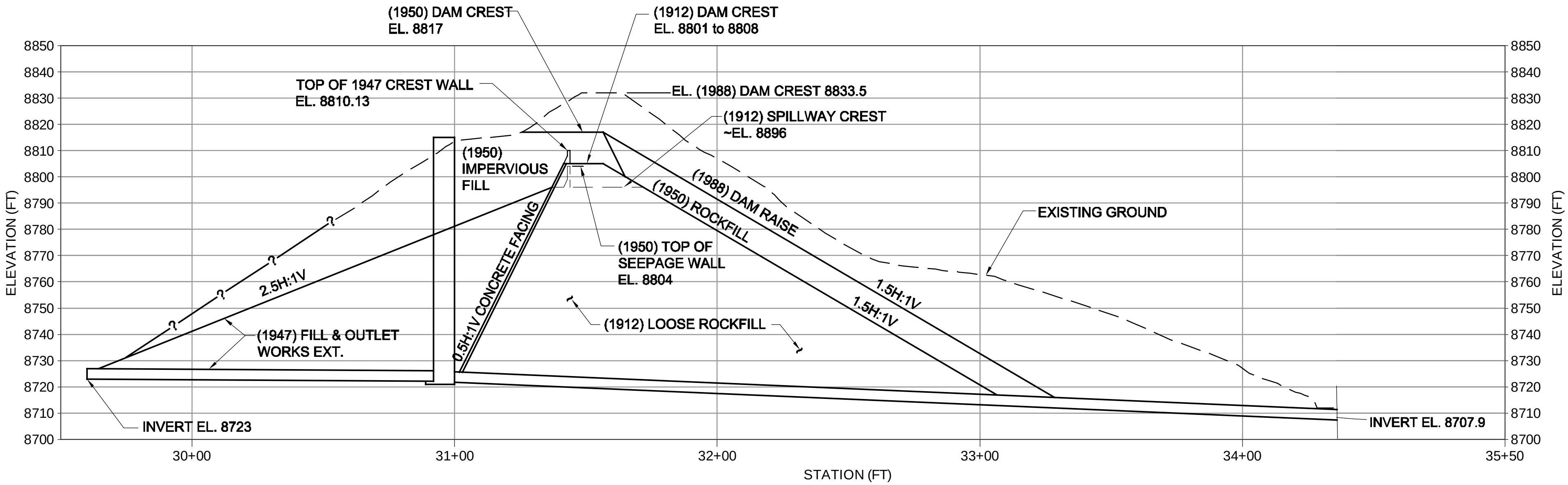
Table 2-1 Summary of Historical Datums and Corresponding NAVD 88 Adjustment

Date	*Drawing Description*	*Referenced Dam Crest Elevation*(1)	*Elevation Adjustment to NAVD 88*(2)	*NAVD 88 Dam Crest Elevation*
1912-1914	Original Construction	190.0	8611 to 8618	8801 to 8808
1947	Upstream Extension of Outlet	83	8722	8805
1949 – 1951	Dam Raise	95	8722	8817
1969(3)	Auxiliary Spillway	83	8722	8805
1970	Discharge Tunnel Repair	8795	22	8817
1976	Outlet Works Shaft Repair	8795	22	8817
1977	Discharge Extension Tube	8795	22	8817
1988	Phase I Dam Raise	8811.5	22	8833.5

(1) Elevations obtained from the drawing set.
(2) Value is added to the referenced dam crest elevation to obtain the crest elevation in NAVD 88.
(3) Elevation appears to have been obtained from documentation prior to the 1950 dam raise and is not representative of the elevation that existed during that time.

A more comprehensive breakdown of the elevations of project features and adjustments to NAVD 88 datum is provided in Appendix A.

URS



*ELEVATIONS SHOWN ARE IN NAVD88.

Job No. : 22242295	EVOLUTION OF THE BEAVER PARK DAM CROSS SECTION
Prepared by : PPP	
Date : 1/13/12	

FIG. 2-1

3.1 GENERAL

A draft probable maximum flood (PMF) hydrology study was completed in July 2011 by CPW. The hydrology study estimated the peak inflow and the maximum reservoir water surface elevation during the PMF event. URS used the data in this hydrology study to estimate the peak inflows and the maximum reservoir water surface elevations for more frequent flood events (e.g., 10-year, 100-year). The parameters presented in the Draft Hydrology Study (CPW, 2011) were generally used with modifications to select parameters applicable to the frequency storm being analyzed.

Because the 2011 Draft Hydrology Study by CPW was completed using the dam and spillway crest information shown on the drawings from the 1988 modifications (Dam Crest equal to 8811.5) discussed in Section 2 of this report, the elevations were adjusted to NAVD 88 (Dam Crest equal to 8833.5) to be consistent with the topographic information used to develop and evaluate the various alternatives.

3.2 HYDROLOGIC MODELING

3.2.1 Watershed Parameters

Table 3-1 summarizes the watershed parameters for Beaver Park Dam. All parameters are from the Draft Hydrology Study (CPW, 2011) except the K_n value and lag time for the frequency storms which were selected by URS for this report.

Table 3-1 Summary of Watershed Parameters

Description	*Parameter*
Watershed Area (mi^2)	47.8
Maximum Elevation (ft, elevation from USGS Quad, msl)	12,751
Minimum Elevation (ft, elevation from USGS Quad, msl)	8,782
Length of Longest Watercourse (mi)	13.1
Distance to basin centroid (mi)	6.6
Basin Slope (ft/mi)	302
Average Weighted Basin Response (K_n) - General Storm PMP(1)	0.225
Average Weighted Basin Response (K_n) - Local Storm PMP(1)	0.065
Average Weighted Basin Response (K_n) - Frequency Storms(1)	0.25
Lag Time (hr) (2) - General Storm PMP	9.94
Lag Time (hr) (2) - Local Storm PMP	2.87
Lag Time (hr) (2) - Frequency Storms	11.1

(1) Selected from Table 7, in the 2008 Hydrologic Basin Response, Phase IIB, Parameter Estimation Guidelines (SEO, 2008)

(2) Lag Time estimated using methodology developed by Reclamation (Cudworth, 1989).

The basin lag characteristics (K_n) and the resulting lag time were adjusted to represent the basin's response to a more frequent storm event (i.e., 10-year, 100-year, etc.).

Basin loss parameters for more frequent storm events were consistent with the 2011 Draft Hydrology Study. The studies used Green and Ampt methodology to estimate rainfall loss parameters for the Beaver Park Dam watershed and these parameters are summarized in Table 3-2.

Table 3-2 Summary of Watershed Loss Parameters

Rainfall Loss Parameter	***Value***
Initial Abstraction	0.2 inches
Hydraulic Conductivity (XKSAT)	0.25 inches/hour
Soil Suction Head (PSIF)	4.9 inches
Soil Moisture Deficit (DTHETA)	0.46
% Impervious (RTIMP)	2%

3.2.2 Precipitation

Frequency storm precipitation values for the Beaver Park Dam area were estimated using isopluvial maps of Colorado obtained from the NOAA Atlas 2, Volume III technical report (NOAA, 1973).

The location and extent of the watershed was estimated on each isopluvial map to determine the 6 and 24 hour precipitation depth values (in inches) for the 100-year return period. Incremental precipitation depth durations for each event were estimated using the procedures documented in the NOAA report. Precipitation depth values for the 500-year and 1,000-year events were linearly extrapolated by plotting the 10-year and the 100-year values on a log normal chart. Incremental precipitation depth durations for the 500-year and 1,000-year events were estimated using the procedures in the NOAA report for the 100-year events.

Table 3-3 summarizes the estimated depth-duration precipitation values for each frequency storm event.

Table 3-3 Frequency Storms Cumulative Precipitation Depth-Duration

Duration	***100-year (in)***	***500-year (in)***	***1,000-year (in)***
5 minute	0.48	0.57	0.62
15 minute	0.95	1.12	1.22
1 hour	1.66	1.97	2.15
2 hour	1.81	2.20	2.42
3 hour	1.91	2.36	2.60
6 hour	2.10	2.66	2.94
12 hour	2.47	3.10	3.45
24 hour	2.85	3.63	3.95

3.2.3 Reservoir Routing

The 2011 Draft Hydrology Study by CPW used HEC-HMS version 3.4 (USACE, 2009) to prepare a hydrologic and reservoir routing model. The storm frequency depth-duration data summarized in Table 3-3 were input into the model, along with dimensionless unit hydrographs developed based on the frequency storm K_n values and corresponding lag time. Table 3-4 provides a summary of the reservoir routing results based on the various storm events analyzed by URS and CPW, with the primary spillway elevation set to account for the desired 3-foot storage increase. The reservoir routing analyses assumed that the outlet works was closed and that the reservoir level was at the spillway crest elevation at the beginning of the flood.

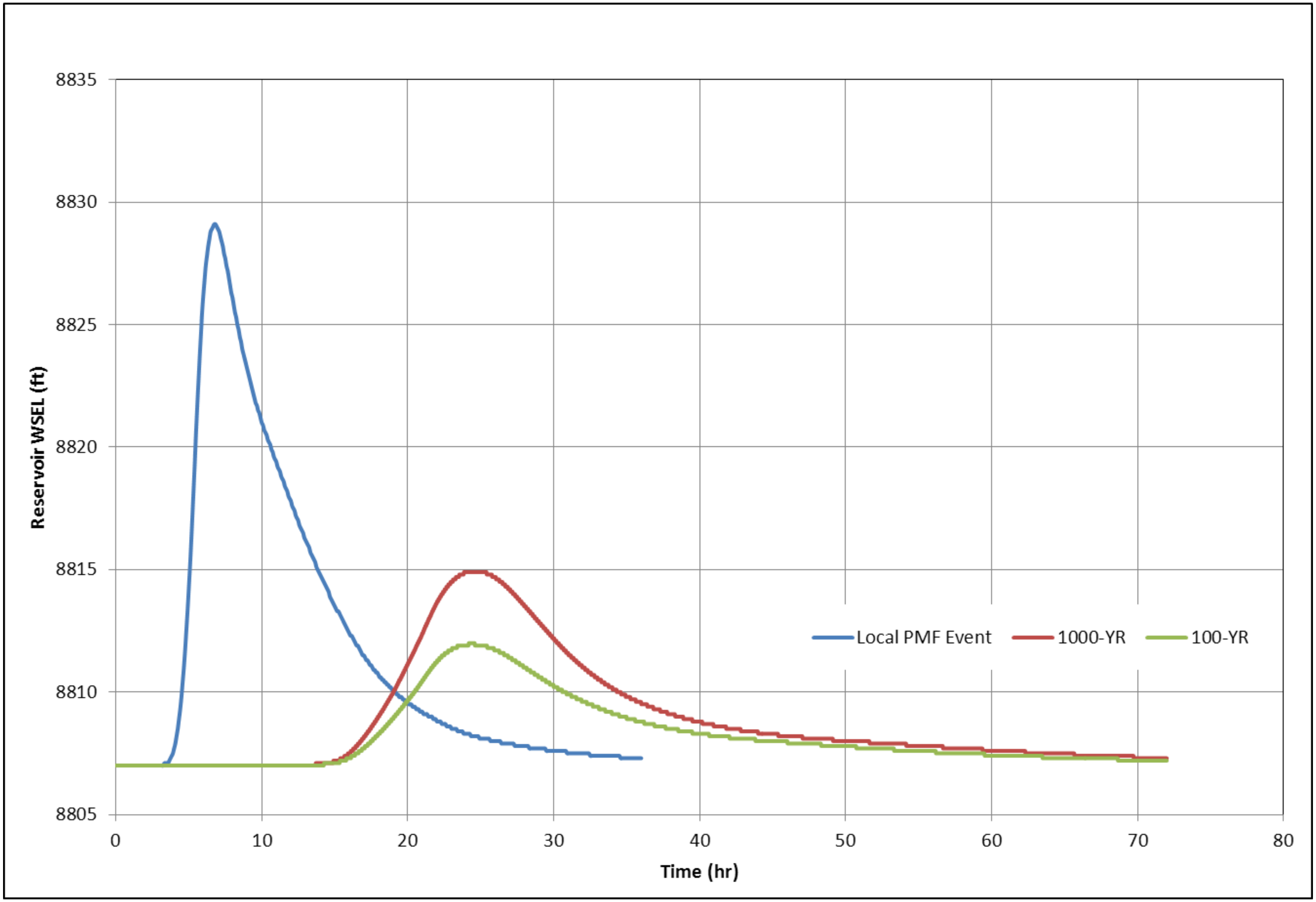
Table 3-4 Summary of Reservoir Routing

	Storm Frequency				
	100-year	***500-year***	***1,000-year***	***Local Storm PMF***	***General Storm PMF***
Duration	24 hour	24 hour	24 hour	6 hour	48 hour
Peak Inflow *(ft^3/s)*	2,904	4,028	4,624	30,323	16,503
Peak Outflow *(ft^3/s)*	2,039	2,748	3,125	16,635	14,904
Max Elevation (ft) (1)	8,812.0	8,813.9	8,814.9	8,829.1	8,827.9
Depth Above Spillway Crest ft)(2)	5.0	6.9	7.9	22.1	20.9
Depth on Access Road (ft)	2.0	3.9	4.9	19.1	17.9
Freeboard (ft)(3)	21.5	19.6	18.6	4.4	5.6

(1)NAVD 88 Datum.
(2) Spillway crest elevation 8807 (Current El 8804 + 3-ft spillway crest raise)
(3)Vertical distance below the dam crest elevation of 8833.5

The results of the routing of the various storm events, it was determined that the dam would not be overtopped during the controlling PMF (local) event. The resulting reservoir water surface versus time relationships for the 100 year, 1,000 year and local PMF events are shown on Figure 3-1.

Figure 3-1 Reservoir Water Surface versus Time for the 100-year, 1,000-year and Local PMF Events



3.3 CONSTRUCTION DIVERSION

Construction of some elements of the rehabilitation alternative requires diversion of Beaver Creek by means of an upstream cofferdam and pumping system. The SEO requires, by Rule 5.10.2 of the 2007 Rules and Regulations for Dam Safety and Dam Construction, the submittal to and acceptance by the SEO of a water diversion plan. The plan details the design storm, drawings and specifications of the diversion features, stability of the features under normal and flood loading conditions, hydraulic capacity of the diversion features, and a plan for removal of the features post construction.

Several sources of hydrologic information were evaluated to develop a range of potential flows for construction diversion.

3.3.1 USGS Regression Equations

USGS regression equations were used as a comparison with resultant HEC-HMS storm frequency peak flows. Based on the Beaver Park Dam watershed location, flood-frequency equations for the Rio Grande region within the state of Colorado were selected (USGS, 2000). Peak flows for the 2, 5, 10, 25, 50, 100, 200, and 500-year storm frequency were calculated.

Other input parameters required to use the regression equations include annual precipitation and watershed area. An average annual precipitation value of 39.66 inches was used based on the rainfall data for the nearby town of South Fork, Colorado. A watershed area of 47.8 mi^2 was used, consistent with the 2011 Draft Hydrology report (CPW).

Table 3-5 presents the resulting peak flows estimated by the regression equations for the Rio Grande region.

Table 3-5 Summary of USGS Rio Grande Regression Peak Flows

Storm Frequency	***Peak Flow (ft^3/s)***
2-year	410
5-year	611
10-year	777
25-year	994
50-year	1,140
100-year	1,390
200-year	1,550
500-year	1,950

The peak flow for the 100-year event determined by the HEC-HMS model was compared with the peak flow calculated by the USGS regression equation for the Rio Grande region estimates the 100-year USGS peak flow at approximately 48 percent of the HEC-HMS 100-year peak flow. The USGS regression equations have a standard error of predication of 52.9 to 89.1 percent for the storm frequency compared.

As a result of the above comparison we concluded that the HEC-HMS model may be overestimating peak flows and maximum water surface elevations and that further analyses may be warranted to better define the frequency flood hydrology for Beaver Park Dam. The following sub-section presents additional evaluations for the frequency floods for construction diversion purposes.

3.3.2 Hydrologic Modeling for Construction Diversion

A preliminary study was performed to estimate the minimum cofferdam heights to completely contain the 2-, 5-, and 10-year storm events without overtopping (zero freeboard). The basin parameters summarized in Section 3.2 above were used with the exception of the selection of the K_n value. A K_n of 0.30 was selected to develop the dimensionless unit hydrograph. The selected K_n value represents the upper bound for 100-year events mountainous areas (Sabol, 2007). The results of the analysis are summarized in Table 3-6.

Table 3-6 Minimum Cofferdam Heights

Storm Frequency	*Peak Flow (ft^3/s)*	*Minimum Height (ft)*	*Storm Volume (af)*
2-year	75	11	85
5-year	391	23	401
10-year	693	31	706

The values shown in Table 3-6 do not include any base flows or additional flows that may result from rainfall occurring on snowpack.

The approximate stage capacity for the 2-, 5-, and 10-year cofferdam heights are summarized in Table 3-7.

Table 3-7 Cofferdam Estimated Stage Capacity Summary (acre-feet)

Elevation	*2-year*	*5-year*	*10-year*
8697	0	0	0
8700	4	4	4
8705	34	34	34
8709	85	85	85
8710		107	107
8715		230	230
8720		381	381
8721		401	401
8725			563
8728			706

3.3.3 Beaver Park Reservoir Daily Estimated Inflows

CPW provided URS with a 28 year (1980 through 2008) dataset of smoothed synthetic daily inflows into Beaver Park Reservoir for the purposes of modeling and managing water rights. The dataset was developed using the flow data from the gage below the reservoir and the stage data from the reservoir to approximate daily inflow into the reservoir. See Appendix B for a detailed summary of the development of the dataset.

To provide additional information for a contractor to consider in the development of storage and pumping diversion system, the dataset provided by CPW was statistically analyzed using a Log Pearson Type III distribution. Table 3-8 summarizes the monthly 50, 75 and 90 nonexceedance flows determined from the analysis and Table 3-9 summarizes the 3, 5, and 7 day 75 and 90 percent nonexceedance flows.

Table 3-8 1980 through 2008 Monthly Estimated Inflow Summary (ft^3/s)

Month	***Probability of Nonexceedence***			***Maximum***
	50	***75***	***90***	
January	13[(1)]	14	15	20
February	13	14	15	20
March	15	18	23	45
April	32	46	67	152
May	96	147	213	470
June	77	128	199	393
July	22	32	47	243
August	18	22	28	110
September	18	23	29	93
October	18	23	29	77
November	15	17	19	27
December	14	15	16	22

(1) The mean daily flow is less than 13 ft^3/s on 50 percent of the days in January

Table 3-9 1980 through 2008 Estimated 3-, 5-, and 7- day 75 and 90 Percent Nonexceedance Inflow Summary (ft^3/s)

Month	*75and 90 Percent Probability of Nonexceedance*					
	3-Day		*5-day*		*7-day*	
	75	*90*	*75*	*90*	*75*	*90*
January	16	16	16	16	16	16
February	16	16	16	16	16	16
March	33	32	31	33	32	31
April	116	107	100	116	107	100
May	327	311	292	327	311	292
June	295	275	259	295	275	259
July	86	78	73	86	78	73
August	44	40	36	44	40	36
September	48	42	39	48	42	39
October	42	39	38	42	39	38
November	23	22	21	23	22	21
December	18	18	17	18	18	17

3.3.4 Conclusions

The results from the numerical analysis performed for the 2-, 5-, and 10-year storm events, the USGS regression equations and the dataset of estimated inflows into Beaver Park Reservoir were compared against one another. It was concluded that the estimated dataset provides an acceptable representation of the inflows that can be expected on a monthly basis. The dataset summaries presented in Tables 3-8 and 3-9 could be used in combination with the peak inflow resulting from the frequency storms presented in Table 3-6 by a contractor to develop a stream diversion plan.

Tables 3-8 and 3-9 reveal that construction activities need to be carefully coordinated during the months of April through July. During these months the estimated inflows increase and may exceed the capacity of the existing outlet works which are estimated to be about 350 ft^3/s. Attempting to divert flows during this period would be challenging and costly, with the chance that the system would be overwhelmed resulting in damage to the structure.

4.1 GEOLOGIC MAPPING

Geologic mapping of the left abutment of the dam site was completed to indicate areas consisting of bedrock, glacial moraine, landslide deposits, and manmade fill. Results of the mapping are shown on Figure 4-1.

The landslide deposit just downstream of the dam on the left abutment was mapped in approximately the same location as shown in a construction inspection report (Wilkinson, E., 1976. Beaver Park Reservoir, C-612 D, W. Div. 3 W. Dist. 20, prepared for Division of Wildlife, November 24). The landslide or slip was on the steep angle of repose slope in glacial materials, and the sinkhole observed in 2010 was about 30 feet behind the top of the slip plane. The cribbing in this area was built in 1916 and 1917 to help stabilize the slide. When the cribbing was moving and showing distress, a small scarp along the top of the slide was mapped. This may be when the cribbing movement was reactivated. The scarp on the side closest to the dam was approximately 5 feet high and probably reflects total movement that occurred. Materials exposed along this 5-foot high scarp consist of glacial materials that appear to be undisturbed. The side of the slide furthest from the dam was only approximately located, because at that side the scarp blends into a talus and colluvial covered slope. The current slide surface was steep, close to the angle of repose of the silty sand, and therefore the surface of the slide, maybe a few feet thick, could slide again, especially if settlement occurs at the toe of the slide. A rock fill berm covered the cribbing and should reduce the likelihood of further slide movement.

The location and geometry of the bedrock suggested that the glacial moraine occupies a paleo-valley in the left abutment of the dam. The moraine may have a thickness of 300 to 400 feet in the paleo-valley and may have caused the present Beaver Creek stream valley to be superimposed on and into bedrock, forming the right abutment of the dam. The presence of the paleo-valley was important to the characterization of the site with respect to piping potential, because it suggested that the left abutment of the dam is founded on a thick sequence of silty sand.

The 300- to 400-foot thickness of glacial materials in the paleo-valley includes glacial materials that were not eroded along the west margin of the valley. On the USGS topography map, the top of the glacial deposits appears to be a flat outwash surface. Examination of this large area in the field suggested that it was actually rolling terrain with eskers, kettles (closed depressions), and an odd drainage pattern. The sandy glacial sediments contain large (5-foot diameter) boulders, suggesting the upper portion of the glacial deposits consists of typical ground moraine. This ground moraine surface was about 200 feet above the crest of the dam. The moraine downstream of the dam was about 150 feet lower than the crest of the dam. The estimated thickness of the glacial deposits filling the paleo-valley would therefore be 200 plus 150 feet, or about 350 feet thick. The bottom of the paleo-valley appeared to be 500 to 800 feet west of the current valley.

4.2 SEISMIC REFRACTION

4.2.1 Introduction

Geophysical techniques were used as part of the geotechnical evaluation for possible improvements at Beaver Park Dam and reservoir. The objectives of the geophysical survey were to:

- Obtain seismic velocity information to help estimate overburden thickness
- Aid in estimation of the elevations of the top of the bedrock surface

This information was sought in selected areas at the site, to help in the alternatives evaluation and in future design efforts. The geophysical method used was seismic refraction. Below is a summary of the results of the geophysical survey. A detailed discussion of the geophysical survey is provided in Appendix C.

4.2.2 Geophysical Results

Four seismic spreads, encompassing two seismic lines (totaling approximately 1,900 linear feet) were surveyed during the field program. The final location of each seismic traverse was chosen based on the survey objectives and the site conditions and constraints.

Seismic refraction data were collected using a signal-enhancing Geometrics Strataview seismograph. The geophone spacing was 20 feet for a spread length of 460 feet. For each seismic source, seismic energy was produced by a small explosive charge. Seismic sources were used at both ends of each spread for forward and reverse travel times and in the middle of the spread to increase near-surface velocity control. Additional offset sources were also used to obtain better coverage of the bedrock surface.

A three-layer model is represents the subsurface along each of the seismic spreads. The near-surface material has a seismic velocity that varies from 870 feet/second (ft/s) to 1,560 ft/s, and has a thickness of 5 feet to about 56 feet. Based on geologic mapping and field observations, seismic layer 1 likely consists of soil and colluvial overburden materials.

Seismic layer 2 has a seismic velocity of 1,570 ft/s to 3,390 ft/s, and a thickness of 10 feet to more than 135 feet. Based on geologic mapping, seismic layer 2 likely correlates to glacial moraine and outwash materials.

Seismic layer 3 has a seismic velocity of 7,560 ft/s to over 10,320 ft/s, and likely consists of variably-weathered bedrock materials. The depth to bedrock along the left abutment varies from about 25 feet to more than 150 feet.

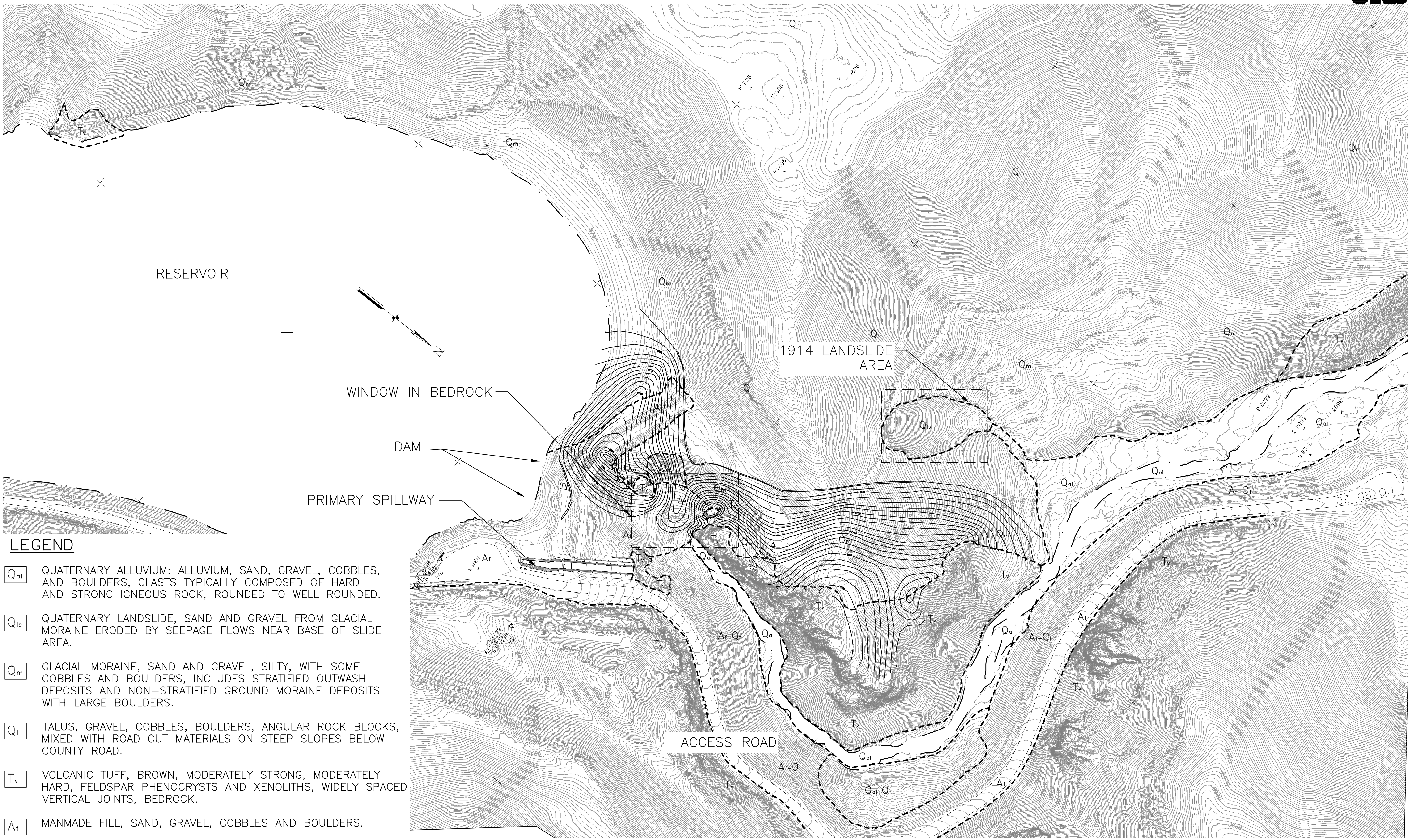
Figure 4-2 shows the estimated bedrock contours along the left abutment of the dam.

This work was conducted in accordance with reasonable and accepted engineering geophysics practice, and the interpretations and conclusions are rendered in a manner consistent with other consultants in our profession. However, all geophysical techniques have some level of uncertainty and limitations. No other representation to the client is expressed or implied, and no warrant or guarantee is included or intended.

4.3 CONCLUSION

The glacial moraine identified by current and previous geologic mapping that occupies the paleo-valley in the left abutment was estimated to have an overall depth to bedrock of 350 feet, which was based on surface observations. The geophysical study estimated the depth above bedrock from about 25 feet to over 150 feet below the reservoir water surface as shown on Figure 4-2.

The results of the geophysical survey confirmed that the paleo-valley walls are relatively steep and the total thickness of the glacial moraine is greater than 350 feet and that the bottom of the paleo-valley is more than 150 feet below the reservoir water surface.



LEGEND

Q_{al}	QUATERNARY ALLUVIUM: ALLUVIUM, SAND, GRAVEL, COBBLES, AND BOULDERS, CLASTS TYPICALLY COMPOSED OF HARD AND STRONG IGNEOUS ROCK, ROUNDED TO WELL ROUNDED.
Q_{ls}	QUATERNARY LANDSLIDE, SAND AND GRAVEL FROM GLACIAL MORAINE ERODED BY SEEPAGE FLOWS NEAR BASE OF SLIDE AREA.
Q_m	GLACIAL MORAINE, SAND AND GRAVEL, SILTY, WITH SOME COBBLES AND BOULDERS, INCLUDES STRATIFIED OUTWASH DEPOSITS AND NON-STRATIFIED GROUND MORAINE DEPOSITS WITH LARGE BOULDERS.
Q_t	TALUS, GRAVEL, COBBLES, BOULDERS, ANGULAR ROCK BLOCKS, MIXED WITH ROAD CUT MATERIALS ON STEEP SLOPES BELOW COUNTY ROAD.
T_v	VOLCANIC TUFF, BROWN, MODERATELY STRONG, MODERATELY HARD, FELDSPAR PHENOCRYSTS AND XENOLITHS, WIDELY SPACED VERTICAL JOINTS, BEDROCK.
A_f	MANMADE FILL, SAND, GRAVEL, COBBLES AND BOULDERS.

— — — GEOLOGIC CONTACT, APPROXIMATE LOCATION

NOTES:

1. TOPOGRAPHIC BASE MAP IS FROM A SURVEY PERFORMED BY URS IN JUNE 2011. THE DATUM IS NAD 83 (COLORADO STATE PLANE, SOUTH ZONE) AND NAVD 88.

0 100 200 400
SCALE IN FEET

URS

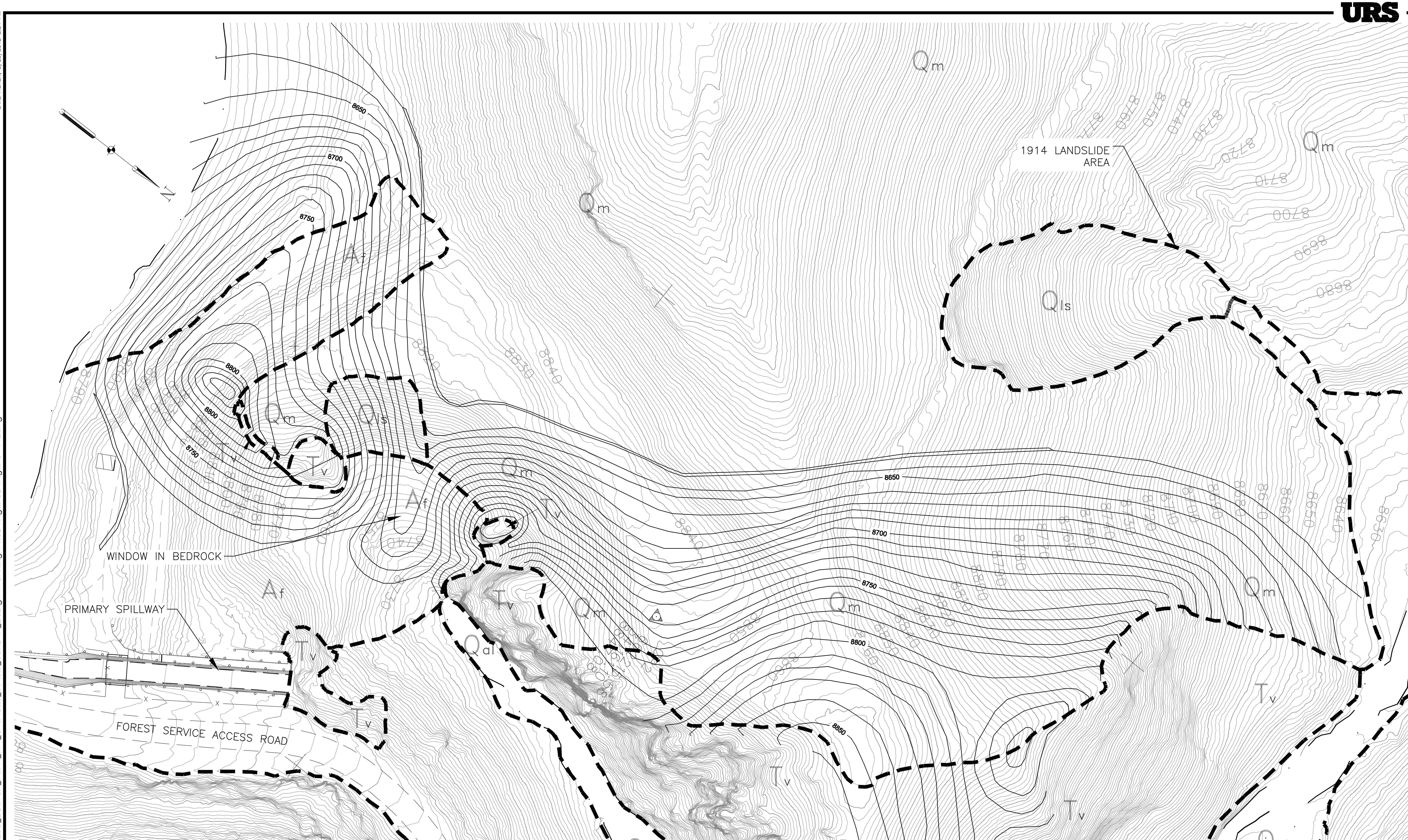
Job No. :	22242295
Prepared By :	CEF
Date :	12/06/11

BEAVER PARK DAM
SURFACE GEOLOGY MAP

FIG. 4-1

N:\Projects\22242295_Beaver_Park_Alt_Std\Sub_00\9.0_Constr_Design\Working Files\figures\Figure 4-1.dwg

PAUL_PERRI, 2/24/2012 8:59 AM



0 37.5 75 150
SCALE IN FEET

Job No.: 22242295	BEAVER PARK DAM BEDROCK GEOLOGY MAP
Prepared By: CEF	
Date: 1/19/12	

FIG. 4-2

5.1 GENERAL

URS developed alternatives to meet the following CPW objectives: a) reduce risks associated with seepage and piping through the left abutment under static (normal operating) conditions (PFM 1), b) accommodate a 3-foot increase in the normal maximum pool level, c) address primary spillway concerns identified by the SEO d) improve the reliability of the operation of the outlet works, and e) removal of the current SEO reservoir restriction.

The alternatives developed included consideration of rehabilitation of the existing dam and construction of a new replacement dam downstream of the existing dam. In the development of the dam rehabilitation alternatives different rehabilitation alternative elements (or design components) were required to achieve the different objectives. In fact, for a complete rehabilitation alternative that addresses all five objectives, five different elements are required. The different rehabilitation alternative elements (elements) were developed separately, in part so that CPW could decide whether it wanted to consider rehabilitation to address only some of the elements, for example, rehabilitation that did not include the 3-foot increase in normal maximum pool or the improved reliability of the outlet works. Ultimately, CPW decided in a design review workshop that the rehabilitation alternative should include all five elements to achieve all five objectives.

The new replacement dam alternative was also developed to achieve all five CPW objectives.

5.2 REHABILIATION ALTERNATIVE ELEMENTS

Based on an initial screening, an element consisting of placement of an upstream blanket to address seepage and piping through the left abutment (PFM 1) was eliminated from further consideration. This element is similar to the reservoir puddle lining operations performed between 1915 and 1916, and it was judged that additional lining would not significantly reduce risk below current levels. It was judged that it would not be possible to adequately identify and treat all potential sources of seepage from the reservoir area into the left abutment to reliably reduce the seepage gradients in the left abutment to prevent internal erosion.

Elements 1A and 1B described below are elements for achieving CPW objective (a), reduction of risk for PFM 1 and (e) removal of existing SEO reservoir restriction. Element 2 described below was developed to achieve CPW objective (d), improvement of outlet works reliability. This was the sole element considered for this objective, for reasons described below. Element 3 described below was developed to achieve CPW objective (c), address SEO concerns regarding the existing spillway and (e) removal of existing SEO reservoir restriction. After extensive consideration, it was judged that this was the only practical element to achieve this objective. Elements 4A, 4B, and 4C are elements developed principally to achieve CPW objective (b), raising the normal pool level 3-feet. These elements would also be desirable to maintain the current normal pool elevation, but they were judged to be necessary for the higher pool level, which would result in more frequent storage of water above the top of the existing concrete wall in the embankment and against the dike on top of the right abutment moraine. Element 5 is not strictly required to achieve the stated CPW objectives, however this element is believed to be a prudent measure to address the seepage observed on the slope of the moraine about 700 feet downstream of the dam. The SEO has also stated that it desires implementation of this element.

As a minimum, element 5 should provide for effective long-term monitoring related to PFM 2, backward erosion piping exiting in the area of the 1914 landslide.

5.2.1 Element 1A - Left Abutment Downstream Filter and Drain System

This element includes the placement of a weighted filter and drain along the slope between the rock knob that forms the left abutment of the dam and the rock knob that is along the left side of the canyon slope near the existing outlet works discharge point. The filter and drain is intended to provide a reasonable level of protection against internal erosion from seepage exiting from the abutment in the window between the two bedrock outcrops and from the contact between the moraine and the bedrock. The filter and blanket would also protect the portal of the abandoned drainage tunnel. Because of the depth to rock in the bottom of the window between the rock knobs, it was judged impractical to extend the filter to rock across the entire width of the window. Rather, the proposed filter was extended out far enough from the slope to make it likely that significant seepage would not flow under the bottom of the collection system.

Figures 5-1 through 5-3 show a plan and sections of the filter and drain system element. An excavated slope of 1.5 horizontal to 1.0 vertical is recommended to provide for a stable slope for placing filter and drain material. Voids in the exposed bedrock, any areas of open work material, and the portal of the drainage tunnel would be treated by filling the voids with a filter compatible coarse grained drain material. The recommended filter and drain thickness is three feet, which is measured normal to the excavated slope and vertically in the bottom of the excavation. For areas that are identified as having open work gravels or larger particles, placement of coarse grained drain material around the exit point for a specified distance prior to placement of the filter is recommended. This would prevent the filter material from falling into the open work materials under gravity.

Seepage flows entering the filter and drain system will be collected by a perforated HDPE pipe or other suitable pipe material that would extend approximately 100 feet from the upstream most portion of the excavated trench down to a point near the discharge of the outlet conduit where flows can be monitored and measured.

To provide weight on top of the filter and drain system to resist the forces that could be exerted from seepage exit locations that may have a direct or indirect connection with the reservoir the system would be backfilled with random fill at a slope of 2H: 1V.

Construction of this design element would require removal and replacement of approximately 120 feet of the existing outlet works conduit. A new steel replacement conduit would be encased in concrete. For purposes of the cost estimate for this element we assumed that dewatering and the removal of the outlet conduit would be a part of Element 2 - Outlet Works Conduit Rehabilitation and Gate Reconfiguration.

5.2.2 Element 1B - Left Abutment Cutoff Wall

This element would involve the construction of a subsurface wall composed of a material that would not be susceptible to erosion due to high seepage gradients. The wall would extend from the ground surface down to the underlying bedrock, which is presumed to be non-erodible. Typically, the wall would be constructed using slurry trench or secant pile methods. The material in the wall would consist of a plastic concrete or conventional concrete to ensure erosion resistance. The wall would connect the rock knob that forms the left abutment of the dam and would extend north, or downstream, of the dam to the rock outcrop that forms the ridge to the north.

The cutoff wall would close a window in the bedrock that exists between the two bedrock outcrops. The length needed to effectively address the concerns with the left abutment was estimated to be about 450 feet north of the existing dam axis. A plan and section of the cutoff wall alignment is shown on Figures 5-4 and 5-5.

5.2.3 Element 2 – Outlet Works Modifications

Reservoir evacuation capability and reliability contribute significantly to estimated probabilities of successful intervention in the event of development of an internal erosion failure mode. Therefore, improved reliability of outlet works operation is highly desirable.

The initial element identified for rehabilitating the outlet works included:

1) Construction of a new outlet works tunnel and pertinent structures.
2) Replacement of the gate operators and rehabilitation of the existing slide gates.
3) Rehabilitation of the existing outlet conduit and replacement and reconfiguration of the control gates.

The construction of a new outlet works tunnel, which would extend through the right abutment of the dam (approximately 600 feet in length), was estimated to cost approximately $2,000 per linear foot for a total tunnel cost of approximately $1,200,000. This figure is limited to the construction of the tunnel only and does not include any other costs, such as upstream and downstream control structures. This cost also does not include the other allowances described in the opinion of probable cost section of this report. Due to the significant costs compared to lining the existing conduit, a detailed evaluation of this alternative was not performed.

The replacement of the gate operators and rehabilitation of the existing slide gates was not developed in detail because we identified the following concerns:

1) The specifics of the existing gates in place are unknown and the gates do not appear to be of standard design and fabrication by any current gate manufacturers.
2) There is limited access to the gates. The reservoir would need to be drawn down or divers would need to be used to access the upstream gate.
3) Poor working conditions. During the 2009 steel liner repair work, the contractor had to deal with significant seepage, restricted work space and poor lighting.
4) This work would have limited benefits related to long-term reliable operations of the outlet works.

The element that was developed to be used in the alternatives is discussed below.

Element 2 - Outlet Works Conduit Rehabilitation and Gate Reconfiguration

This element involves the placement of a steel conduit within the upstream and downstream portions of the existing outlet conduit and grouting the annular space. The existing gates within the gate chamber would be blocked open and a fabricated steel transitional section connecting the upstream and downstream steel liners would be constructed through the existing gate openings. The gate tower would be backfilled with a low strength concrete.

To aid in inspections and maintenance of the conduit and downstream valves, the existing intake structure would be removed and replaced. A new intake structure would be constructed, which would include trashracks and a hydraulically operated guard gate. An air vent pipe and hydraulic lines encased in concrete would extend up the face of the dam.

To control reservoir releases, a new control structure with a hydraulically operated knife gate and baffled dissipation structure would be constructed downstream of the dam. Figures 5-6 through 5-8 show the profile and sections for the rehabilitated outlet works element.

The estimated peak discharge of the rehabilitated outlet at a maximum reservoir water surface elevation (WSE) of 8807 is about 357 ft^3/s. The estimated rating curve of the proposed rehabilitated outlet works is summarized in Table 5-1.

Table 5-1 Estimated Outlet Works Rating Curve Summary

WSE (ft)	***Existing Discharge (ft^3/s)***	***Modified Discharge (ft^3/s)***
8807	585	357
8800	564	344
8790	531	325
8780	504	299
8770	468	281
8760	420	257
8750	376	230
8740	325	199
8730	266	163
8724	222	137

The discharge of the original and modified configuration of the outlet works is outlet controlled. The limiting factor in the discharge potential of the outlet conduit is the overall length of the conduit. This anticipated reduction in discharge capacity could result in more frequent flow occurrences through the primary spillway.

5.2.4 Element 3 - Primary Spillway Concrete Chute Modifications

The existing primary spillway consists of a reinforced concrete side channel inlet structure and chute on the right abutment of the dam. Flow through the spillway discharges from the chute onto a rock covered slope that connects with Beaver Creek at the base of the canyon near the downstream end of the outlet works conduit. Deterioration and undermining of the concrete apron at the end of the chute has caused concern about the long term stability of the chute, and the potential for continued erosion and undercutting if left unchecked

For rehabilitating the primary spillway chute, we initially considered constructing a traditional concrete lined chute from the invert of the spillway control to the toe of the dam with a concrete stilling basin. However, due to the near vertical joints in the bedrock and the overall steepness of the bedrock exposed near the dam, it was felt that founding the structure on bedrock was not economically practical. Therefore, an element for stabilizing the rock slope below the existing spillway chute during more frequent floods was developed. This element was developed with the understanding that during less frequent, larger storm flows, headcutting of the concrete lined chute may be re-initiated. More detailed discussions of this element are provided in the following paragraphs.

The intent of this element is to eliminate a drop at the end of the chute, and provide better control of the discharges during normal spillway operations. The element consists of removing about 24-feet of the end of the existing chute, and constructing a reinforced concrete lined stair-stepped channel and containment wall, with a vertical drop of 20 feet. This new section of chute would be constructed in the bedrock of the right abutment. A grouted riprap lined 20-foot wide channel would be constructed from the end of the chute down to Beaver Creek. The channel alignment would be improved by intercepting the creek in a downstream direction, and farther downstream from the end of the outlet works conduit. Figures 5-9 through 5-11 show the plan, profile and section of the primary concrete chute spillway modifications.

This element will improve operations and flow characteristics during normal spillway operations. However, during floods that exceed the existing spillway capacity, flow will overtop and discharge down the access road along the right abutment. During these larger floods, and certainly for floods approaching the PMF, erosion of the shell materials near the downstream toe of the embankment may occur. However, these occurrences are expected to be very remote and any erosion that would occur is not expected to affect the safe performance of the dam during these events. Repairs should be performed immediately following significant spillway discharge events.

This element also incorporates a 3-foot high spillway crest raise. The design consists of removing the top 2.5 feet of the existing reinforced concrete crest structure, and extending the shape of the crest geometry up to the raised elevation. An overhang into the reservoir side of the crest structure will be required to retain the crest geometry with an efficient flow surface.

The estimated rating curve of the primary spillway with a 3-foot raise is summarized in Table 5-2. This rating curve was obtained from the 2011 Draft Hydrology Study (CPW).

Table 5-2 Estimated Combined Primary Spillway and Outlet Works Rating Curve

WSE (ft)	*Primary Spillway (ft^3/s)*	*Outlet Works Discharge (ft^3/s)*	*Combined discharge (ft^3/s)*
8807	0	357	357
8808	218	359	577
8812	2,060	366	2,426
8816	3,497	374	3,871
8820	4,868	381	5,249
8824	10,077	387	10,464
8828	17,849	394	18,243
8832	26,778	401	27,179
8833.5	30,407	403	30,810

It is anticipated that the access road parallel to the primary spillway will act as an auxiliary spillway for storm events greater than the 50-year event. Additional modifications to the primary spillway to contain flows in the spillway channel can be explored during final design of this element

5.2.5 Element 4 – Dam Crest Modifications

Three elements were developed to address seepage concerns and uncertainties related to flow through the dam crest and left abutment rock knob during elevated pools during flood passage. There is uncertainty about the material used in the two past crest raises and its ability to temporarily store water without through seepage and piping. There is also uncertainty about the condition and elevation of the top of the concrete face/wall in the dam, particularly where the original spillway was blocked with a concrete wall. The foundation condition in the rock knob that forms the left abutment of the dam is also uncertain and a source of concern with respect to seepage under normal pool operations and flood pools.

The following elements are shown on Figures 5-12 and 5-13.

Upstream High Density Polyethylene Liner (Element 4A)

This element would involve the placement of a high density polyethylene (HDPE) liner material connected to the existing concrete wall in the embankment and extending the liner along an excavated slope to an elevation that is above the maximum reservoir elevation estimated to result from the local PMF event. The HDPE liner would extend approximately 280 feet west/southwest from the primary spillway mechanically stabilized earth (MSE) wall along the dam axis to the left abutment. A portion of the MSE wall will be replaced with a reinforced concrete wall to provide a suitable connection location for the HDPE liner. To protect the HDPE liner against puncture, a geotextile fabric and a uniformly graded select fill material will be placed above and below the HDPE liner. The embankment would be restored to the existing crest elevation and upstream slope using the excavated dam embankment material.

Low Permeability Fill (Element 4B)

This element would involve the placement of a low permeability fill material along the alignment of the existing crest wall. The top elevation of the fill material would be above the elevation of the maximum reservoir surface elevation estimated for the local PMF event. To protect the low permeability fill against movement of material into the downstream rockfill a two-stage filter and drain system will be constructed. The filter would consist of a free draining material that is filter compatible with the low permeability fill material. The drain would consist of a coarse grained free draining material that is filter compatible with the filter material and has adequate capacity to collect seepage flows into a toe drain system where flows can be safely discharged. The low permeability fill material would extend approximately 280 feet west/southwest from the primary spillway MSE wall along the axis of the existing crest wall to the left abutment. A portion of the MSE wall would be replaced with a reinforced concrete wall to provide a suitable surface to compact the low permeability fill against. The downstream filter and drain material would be placed against the downstream surface of the low permeability fill. The dam embankment would be restored to the existing crest elevation and slopes using the excavated dam embankment material.

Raised Crest Wall (Element 4C)

This element utilizes the existing alignment of the crest wall for placement of a reinforced concrete wall extension. The top elevation of the wall would be above the elevation of the maximum reservoir surface elevation estimated for the local PMF event. The raised wall would extend approximately 280 feet west/southwest from the primary spillway MSE wall along the axis of the existing wall to the left abutment. A portion of the MSE wall would be replaced with a reinforced concrete wall to provide for a secure watertight connection with the raised wall. The dam embankment would be restored to the existing crest elevation and slopes using the excavated dam embankment material.

Transition between the Dam Crest and the Dike Crest

A mass concrete block section is proposed to provide a continuous upstream connection for Elements Nos. 4A through 4C at the contact between the dam crest and the dike crest. The concrete block section would provide a surface suitable for connecting the HDPE liner against; compacting the low permeability core against; terminating the raised crest wall into and placing the zones of the dike crest against. This transition structure is needed because the rock knob was excavated to an elevation well below the dam crest. There is uncertainty about the condition and the elevation of the rock in this area. As a minimum, dental concrete would be required to treat the rock in the area of the concrete block. Figures 5-12 and 5-13 show a plan and typical section of the transition.

Left Abutment Crest Dike filter/Drain System

To protect the left abutment crest dike from seepage flows that could develop as a result of flood surcharge storage or a raise in the maximum reservoir pool elevation, the filter and drain system would extend up to about elevation of 8827 between Stations 15+00 and 16+00. A typical section of the extension is shown on Figure 5-14

5.2.6 Element 5 - 1914 Landslide Area Filter and Drain System

The landslide occurred in 1914 approximately 700 feet north northwest of the dam (seepage path length from the reservoir to the base of the slide of about 800 ft). A filter, drain, and seepage collection system were proposed for this area in addition to the above elements. This seepage collection system would be focused on the seepage area at the base of the 1914 slide and possibly on other significant seepage exits in the area. This element will provide CPW with the ability to effectively monitor the seepage in the area and identify changes. . This element would also provide protection against piping of moraine in the areas of observed seepage. Figure 5-15 shows a plan and typical section of the filter and drain system. This layout is only conceptual. Additional investigations and field observations will be needed to determine the actual design configuration.

5.3 ALTERNATIVES

Two alternatives were developed and evaluated for the rehabilitation of the existing dam and one alternative was developed consisting of the construction of a new dam.

Alternatives related to rehabilitation of the existing dam were developed to address:

1) Risks associated with seepage and piping through the left abutment rock knob and moraine under static conditions (PFM 1).
2) Seepage concerns through the Phase 1 dam embankment raise completed in 1988 and dike section along the left abutment.
3) Primary spillway discharge concerns identified by the SEO.
4) Improve the reliability of the operation of the outlet works.
5) Removal of the current SEO restriction.

Each alternative is discussed in detail below.

5.3.1 Alternative 1 - Dam Rehabilitation

This alternative was broken down into two alternatives which focused around Element 1A or Element 1B, which specifically addresses reducing the risk associated with PFM 1.

Alternative 1A - Left Abutment Filter and Drain System

This Alternative groups the following five elements:

- Element 1A - Left Abutment Downstream Filter and Drain System. This element addresses the left abutment seepage risk (PFM 1)
- Element 2 - Outlet Works Conduit Rehabilitation and Gate Reconfiguration. This element provides for a reliable outlet works that can be used to effectively regulate reservoir pool elevation and/or to evacuate the reservoir in the event of an emergency.
- Element 3 - Primary Concrete Chute Spillway Modifications. This element addresses the dam safety concerns identified by the SEO.
- Element 4 - Crest Modifications. These elements accommodate normal pool operations, flood surcharge, and the proposed 3-foot raise in the reservoir pool elevation.
- Element 5 - 1914 Landslide Area Filter and Drain System. This element provides prudent measure to address observed downstream seepage.

Alternative 1B - Left Abutment Cutoff Wall

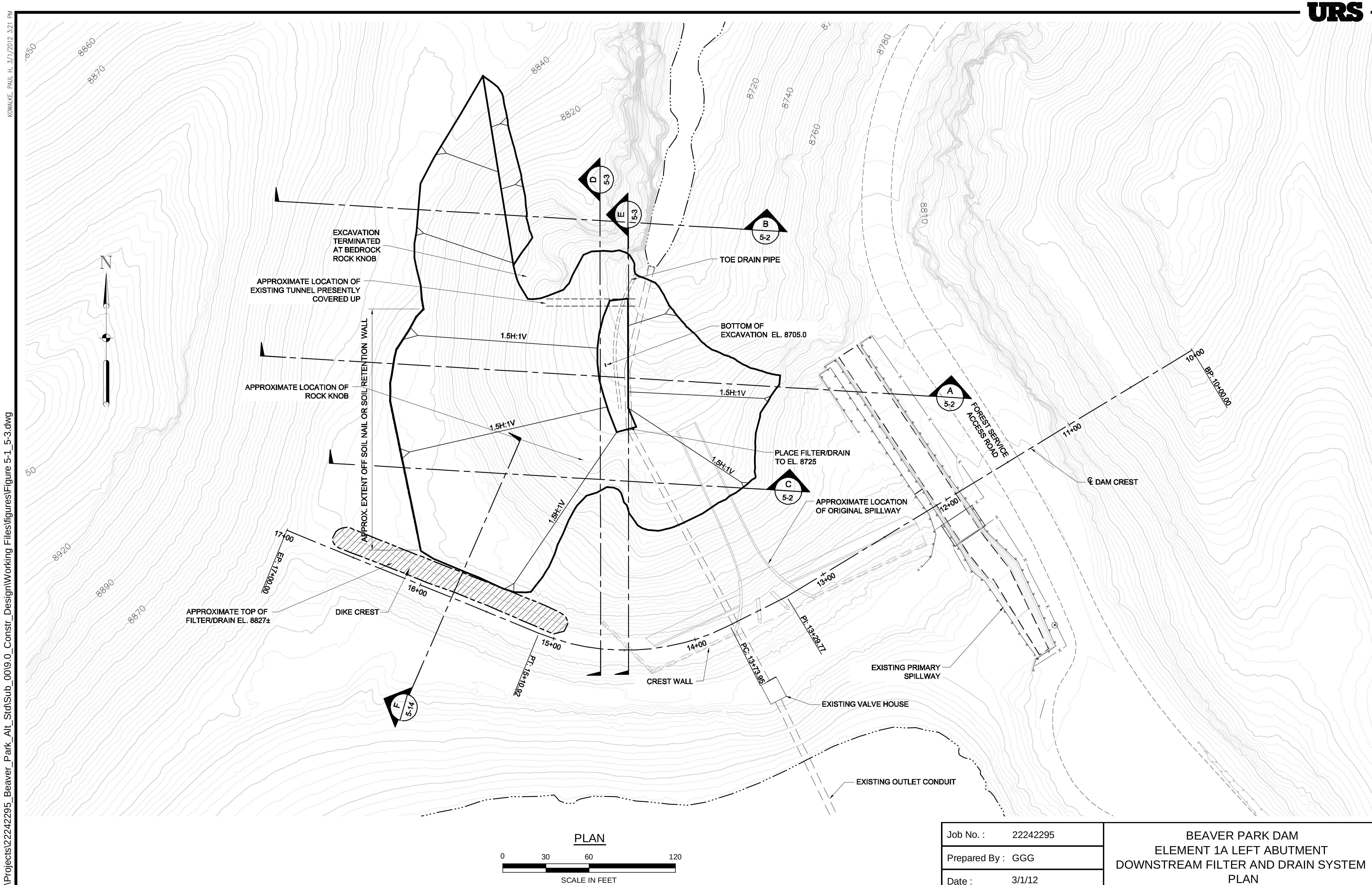
Element 1B - Left Abutment Cutoff Wall This element was developed as an alternate approach to addressing the left abutment seepage risk (PFM 1). Elements 2 through 5 remain as outlined above to address the issues highlighted in Alternative 1A.

5.3.2 Alternative 2 - New RCC Dam

The alternative to construct a new RCC dam downstream of the existing dam was considered. The dam would be approximately 130 feet high, measured from the assumed upstream contact with bedrock to the top of the upstream parapet wall. The emergency spillway would be located in the center of the dam and controlled by an ogee crest.

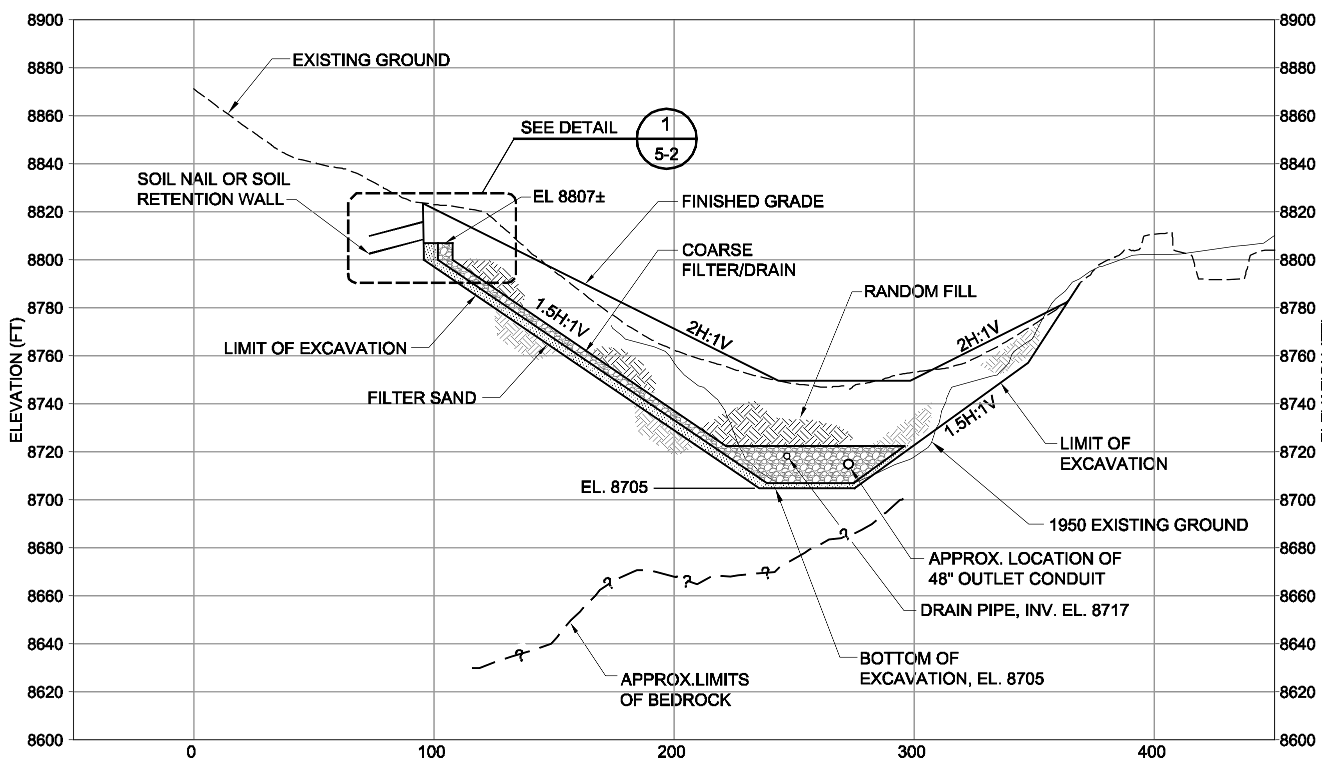
The proposed alignment of the RCC dam would be downstream of the existing dam. To achieve a reservoir capacity comparable to the capacity with the desired 3-foot raise in storage, the crest of the RCC dam would be above the existing County Road 20, requiring relocation of the road to accommodate the RCC dam. Extensive treatment of the left abutment area foundation is also anticipated, because the geology in the left abutment at the new dam location appears to be similar to or perhaps worse than that at the existing dam. Figures 5-16 and 5-17 show a simplified plan and profile of this alternative.

Construction of a new RCC dam downstream of the existing dam may resolve some of the PFMs identified in the 2008 risk assessment; however, construction of the dam may in fact aggravate some of the PFMs and create new ones. The failure mode through the moraine in the area of the 1914 landslide (PFM 2) could pose a major design challenge as the seepage distance from the reservoir to the seepage exit would be decreased. This decrease in seepage length would increase the seepage gradient through the moraine and the increased gradient could increase the risk from this failure mode.



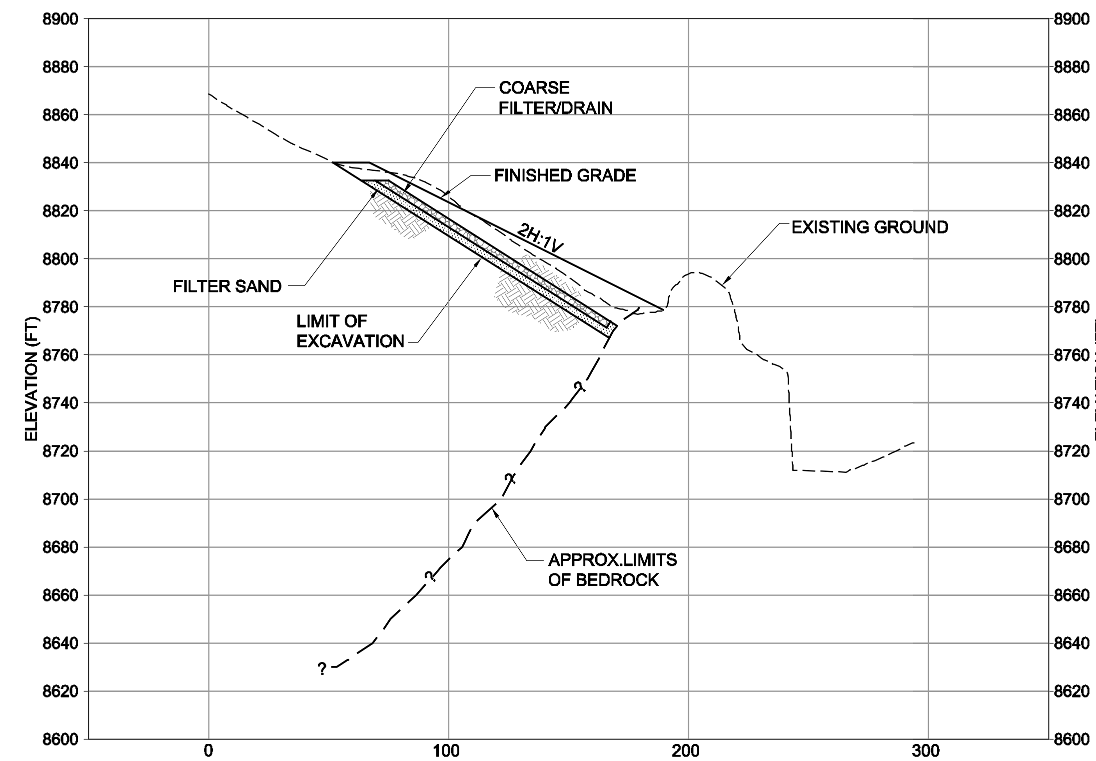
Job No. : 22242295	BEAVER PARK DAM
Prepared By : GGG	ELEMENT 1A LEFT ABUTMENT DOWNSTREAM FILTER AND DRAIN SYSTEM
Date : 3/1/12	PLAN

FIG. 5-1



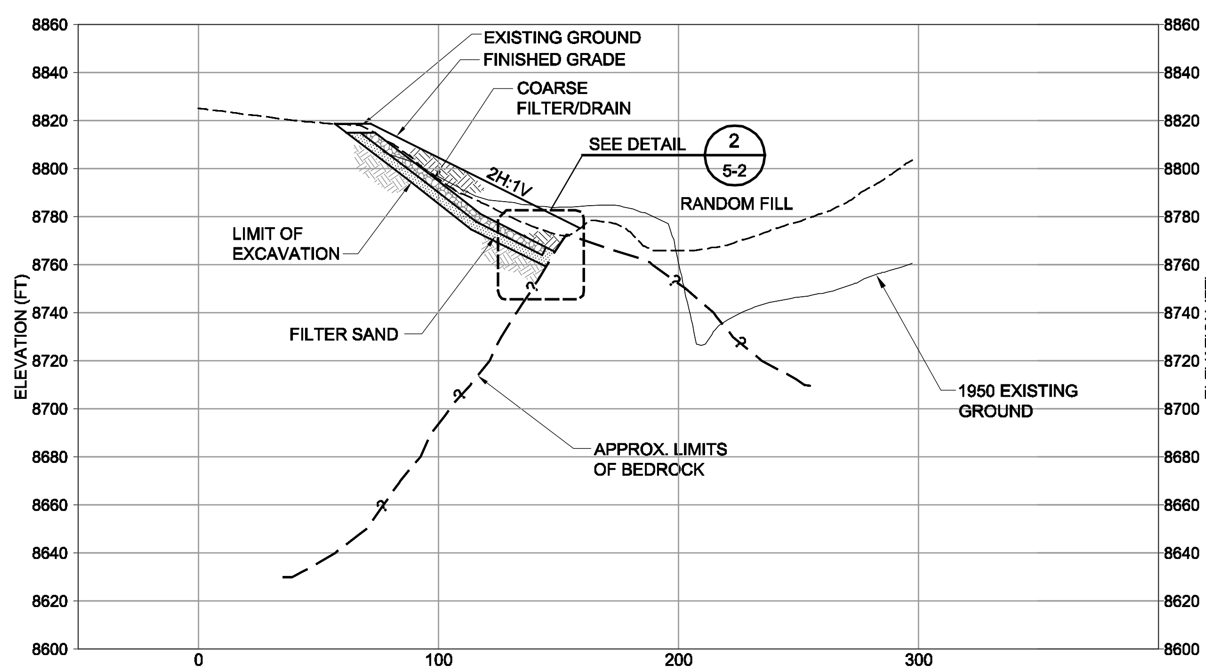
FILTER BLANKET SECTION A 5-1

0 40 80 120
SCALE IN FEET



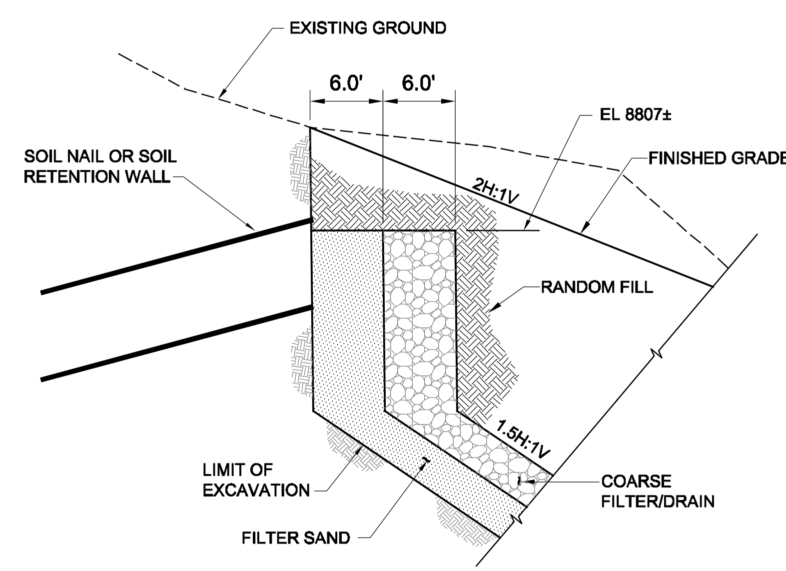
SECTION B 5-1

0 40 80 120
SCALE IN FEET

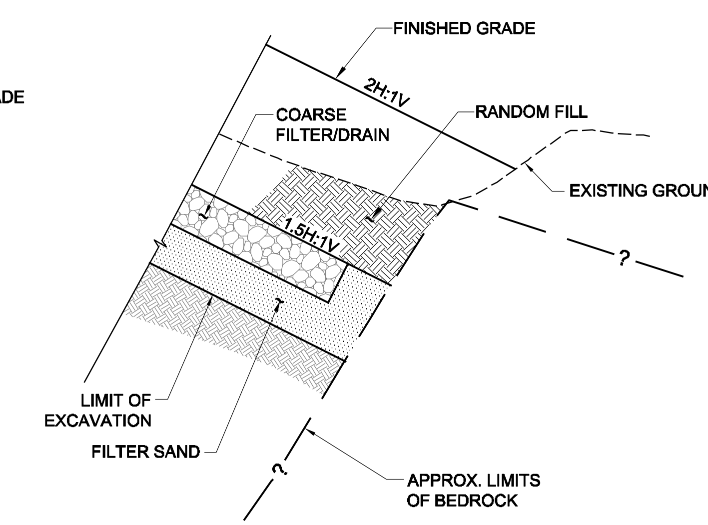


SECTION C 5-1

0 40 80 120
SCALE IN FEET



DETAIL 1 5-2
NOT TO SCALE



DETAIL 2 5-2
NOT TO SCALE

Job No. : 22242295	BEAVER PARK DAM ELEMENT 1A LEFT ABUTMENT DOWNSTREAM FILTER AND DRAIN SYSTEM SECTION AND DETAILS
Prepared By : GGG	
Date : 2/20/12	

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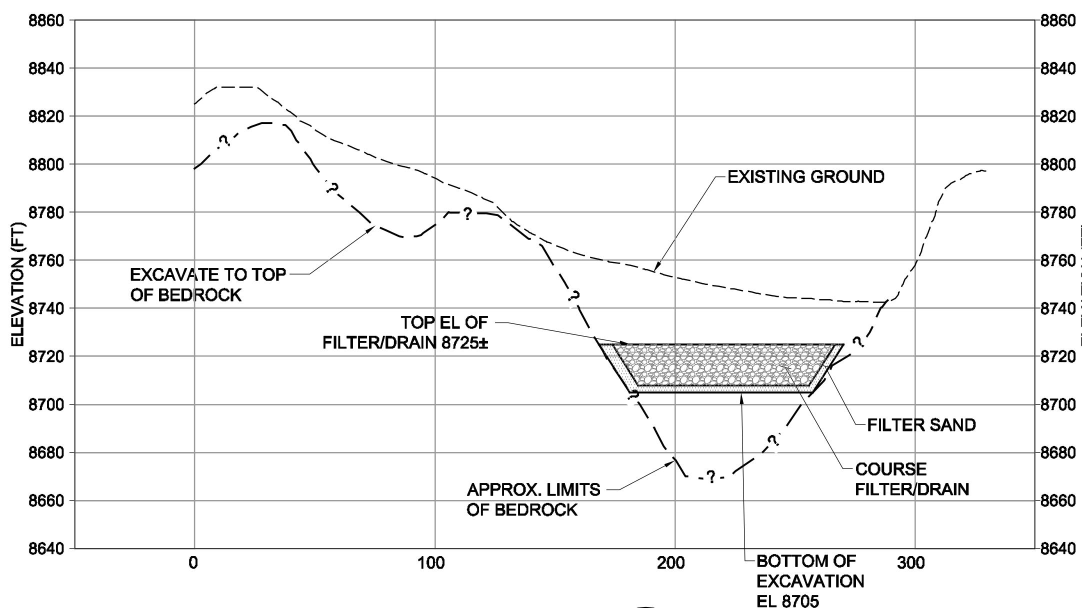
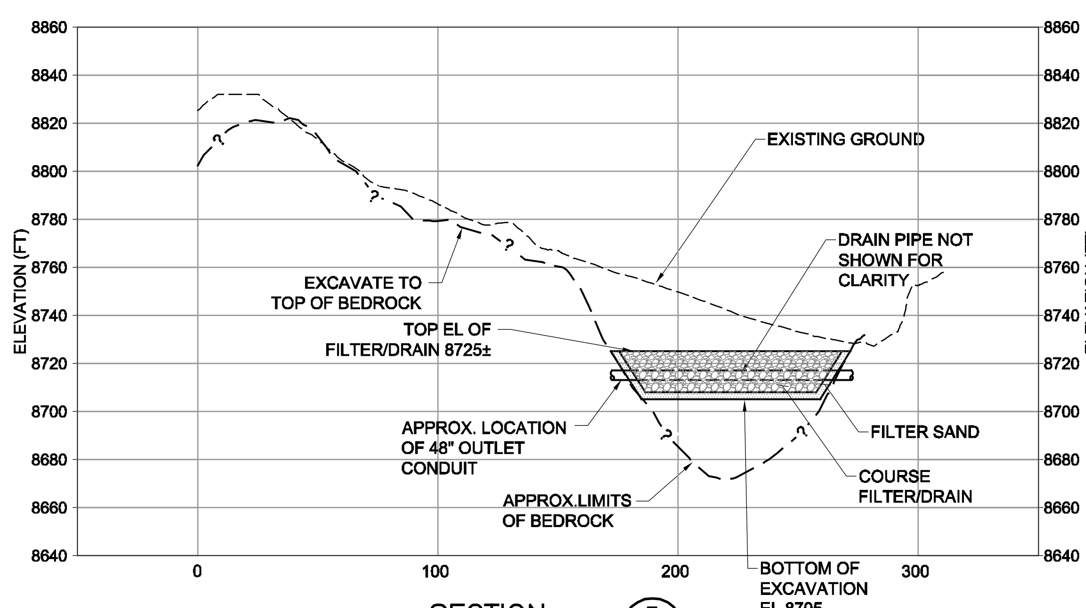


SECTION D 5-1

0 40 80 120
SCALE IN FEET



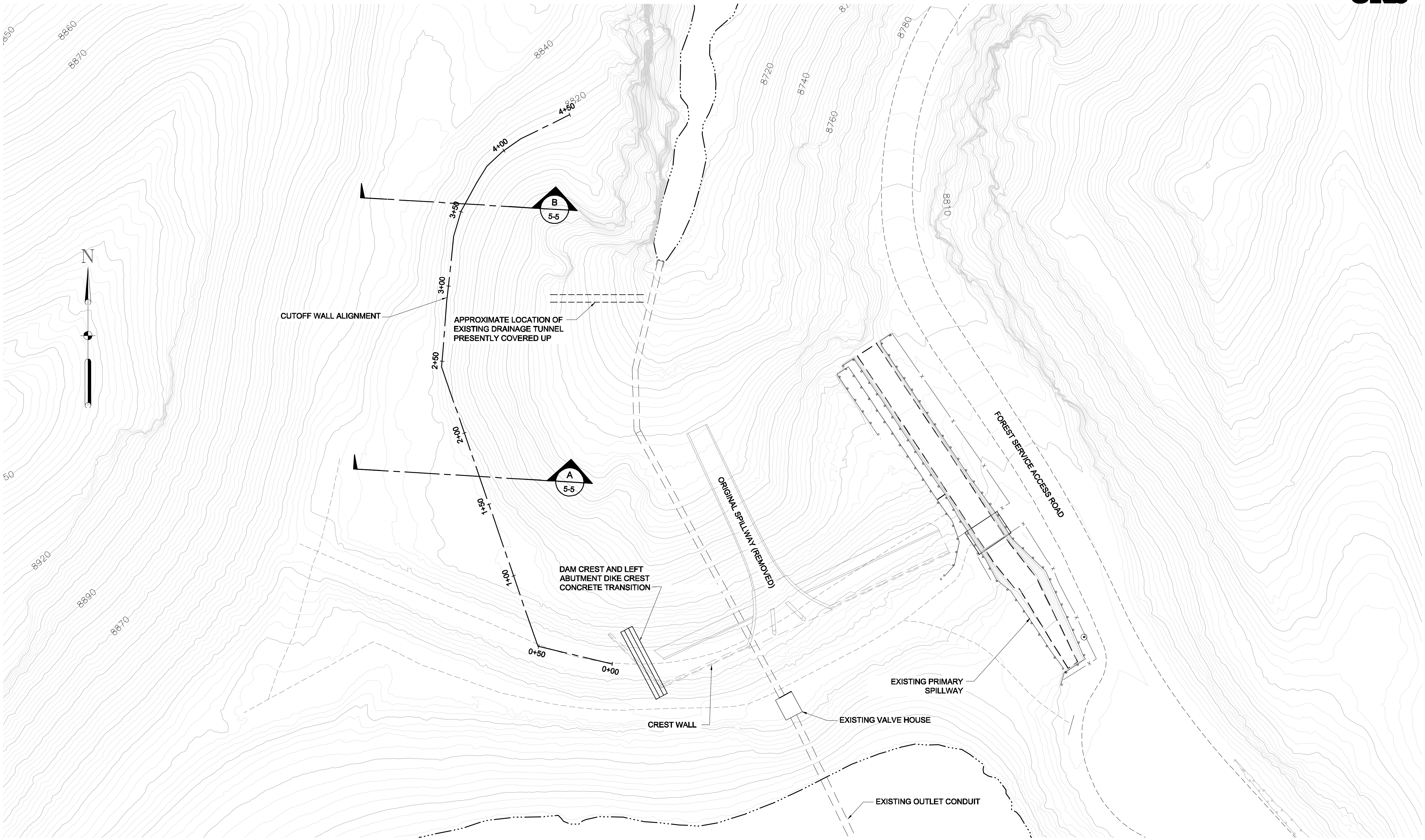
SECTION E 5-1

0 40 80 120
SCALE IN FEET

N:\Projects\22242295_Beaver_Park_Alt_Std\Sub_0019.0_Constr_Design\Working Files\figures\Figure 5-1_5-3.dwg

Job No. :	22242295	BEAVER PARK DAM ELEMENT 1A LEFT ABUTMENT DOWNSTREAM FILTER AND DRAIN SYSTEM SECTIONS
Prepared By :	GGG	
Date :	2/20/12	

FIG. 5-3



PLAN

0 30 60 120
SCALE IN FEET

Job No. :	22242295
Prepared By :	GGG
Date :	2/20/12

BEAVER PARK DAM
ELEMENT 1B LEFT ABUTMENT
CUTOFF WALL
PLAN

FIG. 5-4

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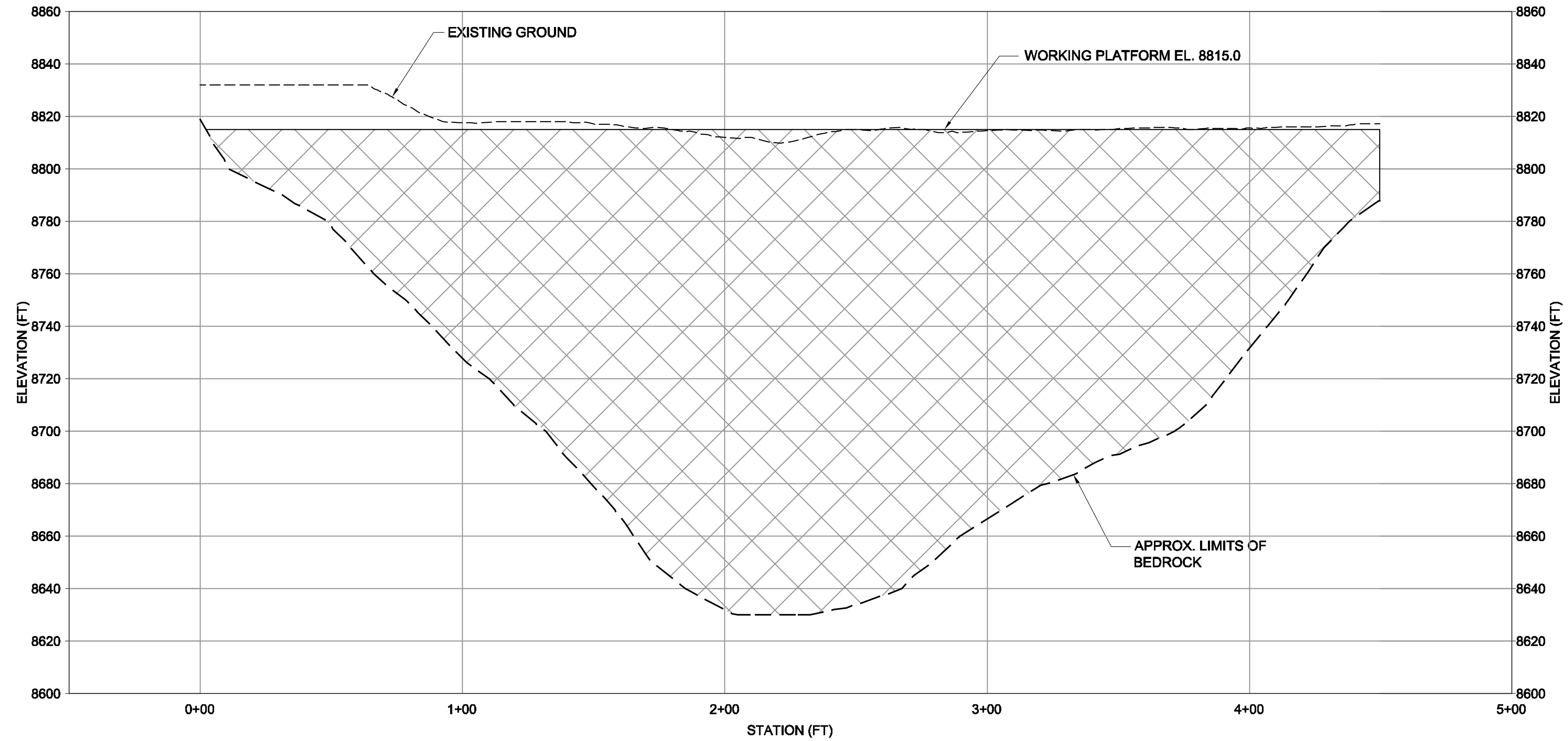
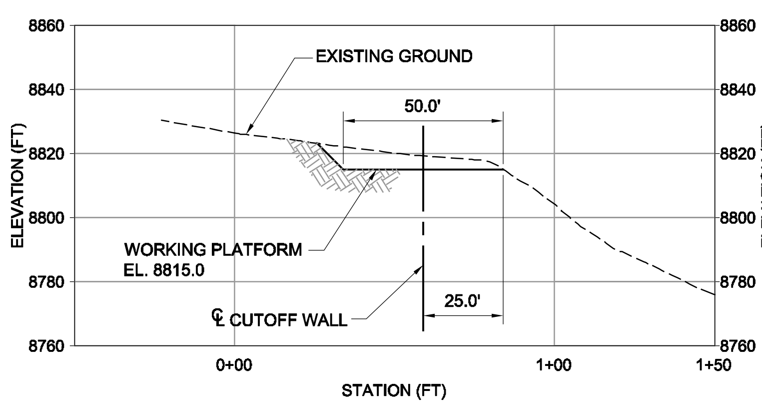


LEFT ABUTMENT CUTOFF WALL PROFILE

0 30 60 90

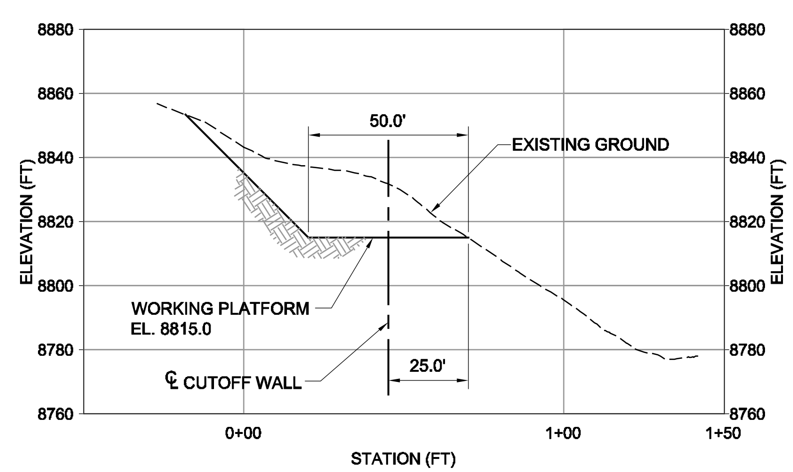
SCALE IN FEET



SECTION A 5-4

0 30 60 90

SCALE IN FEET



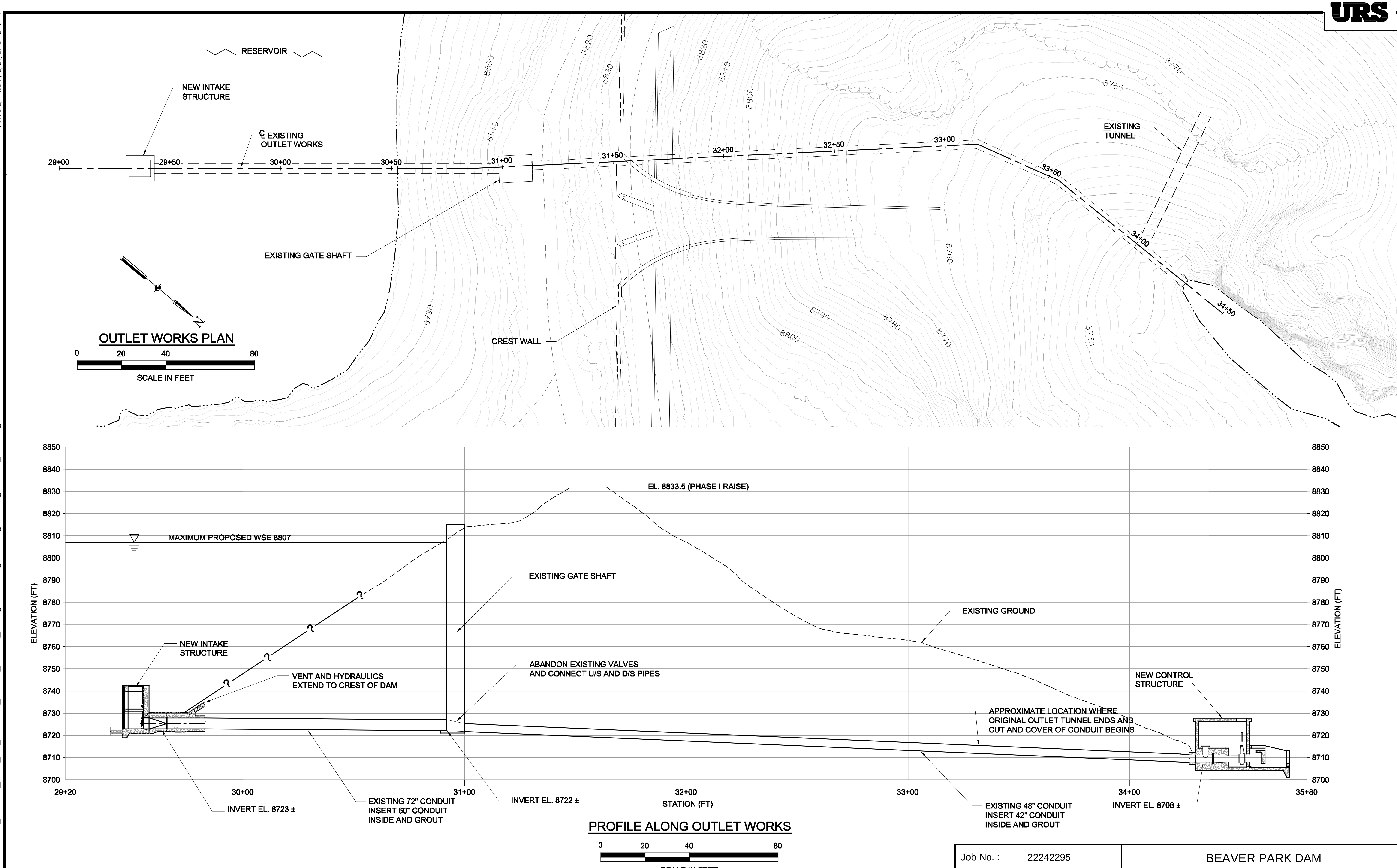
SECTION B 5-4

0 30 60 90

SCALE IN FEET

Job No. :	22242295	BEAVER PARK DAM ELEMENT 1B LEFT ABUTMENT CUTOFF WALL PROFILE AND SECTIONS
Prepared By :	GGG	
Date :	2/20/12	

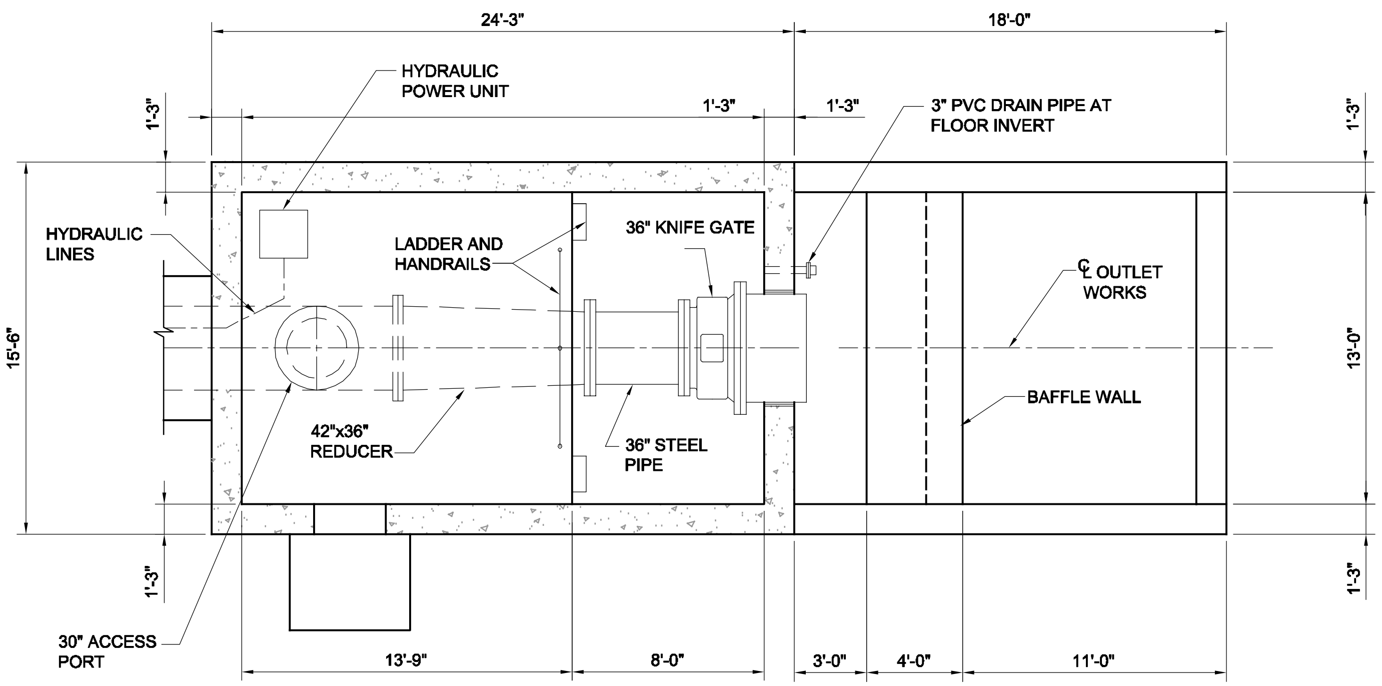
FIG. 5-5



Job No. : 22242295	BEAVER PARK DAM ELEMENT 2 OUTLET WORKS CONDUIT REHABILITATION AND GATE RECONFIGURATION PLAN AND PROFILE
Prepared By : GGG	
Date : 2/24/12	

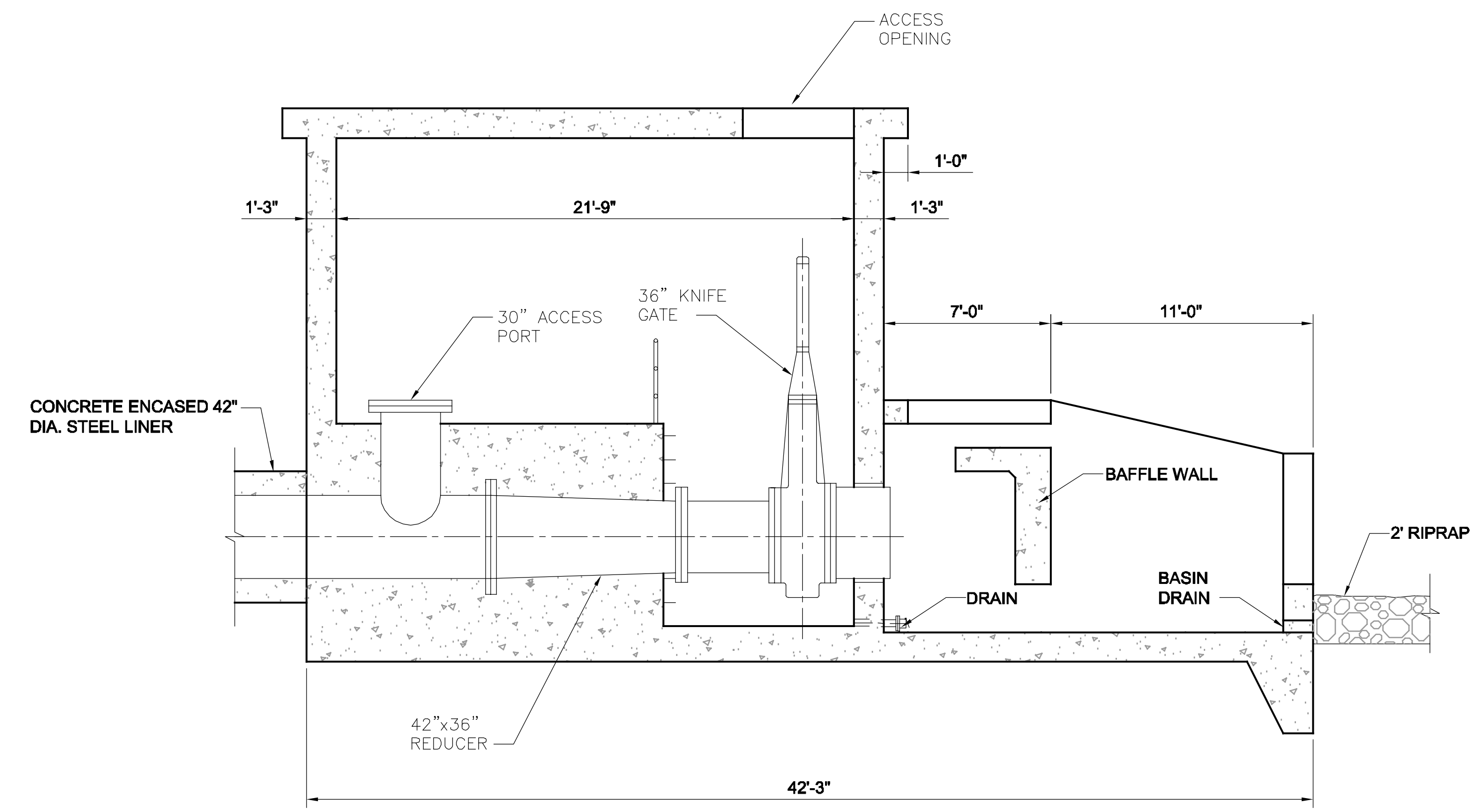
FIG. 5-6

URS



PLAN

0 4 8 12
SCALE IN FEET



PROFILE

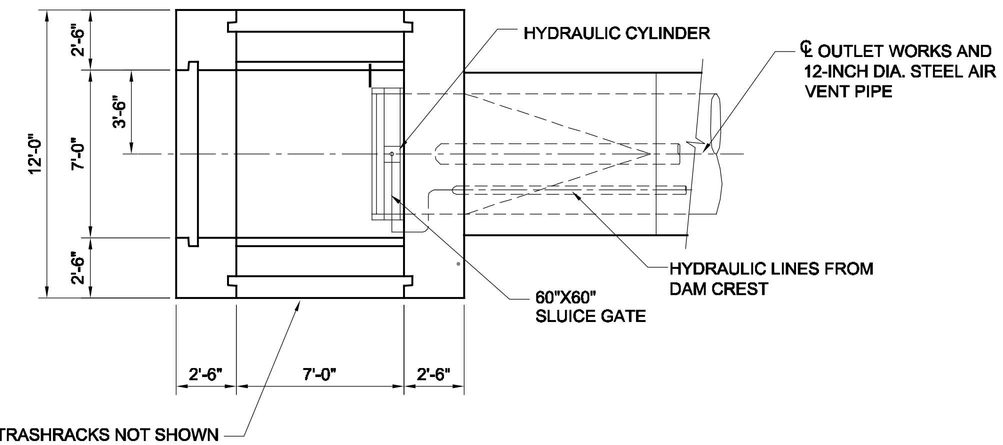
0 4 8 12
SCALE IN FEET

DOWNSTREAM CONTROL STRUCTURE

Job No. :	22242295
Prepared By :	GGG
Date :	2/20/12

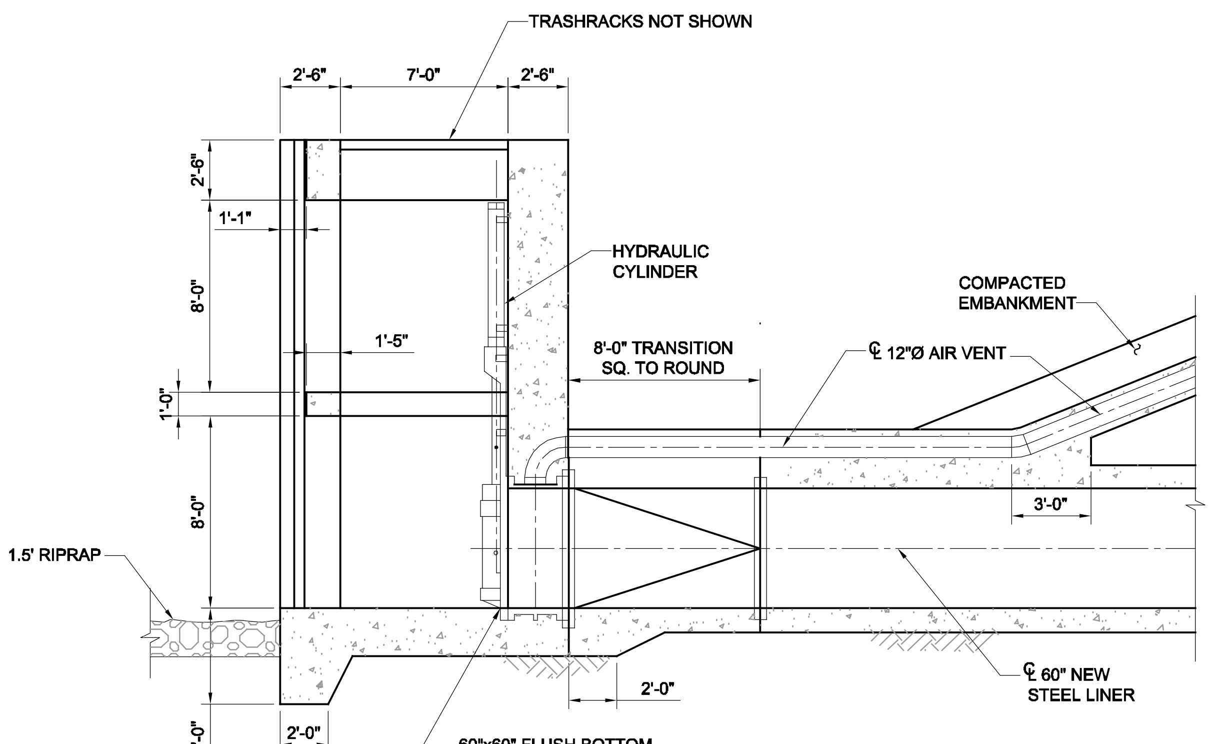
BEAVER PARK DAM
ELEMENT 2 OUTLET WORKS
REHABILITATION AND GATE
RECONFIGURATION CONTROL STRUCTURE

FIG. 5-7



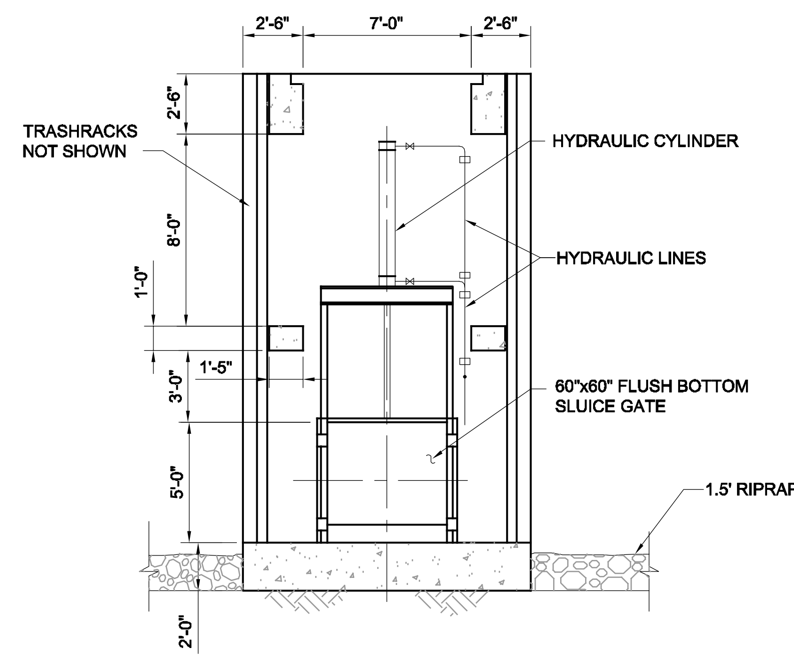
PLAN

SCALE IN FEET



PROFILE

SCALE IN FEET



SECTION

SCALE IN FEET

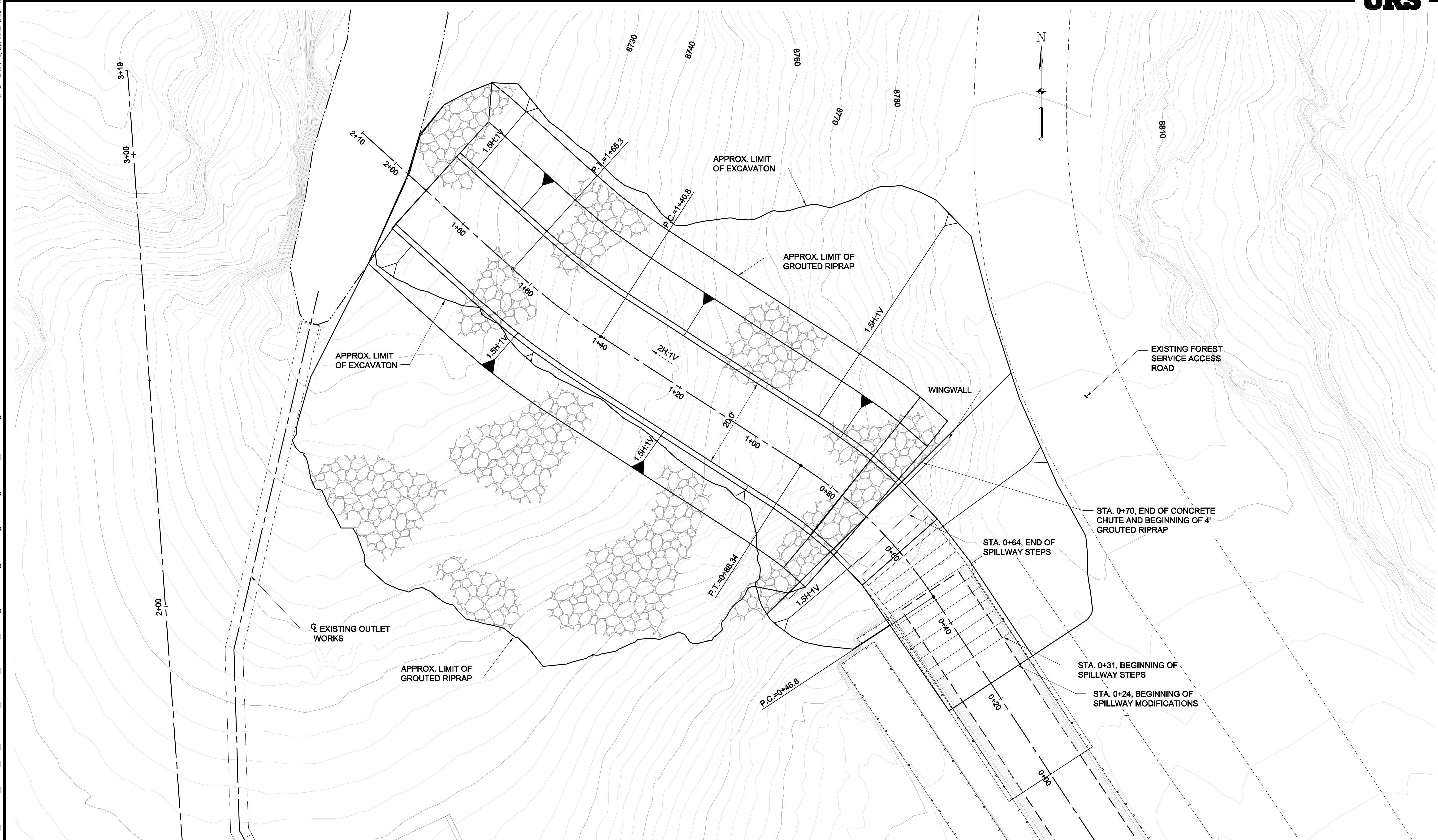
UPSTREAM INTAKE STRUCTURE

Job No. :	22242295
Prepared By :	GGG
Date :	2/22/12

BEAVER PARK DAM
ELEMENT 2 OUTLET WORKS CONDUIT
REHABILITATION AND GATE
RECONFIGURATION INTAKE STRUCTURE

FIG. 5-8

URS



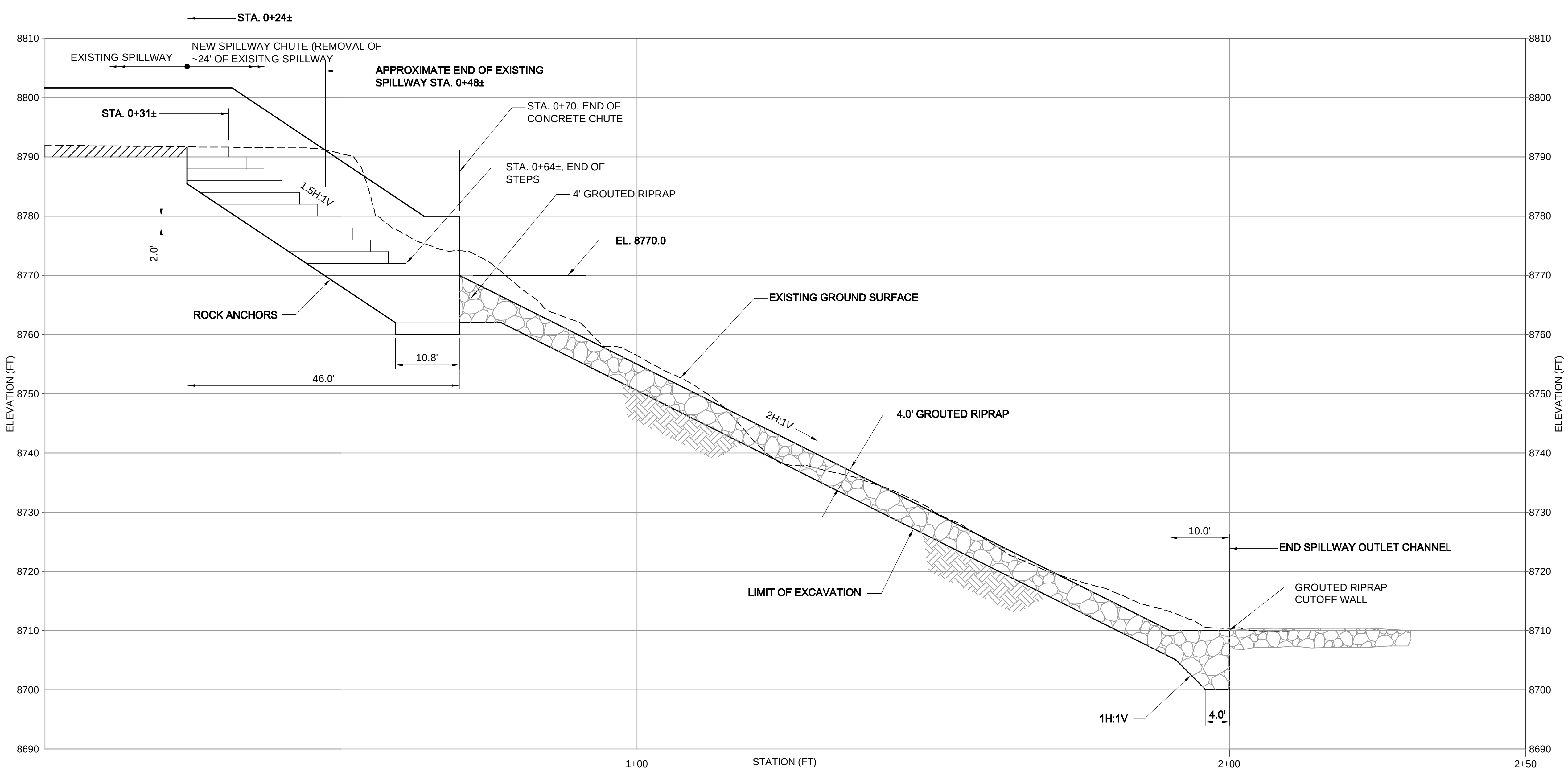
0 10 20 40

SCALE IN FEET

Job No. :	22242295	BEAVER PARK DAM ELEMENT 3 PRIMARY SPILLWAY CONCRETE CHUTE MODIFICATIONS PLAN
Prepared By :	GGG	
Date :	2/20/12	

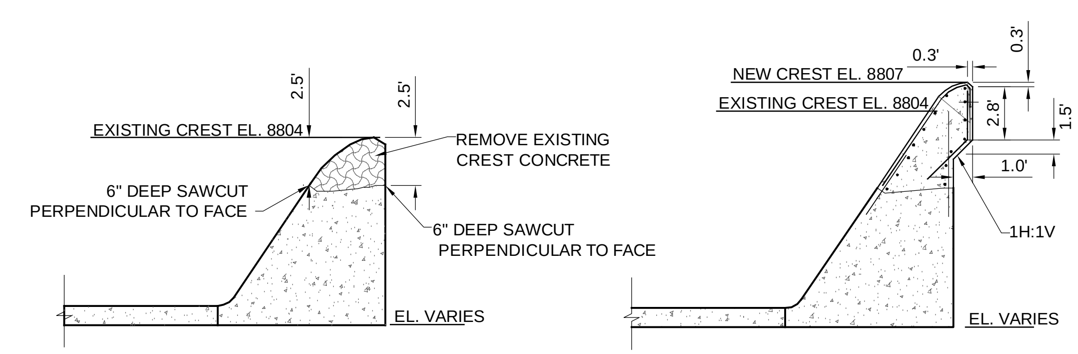
FIG. 5-9

URS



PRIMARY SPILLWAY PROFILE

0 20 40 60
SCALE IN FEET

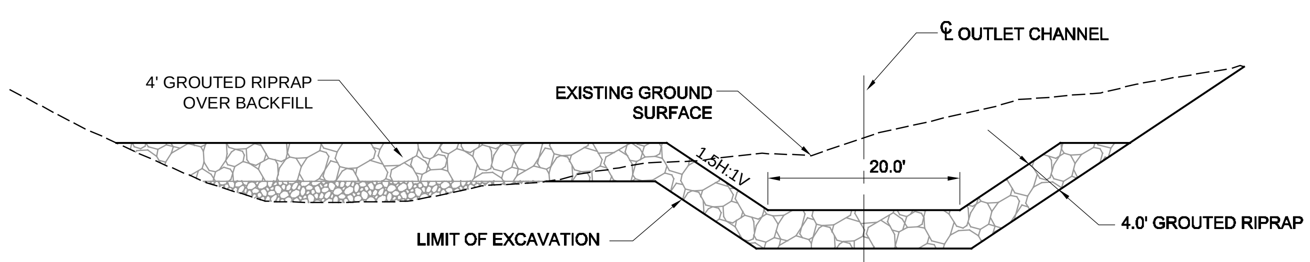


EXISTING CREST

RAISED CREST

SPILLWAY CREST RAISE ALTERNATIVE

0 10 20 30
SCALE IN FEET



TYPICAL PRIMARY SPILLWAY OUTLET CHANNEL SECTION

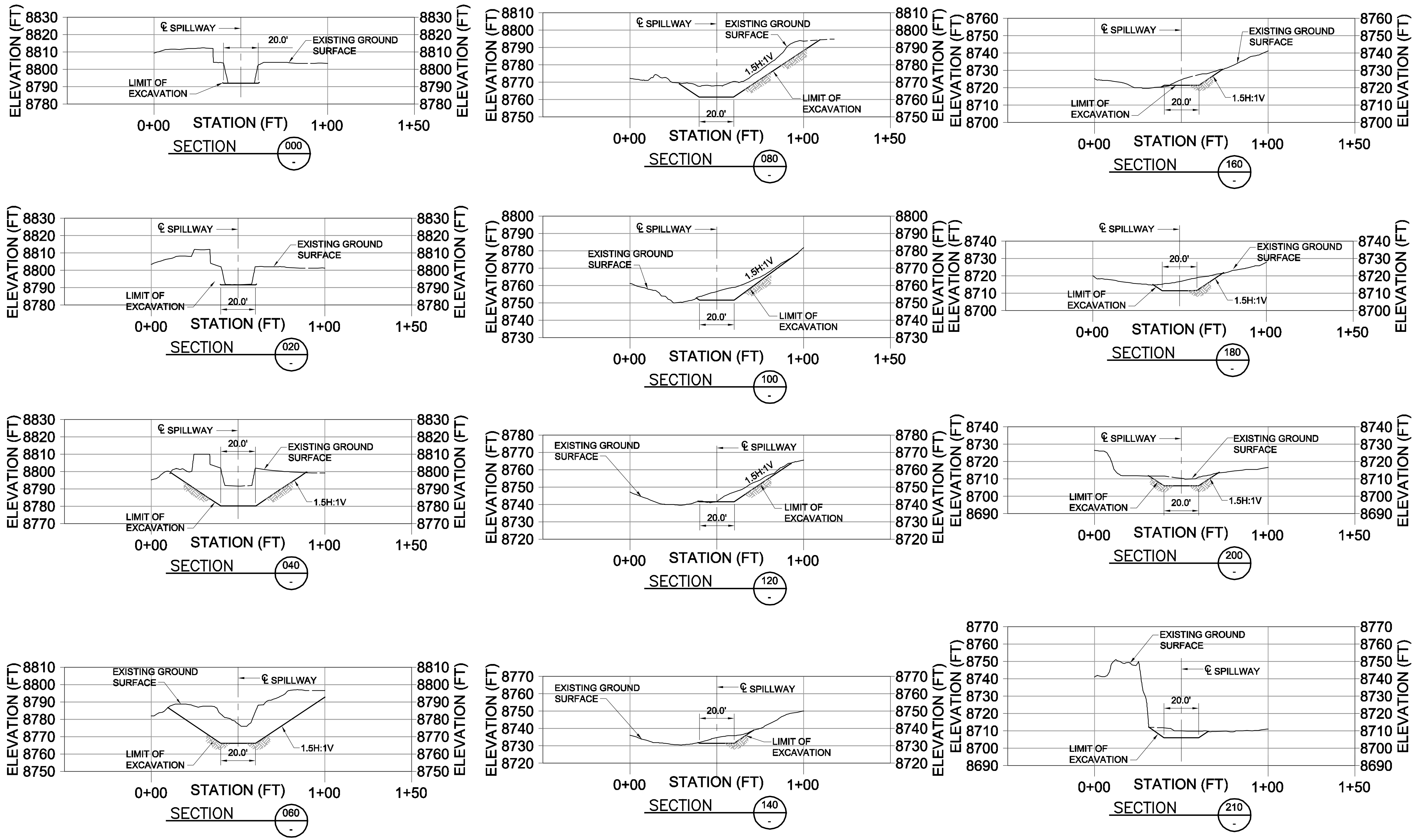
0 10 20 30
SCALE IN FEET

Job No. :	22242295
Prepared By :	SSH
Date :	2/24/12

BEAVER PARK DAM
ELEMENT 3
PRIMARY SPILLWAY CONCRETE CHUTE
MODFICATIONS
PROFILE AND SECTION

FIG. 5-10

URS



NOTE: SECTIONS ARE LOOKING DOWNSTREAM

0 30 60 120
SCALE IN FEET

Job No. :	22242295
Prepared By :	GGG
Date :	2/20/12

BEAVER PARK DAM
ELEMENT 3
PRIMARY SPILLWAY CONCRETE
CHUTE MODIFICATIONS
SECTIONS

FIG. 5-11

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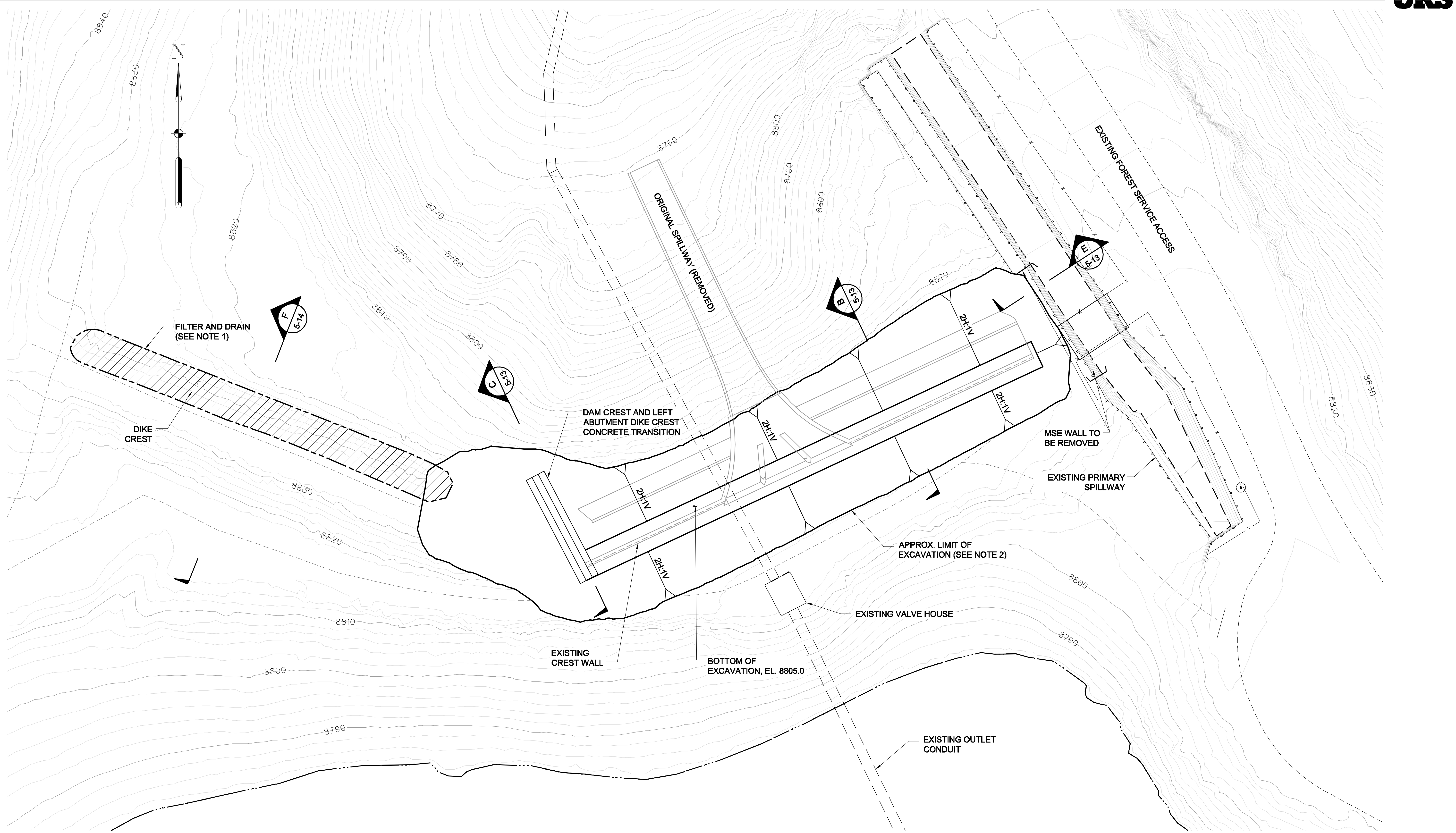

NOTES:

1. FILTER AND DRAIN TO BE PLACED ON DOWNSTREAM SLOPE.
2. EXCAVATION SHOWN IS FOR ELEMENT NO. 4 (FIGS. 5-11 AND 5-12).

PLAN

0 20 40 80

SCALE IN FEET

Job No. :	22242295	BEAVER PARK DAM ELEMENT 4 DAM CREST MODIFICATIONS PLAN
Prepared By :	GGG	
Date :	2/20/12	

FIG. 5-12

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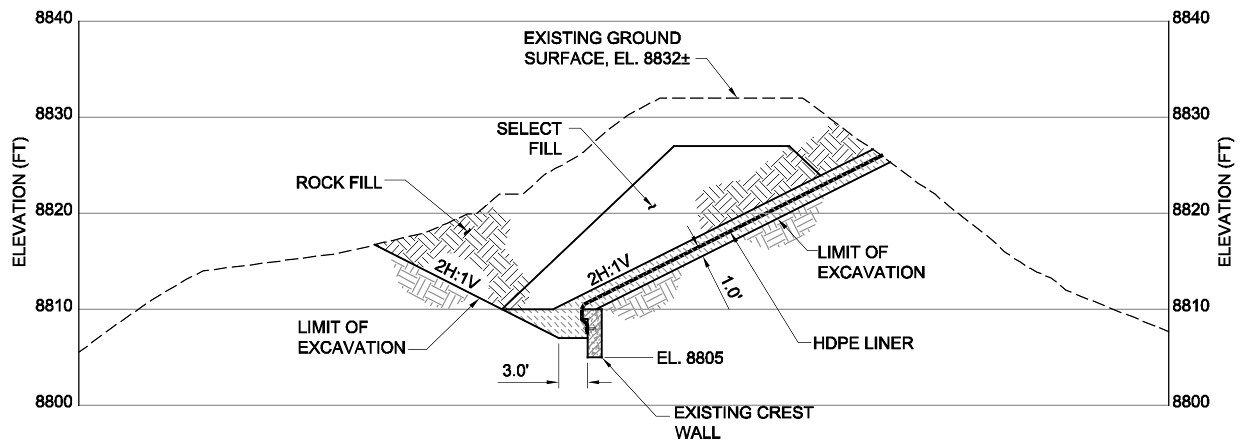
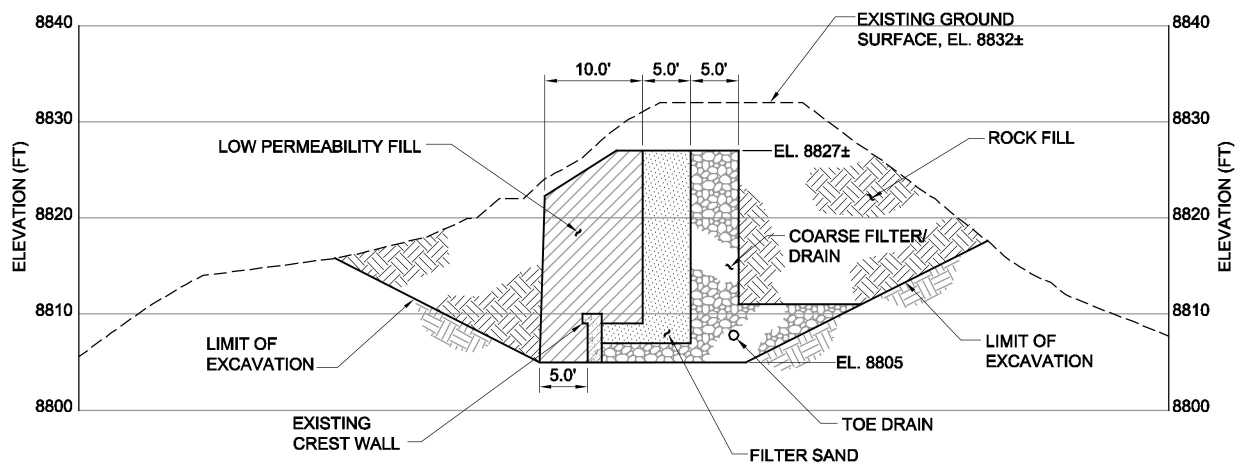


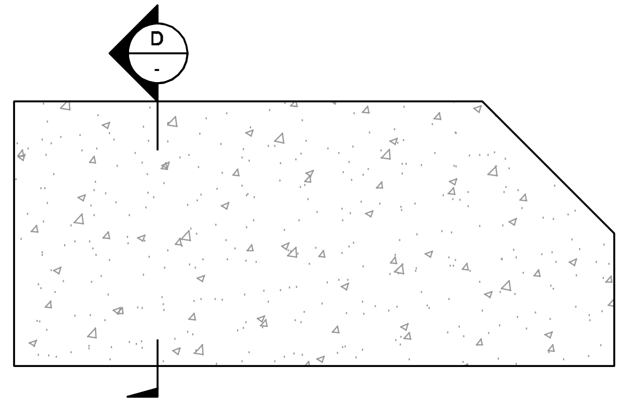
ELEMENT NO. 4A HDPE LINER SECTION (B / 5-12)

0 5 10 15
SCALE IN FEET



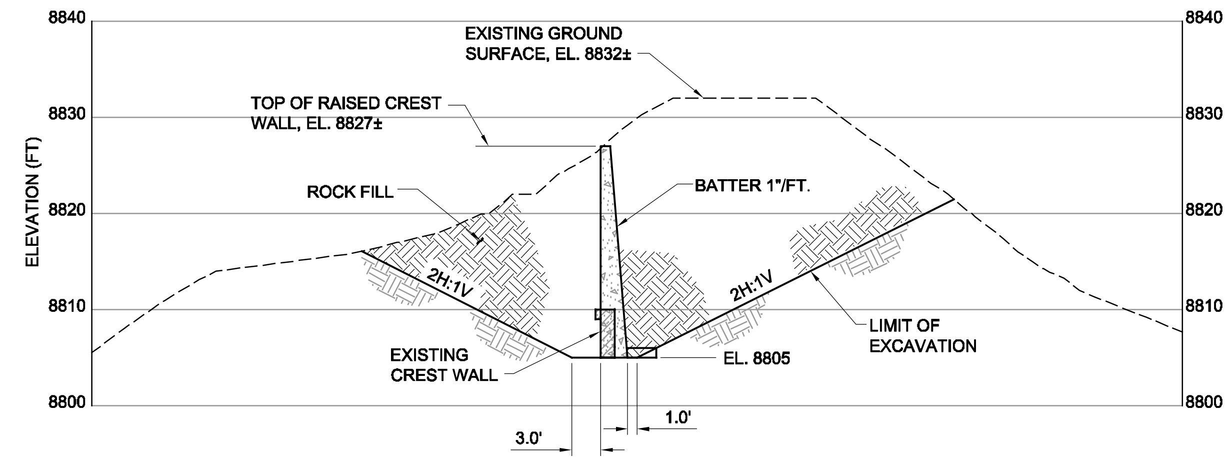
ELEMENT 4B LOW PERMEABILITY FILL SECTION (B / 5-12)

0 5 10 15
SCALE IN FEET



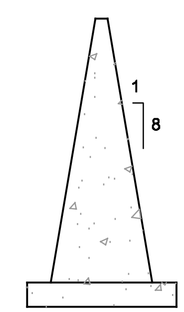
TRANSITION BETWEEN THE DAM CREST AND THE LEFT ABUTMENT DIKE CREST (C / 5-12)

0 8 16 24
SCALE IN FEET



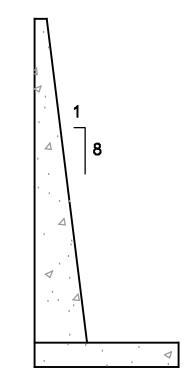
ALTERNATIVE 4C RAISED CREST WALL SECTION (B / 5-12)

0 5 10 15
SCALE IN FEET



TRANSITION BETWEEN THE DAM CREST AND THE LEFT ABUTMENT DIKE CREST (D / -)

0 8 16 24
SCALE IN FEET

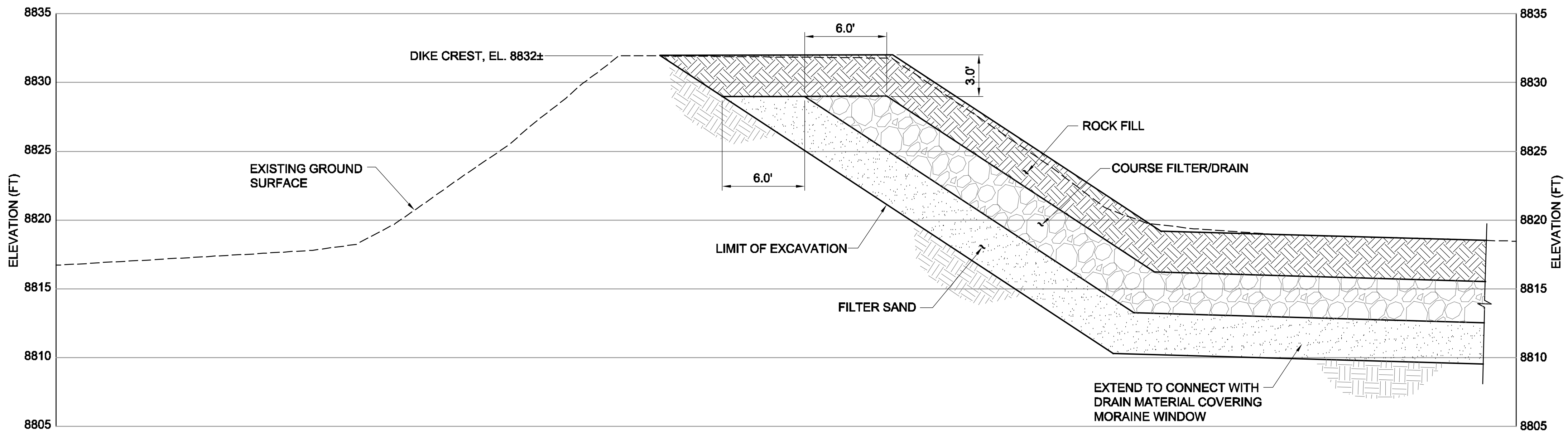


PRIMARY SPILLWAY MSE WALL REPLACEMENT SECTION (E / 5-12)

0 8 16 24
SCALE IN FEET

Job No. :	22242295	BEAVER PARK DAM ELEMENT 4 DAM CREST MODIFICATIONS SECTIONS AND DETAILS
Prepared By :	GGG	
Date :	2/20/12	

FIG. 5-13



DIKE CREST FILTER/DRAIN SECTION F/5-1 F/5-12

0 5 10 15
SCALE IN FEET

Job No. :	22242295
Prepared By :	GGG
Date :	2/20/12

BEAVER PARK DAM
LEFT ABUTMENT CREST DIKE
FILTER / DRAIN SYSTEM
SECTION

FIG. 5-14

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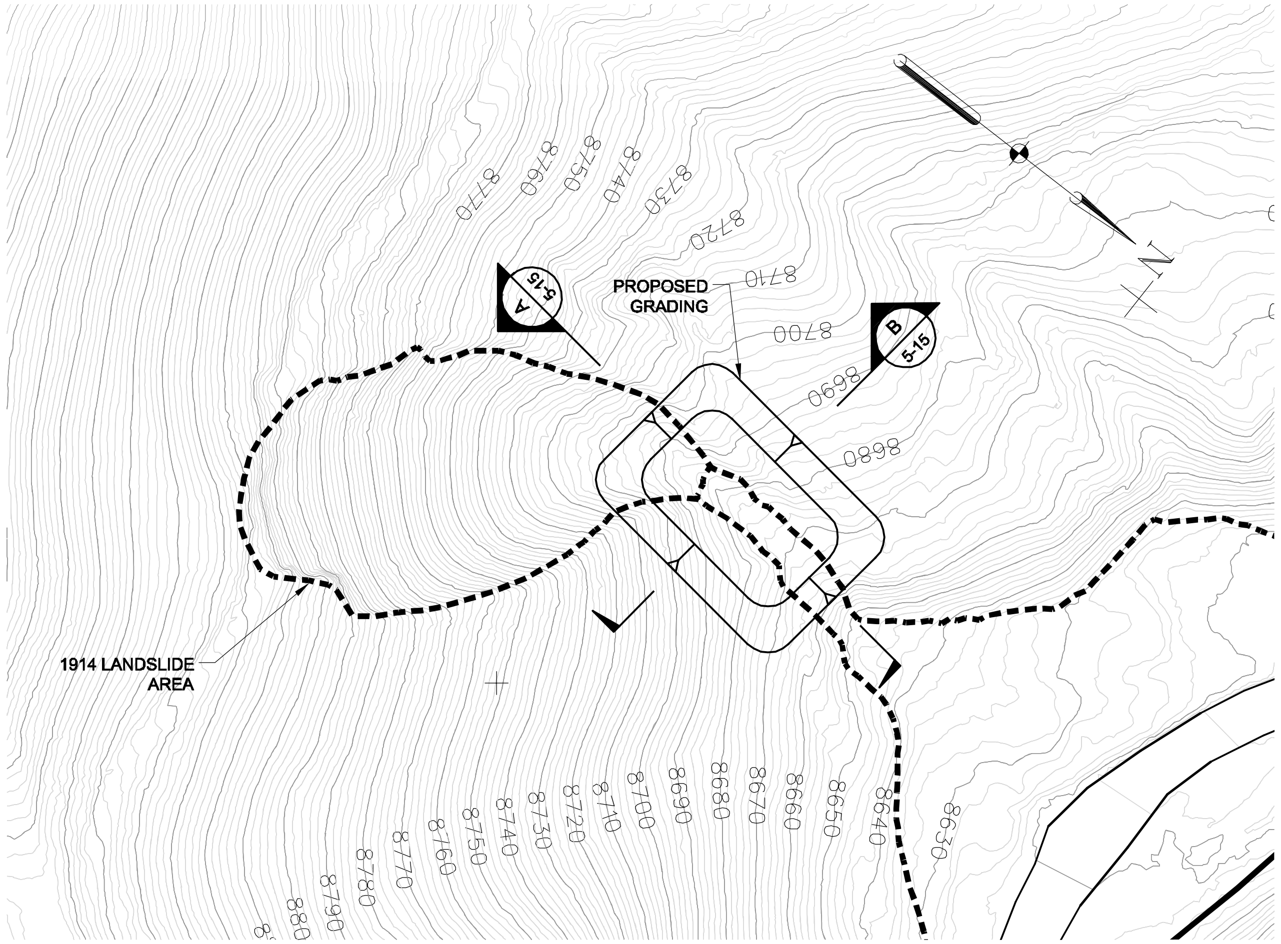
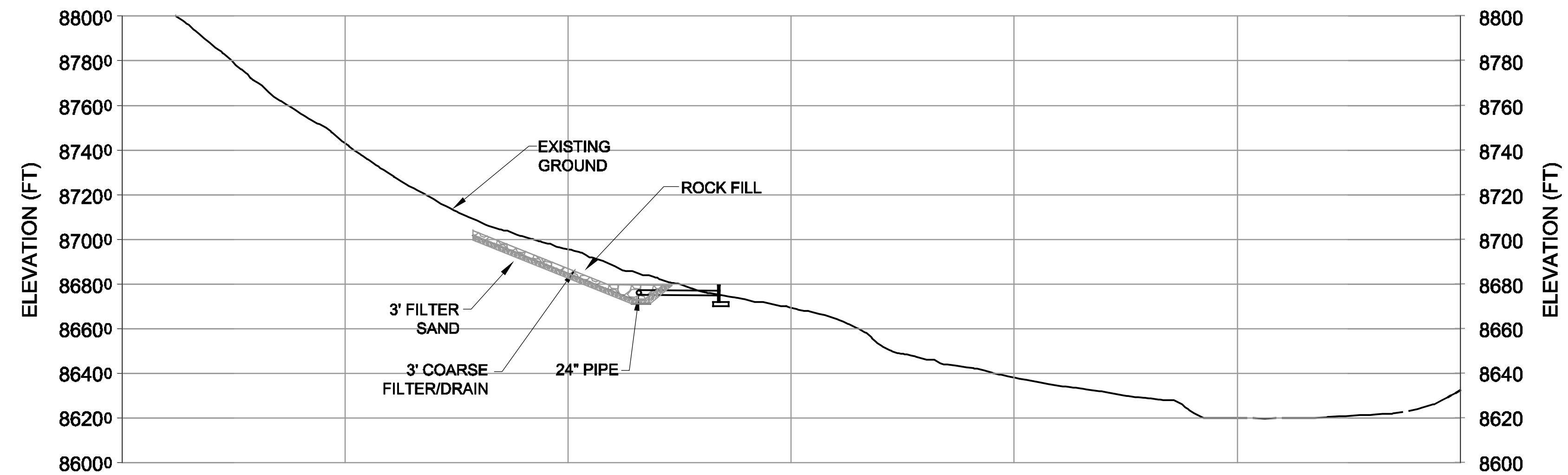


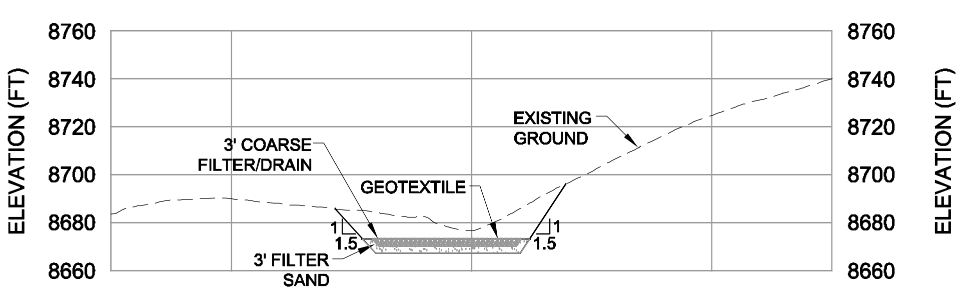
PLAN

0 50 100 150
SCALE IN FEET



TYPICAL SECTION A 5-15

0 40 80 120
SCALE IN FEET



TYPICAL SECTION B 5-15

0 40 80 120
SCALE IN FEET

NOTES:

1) FILTER SAND AND COARSE FILTER/DRAIN ASSUMED FOR COST ESTIMATED PURPOSES EXTENTS AND THICKNESS OF FILTER AND COARSE FILTER/DRAIN TO BE DETERMINED IN THE FIELD
2) SEE FIGURE 4-1 FOR RELATIVE LOCATION

Job No. : 22242295	ELEMENT 5 1914 LANDSLIDE AREA FILTER AND DRAIN SYSTEM
Prepared By : PPP	
Date : 8/11/11	

FIG. 5-15

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KOWALKE, PAUL H. 2/24/2012 10:07 AM

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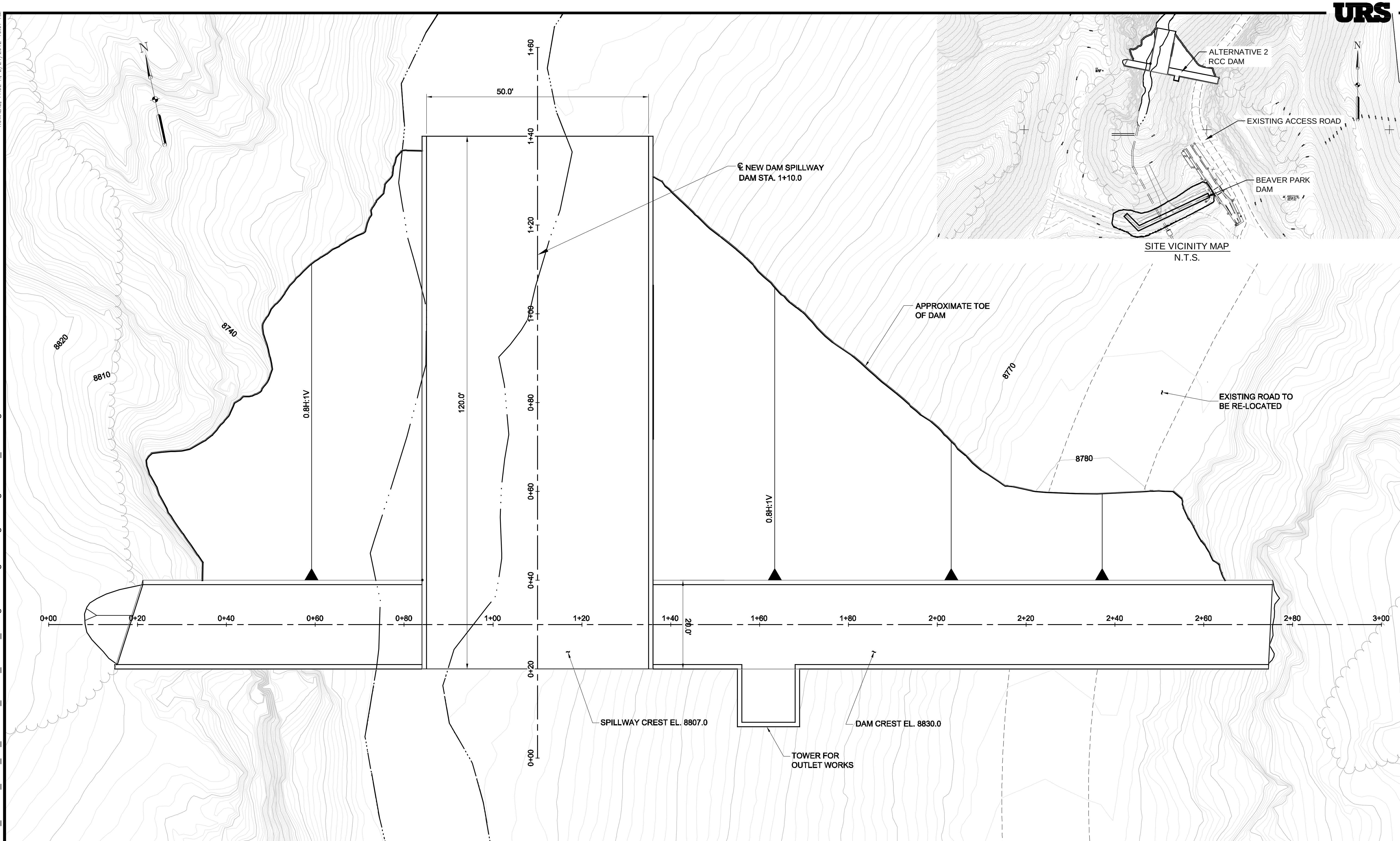
URS



PLAN

0 10 20 40

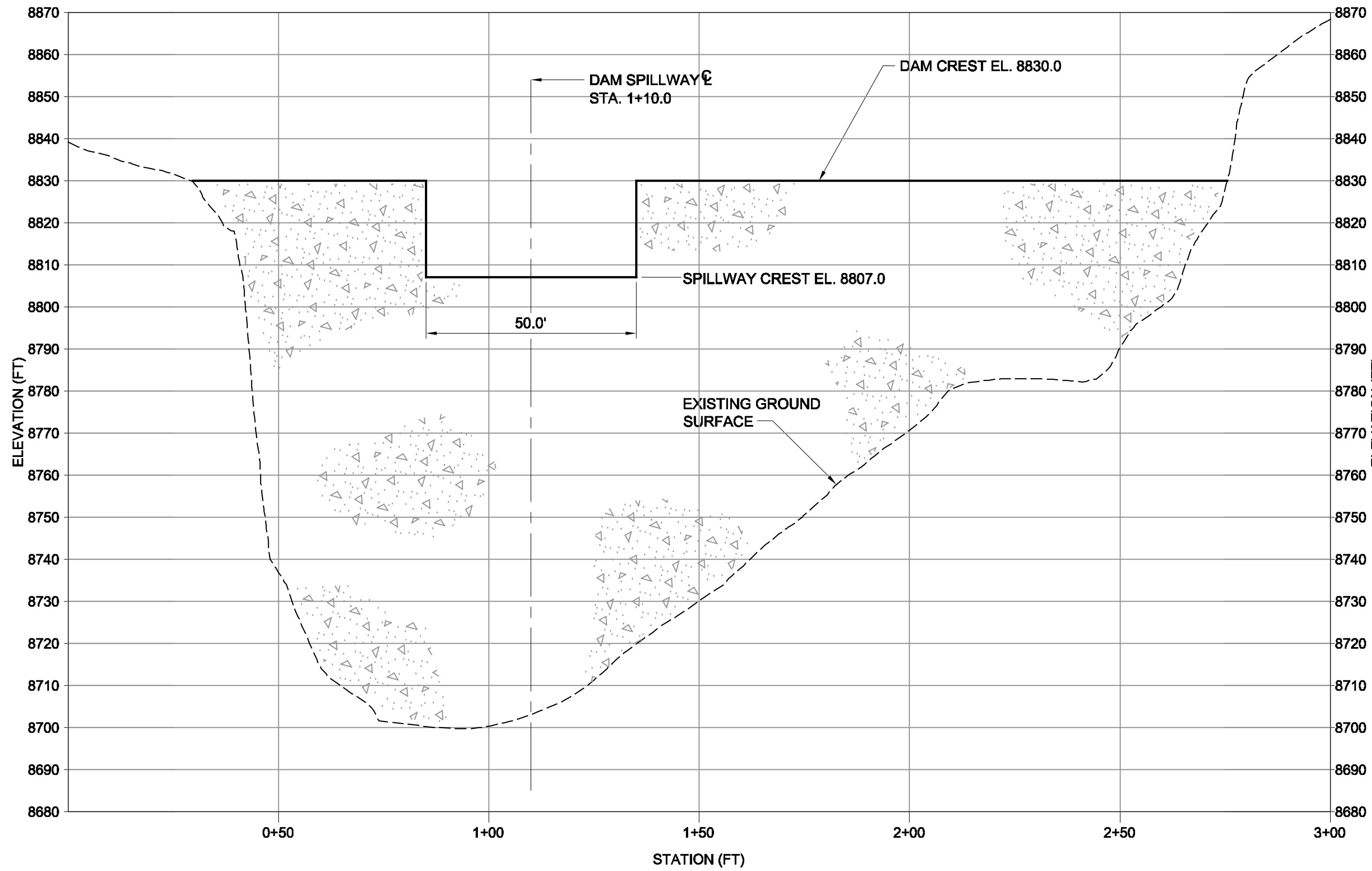
SCALE IN FEET

Job No. :	22242295
Prepared By :	GGG
Date :	2/22/12

BEAVER PARK DAM
ALTERNATIVE 2
RCC DAM PLAN

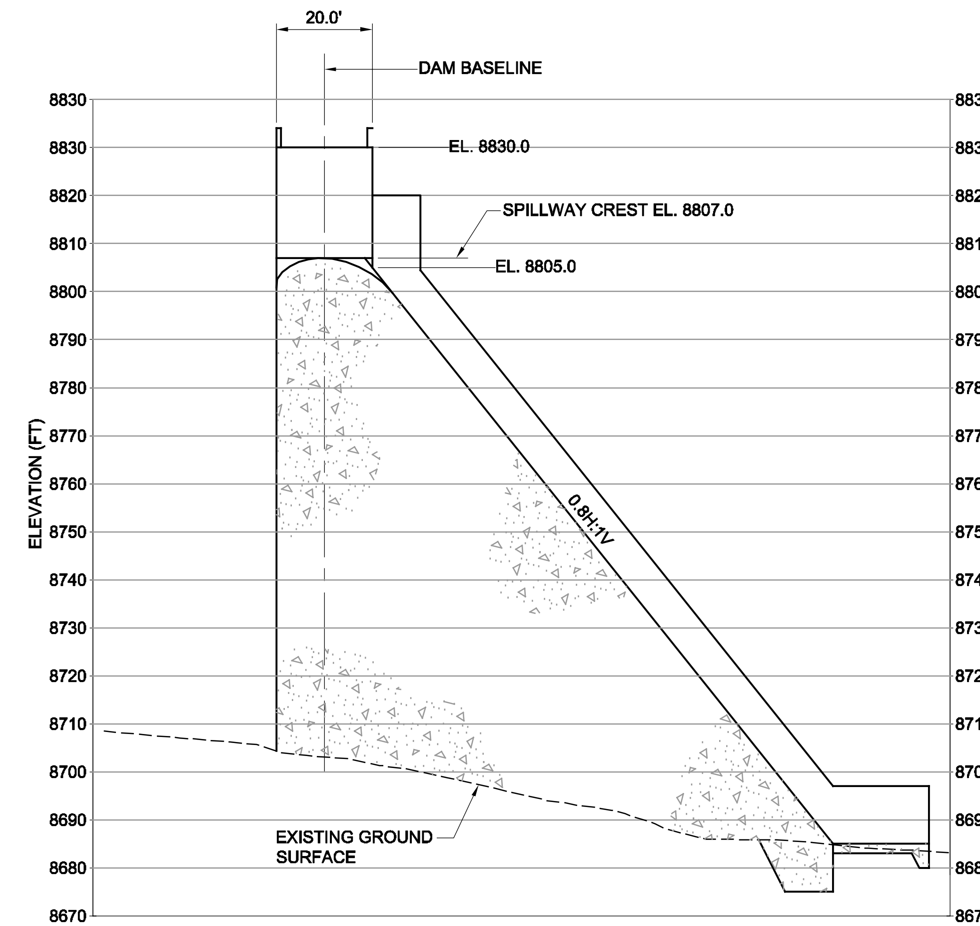
FIG. 5-16

URS



DAM ℄ SECTION

0 20 40 60
SCALE IN FEET



DAM SPILLWAY SECTION

0 20 40 60
SCALE IN FEET

Job No. :	22242295
Prepared By :	GGG
Date :	2/22/12

BEAVER PARK DAM
ALTERNATIVE 2
RCC DAM PROFILE AND SECTION

FIG. 5-17

6.1 GENERAL

The proposed alternatives are for the purpose of providing a facility with improved safety and reliability. Existing safety concerns addressed by the alternatives include the following:

- The potential for piping failure in the left abutment moraine area that was highlighted by the 2010 sinkhole.
- The potential for failure as the result of seepage through the dam or dike crest during normal pool operations and flood surcharge.
- The reliability of the outlet works operation that was highlighted by the lining failure in 2009.
- Erosion in the primary spillway discharge area that could threaten the emergency spillway structure and is also a concern of the SEO.

These concerns have resulted in reduced reservoir storage and use of the outlet works to pass inflows to limit the use of the spillway. Detailed review of the records of past performance and modifications identified additional concerns about seepage through the crest of the dam and the crest of the dike that may result in dam failure during flood surcharge storage and if the normal maximum pool level is raised 3 feet, as desired by CPW.

URS performed a risk study in 2010. This study was limited in scope in that it only considered failure modes associated with operation at normal full reservoir storage. The objectives of the study were to examine the need for limiting the reservoir storage and to establish a reasonable limitation on storage. All of the identified PFMs that were estimated to be significant contributors to risk were those that involved internal erosion through the moraine to the left of the rock knob. The largest contributor was piping through the left abutment moraine exiting through the bedrock window immediately downstream of the left abutment of the dam, designated as PFM 1. Two other failure modes were judged to be significant. One was piping along a longer path that exits in the valley wall in the area of the 1914 landslide, designated as PFM 2, and the other was piping of embankment materials through the left abutment rock knob, designated as PFM 3.

There have been no detailed dam breach and inundation studies that would allow an estimate of the loss-of-life consequences. A rough best estimate of 25 lives lost as the result of a dam failure was used in the 2010 risk study. The discussions in this report are focused on the estimated annual probability of dam failure (as a measure of risk) with the proposed alternatives modifications since there has been no refinement of the loss-of-life consequences estimate.

The risk assessment discussions for this report include several items that were not a part of the previous risk assessment. The following discussions include the effects the flood surcharge loadings and associated spillway discharge and the desired 3 feet of added storage. Seismic loading is not addressed but the characteristics of the dam and the moraine deposit along with the more than 25 feet of freeboard for normal operating pools suggest that the potential for seismic failure would not pose significant risk for this facility. There is some possibility that the current system for gate operations could be damaged by seismic loading but the proposed rehabilitation should eliminate that concern.

The risk assessment presented in this section is based on the assumption that the additional 3 feet of storage has been added.

6.2 RISKS ASSOCIATED WITH FAILURE THROUGH THE LEFT ABUTMENT AREA

The risks associated with piping through the left abutment presented in the 2010 risk study were dominated by PFM 1. Table 6-1 presents the probabilities associated with each node on the event tree for the 2010 estimates for the unmodified dam along with the estimates that have been revised to reflect the estimated effects of the of Alternative 1A - Left Abutment Downstream Filter and Drain System and Alternative 1B - Left Abutment Cutoff Wall on those probabilities.

Table 6-1 - Estimated Annual Probabilities of Failure for PFM 1

Node	***2010 Estimate***	***Filter and Drain (Alternative 1A)***	***Cutoff Wall (Alternative 1B)***
Reservoir Rises	1.0	1.0	1.0
Initiation - Erosion Starts (Considers re-initiation)	0.05	0.05	***0.03***
Continuation - Unfiltered Exit	0.9	***0.005***[(1)(2)]	0.9
Progression - Roof Forms	0.5	0.5	0.5
Progression - Upstream Zone Fails to Fill Pipe	0.99	0.99	0.99
Progression - Constriction or Upstream Zone Fails to Limit Flows	0.99	0.99	***0.005***
Intervention fails	0.7	***0.5***	0.7
Dam Breaches	0.99	0.99	***0.5***
Annual Probability of Failure	1.5×10^{-2}	***6.0×10^{-5}***	***2.4×10^{-5}***

(1) Changes in the probability estimates are in bold and italicized
(2) 0.005 is the best estimate from a range of 0.01 to 0.001

Alternative 1A is expected to slightly increase the probability of intervention as a result of the ability to monitor the seepage flows from the filter and drain system. The major effect of the filter and drain is to provide a low probability of an unfiltered seepage exit. The estimated annual probability of failure for this failure mode with the abutment filter and drain in place is ***6.0 X 10^{-5}***.

Alternative 1B is expected to slightly reduce the probability of initiation as the result of potentially reduced exit gradients. It is also expected to moderately reduce the likelihood of breach since it might constrain any pipe to such a long path that the reservoir would drain without forming a full breach. The major effect of this modification is seen in the much lower probability of progression as the result of flow being limited by a well-constructed wall tied into bedrock and having no significant windows. The estimated annual probability of failure for this failure mode following the construction of the cutoff wall is ***2.4 X 10^{-5}***.

The 2010 risk study estimated probability of failure for piping through the moraine in the area of the 1914 landslide, PFM 2, was based on an assumed 1,500-foot distance between the reservoir and the seepage exit point. The recent mapping of the site has shown this distance to be approximately 800 feet. The proposed modification that addresses PFM 2 is the weighted filter and drain to be placed in the lower area of the landslide, where seepage is exiting, which will provide filtering of known seepage exits related to the landslide and provide the ability to monitor trends in the seepage quantity relative to reservoir operations. The 2010 risk study estimate of annual probability of failure for this failure mode of ***4.0 X 10^{-6}*** is still considered to be valid. It is possible that the cutoff wall alternative would make this failure mode more likely by reducing seepage near the dam and directing more seepage to the landslide area. It should be noted that there is considerable uncertainty about the significance of the 1914 landslide relative to the potential for a piping failure through the moraine. There is also uncertainty about the positive effects of the drainage tunnel, puddling in the reservoir, and fill placed on the upstream face of the moraine. There have been no reports of sliding following the 1914 event which indicates that the remedial measures were effective.

The probability of failure for piping through the left abutment rock knob should be reduced somewhat by the proposed concrete block and associated dental concrete proposed for the area between the dam and dike crest. However, the 2010 risk study estimate of annual probability of failure of ***2.5 X 10^{-7}*** is still judged to be reasonable.

The effect of the proposed increased storage and the associated 3 feet of head on the estimated annual probabilities of failure in the left abutment area is judged to be minimal. Logically the probabilities would increase but the additional gradient is small and would not significantly influence the estimates, considering the inherent imprecision of the process.

The effect of the hydrologic (flood surcharge) loading on the estimated annual probabilities of failure is also judged to be numerically insignificant because the significant flood surcharges are infrequent and of short duration.

6.3 RISKS ASSOCIATED WITH SEEPAGE THROUGH THE DAM AND DIKE CRESTS OR OVERTOPPING

Annual probabilities of failure as the result of seepage through the dam or dike crest or overtopping were not estimated during the 2010 risk analysis. For normal operations, these failure modes were not judged to be a source of significant risk, as discussed in the 2010 risk analysis report. There is concern about potential failure resulting from seepage through the crest during flood surcharge based on uncertainties about the existing conditions. There are uncertainties about the elevation and condition of the top of the concrete face/spillway closure wall in the dam and the materials that were used to raise the dam and dike in the 1949-52 time period and in 1988. There is no indication that filters or zones that would provide filtering were used in those raises. The crest modification alternatives, which provide a positive seepage barrier and a filter/drain system in the dam crest and an engineered filter and drain in the dike crest, address the uncertainties. These alternatives are considered prudent whether or not the 3 feet of additional storage is included in the modifications, based on the uncertainties.

Elements 4A through 4C were developed to protect the dam and dike crests for reservoir levels for floods up to the PMF magnitude. Flood routings indicate that the current crest levels are approximately 4 feet above the PMF level. The dam seepage barrier and the dike crest filter and drain extend to the elevation of the PMF flood surcharge. Since there is freeboard above the PMF level and failure caused by seepage would not be certain even if the reservoir level exceeded the elevation of the seepage barrier or the filter and drain, the annual probability of failure for crest-related failure modes is estimated to be at least one order of magnitude less likely than the frequency of the PMF. On the assumption that the PMF has a recurrence interval of no less than 10,000 years, the annual probability of failure for crest failure is estimated to be less than ***1.0 X*** $\boldsymbol{10^{-5}}$.

6.4 RISKS ASSOCIATED WITH PRIMARY SPILLWAY EROSION

Two potential failure modes associated with primary spillway erosion are envisioned. One is backward erosion that eventually undermines the spillway structure and the tie back wall resulting in failure and breach of the right dam abutment. The other potential failure mode is erosion of the toe of the embankment that progresses upstream until the downstream shell of the embankment can no longer support the water barrier in the dam. The conditional probability of significant backward erosion of either the rock under the spillway structure or the embankment shell is dependent on the magnitude and the duration of the spillway discharge.

Based on the hydrology presented in Section 3, it is judged that the duration and discharge for floods less than the 100-year flood would make it virtually impossible that significant erosion would occur. For these floods, given significant erosion, it is considered inconceivable that the erosion would progress to a point that would result in a breach of the dam. For floods greater than the 100-year flood and less than the 1,000-year flood, it is considered unlikely that significant erosion would occur and, given significant erosion, virtually impossible that the erosion would result in dam breach. For floods greater than the 1,000-year flood including the PMF, it is considered likely that significant erosion would occur and, given significant erosion, very unlikely that the extent of that erosion would result in a dam breach.

Based on this assessment the annual probability of dam failure as a result of operation of the primary spillway with Element 3 in place would be about ***1.0 X 10***$^{-5}$.

6.5 RISKS ASSOCIATED WITH THE OUTLET WORKS

No significant potential dam failure modes have been identified for the outlet works. It is emphasized that the estimates of failure probability for the left abutment seepage failure modes assume the reliable operation of the outlet works. The capacity of the outlet provides for significant drawdown capability that would provide the possibility of intervention should evidence of the development of a failure mode be observed.

6.6 SUMMARY

Based on the above discussion, the total annual probability of failure for all failure modes that were identified as significant contributors to risk is less than ***1.0 X 10***$^{-4}$.

The piping failure modes through the left abutment moraine would remain as the most significant risk contributors following the construction of the recommended modifications. There are a number of uncertainties associated with the subsurface conditions in this area. These uncertainties include the nature of the moraine and the disturbance that has occurred as a result of past seepage incidents and the construction and operation of the drainage tunnel. These uncertainties call for careful observations during the construction of the remedial measures and careful and frequent monitoring during the initial years of resumed normal reservoir operation.

In addition to the seepage measurements described for Alternative 1A, it would be prudent to install 2 or 3 piezometers in the moraine between the dike and the 1914 landslide area.

7.1 GENERAL

URS prepared an opinion of probable project cost and a construction schedule was estimated for each element and alternative discussed in Section 5. The opinion of probable project cost includes estimates for recognized items that are required for the construction of each element and alternative, design and construction engineering, and permitting. An Unlisted Items line item was also included to account for cost items that have not yet been identified at this limited level of project definition. Also included in the cost is allowance for mobilization/demobilization, and preparatory work. A 30 percent contingency was also included in the cost to account for: a) additional changes in conditions as a result of additional investigations b) changes /additions to the project as the project develops further. The preliminary construction schedule for each element was estimated based on URS' experiences on other similar projects.

The opinions of probable cost and construction schedules for each element are discussed below. A detailed breakdown of each opinion of probable cost is located in Appendix D.

7.2 DAM REHABILITATION ELEMENTS

7.2.1 Element 1A - Left Abutment Downstream Filter and Drain System

The opinion of probable project cost for this element is $6,392,100. This figure includes excavation, filter/drain system placement and fill placement on top of the system. To minimize excavation along a portion of the moraine slope, a sheetpile and tieback system was also included in the cost. Unlisted items for this element include, but are not limited to the following:

1) Possible measures to address the actual condition of the bedrock and moraine materials, which is not fully defined. The bedrock and moraine materials may contain voids or open work sands and gravels of unknown area and depth that would have to be treated on a case by case basis with the placement of drain material to fill the voids prior to placement of the filter and drain system.
2) Concrete headwall and measuring device at the outfall of the drain pipe.
3) Provision for backfilling any existing sinkholes that may be exposed during excavation of the moraine materials.
4) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
5) Stormwater management and site reclamation.

A construction schedule to complete this element was estimated to be about 8 to 12 months.

7.2.2 Element 1B - Left Abutment Cutoff Wall

The opinion of probable project cost for this element is $33,082,400. This figure includes excavation of a working platform necessary to construct the cutoff wall along the proposed alignment. The cost for this alternative is dependent on the construction method and unit rate assumed for the cutoff wall. The depth and type of material that the wall needs to be constructed through make it specialized construction. Our experience on recent and current similar projects indicates that the unit rate could vary between $200 and $400 per square foot of wall. For the purposes of this estimate, we have assumed a mid-range rate of $300 per square foot.

Work items for this alternative, which have not been identified in the estimated quantities because of the lack of detail in the current designs and which will be covered by the allowance for unlisted items include but are not limited to the following:

1) Construction methods and working platform width to effectively construct a cutoff wall at this depth vary from secant piles to panel walls. Each method has field considerations that would need to be engineered and constructed.
2) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
3) Stormwater management and site reclamation.

A construction schedule to complete this alternative was estimated to be about 12 to 18 months.

7.2.3 Element 2 Outlet Works Conduit Rehabilitation and Gate Reconfiguration

The opinion of probable project cost for this element is $3,892,200. This figure includes construction costs associated with slip lining of the existing upstream and downstream conduits, a downstream control structure with a 36-inch knife gate, upstream inlet structure with a hydraulically operated 60-inch by 60-inch sluice gate, abandonment of the existing gate vault, and an upstream diversion system.to control incoming stream flows.

Unlisted items for this element include, but are not limited to the following:

1) Valve instrumentation and controls.
2) Surface preparation and cleaning of the existing tunnel.
3) Abandonment or removal of the existing gates.
4) Miscellaneous metals (doors, roof hatches, handrails, guard rails, fencing, etc.)
5) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
6) Stormwater management and site reclamation.

A construction schedule to complete this element was estimated to be about 8 to 12 months.

7.2.4 Element 3 - Primary Spillway Concrete Chute

The opinion of probable project cost for this element is $1,704,300. This figure includes excavation, construction of the spillway chute, and a grouted riprap channel and plunge pool.

Unlisted items for this element include, but not limited to the following:

1) Anchors that may be needed for the stepped concrete chute.
2) Removal of about 25 feet of the existing primary spillway concrete chute.
3) Miscellaneous metals (handrails, guard rails, fencing, etc.)
4) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
5) Stormwater management and site reclamation.

A construction schedule to complete this element was estimated to be about 8 to 12 months.

7.2.5 Element 4A - Upstream High Density Polyethylene Liner

The opinion of probable project cost for this element is $1,237,600. This figure includes construction costs associated with excavation, select fill, geotextile and HDPE installation, and rock fill.

Unlisted items for this element include, but are not limited to the following:

1) Provisions for anchoring the HDPE liner to the concrete wall and transition section.
2) Foundation treatment or repairs to the existing concrete parapet wall.
3) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
4) Stormwater management and site reclamation.

A construction schedule to complete this element was estimated to be about 4 to 6 months.

7.2.6 Element 4B - Low Permeability Core

The opinion of probable cost for this element is $1,280,500. This figure includes construction costs associated with excavation, select fill, low permeability core, filter and drain, and rock fill.

Unlisted items for this element include, but are not limited to the following:

1) Foundation treatment or repairs to the existing concrete parapet wall.
2) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
3) Stormwater management and site reclamation.

A construction schedule to complete this element was estimated to be about 4 to 6 months.

7.2.7 Element 4C - Raised Crest Wall

The opinion of probable cost for this element is $1,857,700. This figure includes construction costs associated excavation, select fill, steel reinforced raised wall section, rock fill.

Unlisted items for this element include, but are not limited to the following:

1) Foundation treatment or repairs to the existing concrete parapet wall.
2) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
3) Stormwater management and site reclamation.

A construction schedule to complete this element was estimated to be about 6 to 8 months.

7.2.8 Element 5 - 1914 Landslide Area

The opinion of probable project cost for this element is $725,000. This figure includes excavation, filter, drain system placement and fill placement on top of the system. Unlisted items for this element include, but are not limited to the following:

1) The extent that foundation treatment that may be required prior to placing the filter material.
2) Seepage collection structure(s) and measuring device(s).
3) Provisions for personnel to access and monitor the area visually and to measure flows.
4) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
5) Stormwater management and site reclamation.

A construction schedule to complete this element was not determined as it was assumed that the design element would be completed concurrent with other design elements.

7.3 ALTERNATIVE 1 - DAM REHABILITATION

7.3.1 Alternative 1A - Left Abutment Filter and Drain System

Alternative 1A was developed by selecting the lowest cost option for each of the five elements. This included Element 1A - Left Abutment Downstream Filter and Drain System. The opinion of probable project cost for this alternative is $13,951,000. A summary of the Elements included Alternative 1A and their corresponding cost is shown in Table 7-1. The construction schedule to complete all the elements in Alternative 1A is estimated to be approximately six to eight months. This would likely require two construction seasons due to the reservoir inflow constraints during the spring. Elements 2 and 3 could be constructed during the first construction season with remaining elements being constructed during the second construction season. If the contractor were to experience construction delays, Element 1A could be partially completed during the first construction season.

Table 7-1 - Alternative 1A Cost Summary

Element	*Element Description*	*Opinion of Probable Cost*
1A	Left Abutment Downstream Filter and Drain System	$ 6,392,100
2	Outlet Works Rehabilitation and Gate Reconfiguration	$ 3,892,000
3	Primary Spillway Concrete Chute	$ 1,704,300
4A	Crest Modifications (HDPE Liner)	$ 1,237,600
5	1914 Landslide Filter and Drain System	$ 725,000
	Alternative 1A Opinion of Probable Cost	***$ 13,951,000***

7.3.2 Alternative 1B - Left Abutment Cutoff Wall

Alternative 1B is similar to Alternative 1A except that Element 1B – Left Abutment Cutoff Wall was used because it represented the greatest improvement in risk reduction. The opinion of probable project cost for this alternative is $40,641,300. A summary of the elements included in Alternative 1B and their corresponding costs is shown in Table 7-2. The construction schedule for Alternative 1B is estimated to be approximately 10 to 13 months. This would also likely require two construction seasons. Element 1B is the driving element in the schedule. It is independent of the other elements and is not constrained by season or runoff. With regard to the remaining elements, the construction sequencing should be similar to Alternative 1A, with the construction of Elements 2 and 3 being completed in the first construction season followed by the remaining elements constructed in the second season.

Table 7-2 Alternative 1B Cost Summary

Element	*Element Description*	*Opinion of Probable Cost*
1B	Left Abutment Cutoff Wall	$ 33,082,400
2	Outlet Works Rehabilitation and Gate Reconfiguration	$ 3,892,000
3	Primary Spillway Concrete Chute	$ 1,704,300
4A	Crest Modifications (HDPE Liner)	$ 1,237,600
5	1914 Landslide Filter and Drain System	$ 725,000
	Alternative 1B Opinion of Probable Cost	***$ 40,641,300***

7.4 ALTERNATIVE 2 - NEW RCC DAM

The opinion of probable project cost for this alternative is $45,120,600. The general summary of the costs is shown in Table 7-3.

Table 7-3 Alternative 2 Cost Summary

Description	***Opinion of Probable Cost***
RCC Dam	$ 19,052,000
Outlet Works	$ 1,540,350
Left Abutment Seepage Control Measures Allowance	$ 22,750,000
County Road 20 Relocation Allowance	$ 1,778,250
Alternative 2 Opinion of Probable Cost	***$ 45,120,600***

Unlisted items for this alternative include, but are not limited to the following:

1) Treating the area of the drainage tunnel to prevent reservoir head charging the moraine.
2) Dam instrumentation, gate and valve instrumentation and controls.
3) Miscellaneous metals (doors, roof hatches, handrails, guard rails, fencing, etc.)
4) Contractor obtained permits and compliance (stormwater, dewatering, etc.).
5) Stormwater management and site reclamation.

A construction schedule to complete this alternative was estimated to be about 15 to 21 months.

7.5 LIMITATIONS

The opinions of probable costs presented in this Section are based on information developed for the design and our knowledge of market conditions at the time of preparation of the opinions. Construction cost has been estimated with the use of a combination of historical unit pricing and detailed unit pricing, depending on the availability of information. The logic, methods and procedures for developing costs, is believed to be typical for the construction industry.

Accuracy is not guaranteed and the use of unit pricing should not be deemed as an offering or proposal with respect to the outcome of the cost of an activity or project. Unit price opinions are subject to change with proper notice. Any estimate of unit prices is not intended to predict the outcome of hard dollar results from open and competitive bidding.

The estimates shown, and any resulting conclusions on project financial or economic feasibility or funding requirements, have been prepared for guidance in project evaluation and implementation from the information available at the time of the estimate.

The final costs of the project and resulting feasibility will depend on actual labor and material costs, competitive market conditions, actual site conditions, final project scope, implementation schedule, continuity of personnel and engineering, and other variable factors. As a result, the final project costs may vary from the estimates presented herein.

Because of these factors, project feasibility, benefit/cost ratios, risks, and funding needs must be carefully reviewed prior to making specific financial decisions or establishing project budgets to help ensure proper project evaluation and adequate funding.

8.1 ALTERNATIVE SELECTION

An Alternative Selection Workshop was held at CPW on December 9, 2011. The elements and the alternatives were discussed during the workshop and the reasons behind determining the selected alternative are discussed below.

8.1.1 Alternatives Not Selected

Alternative 1B - Left Abutment Cutoff Wall and Alternative 2 - New Roller Compacted Concrete Dam were not selected. The primary factor that contributed to ruling out the above alternatives is that the opinions of probable cost for these alternatives were each 3 times greater than that for Alternative 1A, as discussed in Section 7 of this report.

8.1.2 Selected Alternative

Alternative 1A was judged to be preferred. This alternative includes the following elements:

- Element 1A - Left Abutment Downstream Filter and Drain System
- Element 2 - Outlet Works Rehabilitation and Gate Reconfiguration
- Element 3 - Primary Spillway Concrete Chute
- Element 4A - Crest Modifications - HDPE Liner
- Element 5 - 1914 Landslide Filter and Drain System

This alternative satisfactorily addresses all of CPW's objectives and the estimated risk reduction provided by this alternative is only slightly less than that for the much more expensive Alternative 1B.

8.2 CONCEPTUTAL CONSTRUCTION SCHEDULE

The conceptual project and construction schedule was developed in Microsoft Project. The estimated project and construction schedule reflects milestones, major construction activities sequencing, and critical path analysis. The estimated construction schedule is organized by the design elements discussed in this design report have been identified as the following construction milestones:

- Element 1A - Left Abutment Down Stream Filter Drain System
- Element 2 - Outlet Works Conduit
- Element 3 - Preliminary Spillway Concrete Chute
- Element 4 - Upstream High Density Polyethylene Liner
- Element 5 - 1914 Landslide Area Filter and Drain System

The construction milestones have been sequenced and/or constrained by site access, spring runoff, and reservoir drawn down in the first construction season and suspending or completing work by December.

A notice to proceed and construction start date has been predicated on estimates for completion of design and the issuance of construction permits which are assumed to be in place by January or February 2013. The general project schedule is shown in Figure 8-1. A more detailed project schedule is shown in Figure 8-2.

The goal for the project is to secure construction permits in early 2013, obtain Colorado State Engineer approval, receive, evaluate and award a contract by July 2013 and issue notice to proceed by November 2013. Site access and construction would be expected to start following spring run-off in July of 2014. In order to achieve this goal project approval and funding are dependent on continued collaboration among CPW, the San Luis Valley Irrigation District and the Colorado Water Conservation Board to allow rehabilitation of both Rio Grande Reservoir and Beaver Park Reservoir. If project approval, funding and permits are implemented as planned early start construction activities in July 2014 will include:

- Development of site access and management spring run-off
- Excavation, fill, drain and filter and pipe for Element 1A
- Demolition, steel conduit and concrete for Element 2
- Suspend work for winter shut down December 2014

All other construction activities including excavations, fill, filter and drain, pipe, riprap and concrete for Elements 3, 4 and 5 will commence in June 2015 with anticipated completion in October 2015.

It is estimated that the construction activities in 2014 will be generally concurrent for Element 1A and Element 2 with the exception of starting the excavations in Element 1A earlier to provide for installation of the new steel pipe conduit in the outlet works conduit improvements. In 2015 it is estimated that Element 3 and 4 will work concurrently followed by Element 5 which will overlap Element 4.

Several critical activities have been identified for project and construction progress in 2014. These include, but are not necessarily limited to, the approval and issuance of construction permits, spring run-off based on historical data, excavations of the left abutment downstream filter and drain system to allow for demolition and installation of the new steel conduit in the improved outlet works.

Figure 8-1 General Project Schedule

ID	Task Name	Duration	Start	Finish
1	Complete Design for SEO Review	24 wks	Mon 7/2/12	Fri 12/14/12
4				
5	Permit Application and Issue	52 wks	Mon 7/2/12	Fri 6/28/13
2				
3	SEO Review, Adjust, Approval	24 wks	Mon 12/17/12	Fri 5/31/13
6				
7	Complete Bid Documents	5 wks	Mon 6/3/13	Fri 7/5/13
8				
9	Advertise for Bid	0 wks	Fri 7/5/13	Fri 7/5/13
10				
11	Receive, Evaluate, Award Bid	10 wks	Mon 7/8/13	Fri 9/13/13
12				
13	Notice to Proceed	0 wks	Fri 11/1/13	Fri 11/1/13
14				
15	Submittal Review, Approval	8 wks	Fri 11/1/13	Thu 12/26/13
16				
17	Preconstruction Planning and Meeting	8 wks	Fri 11/1/13	Thu 12/26/13
18				
19	Spring Runoff	9 wks	Mon 4/14/14	Fri 6/13/14
20				
21	**Mobilize 2014**	**4 wks**	**Mon 6/16/14**	**Fri 7/11/14**
24				
25	**Left Abutment Downstream Filter Drain System**	**18 wks**	**Mon 7/14/14**	**Fri 11/14/14**
32				
33	Draw Down Reservoir	6 wks	Mon 7/14/14	Fri 8/22/14
34				
35	**Outlet Works Conduit**	**13 wks**	**Mon 8/25/14**	**Fri 11/21/14**
40				
41	Suspend Work for Winter	22 wks	Mon 12/15/14	Fri 5/15/15
42				
43	Mobilize 2015	2 wks	Mon 5/18/15	Fri 5/29/15
44				
45	**Preliminary Spillway Concrete Chute**	**18 wks**	**Mon 6/1/15**	**Fri 10/2/15**
49				
50	**Upstream High Density Polyethylene Liner**	**14 wks**	**Mon 6/1/15**	**Fri 9/4/15**
55				
56	**1914 Landslide Area Filter and Drain System**	**15 wks**	**Mon 7/13/15**	**Fri 10/23/15**

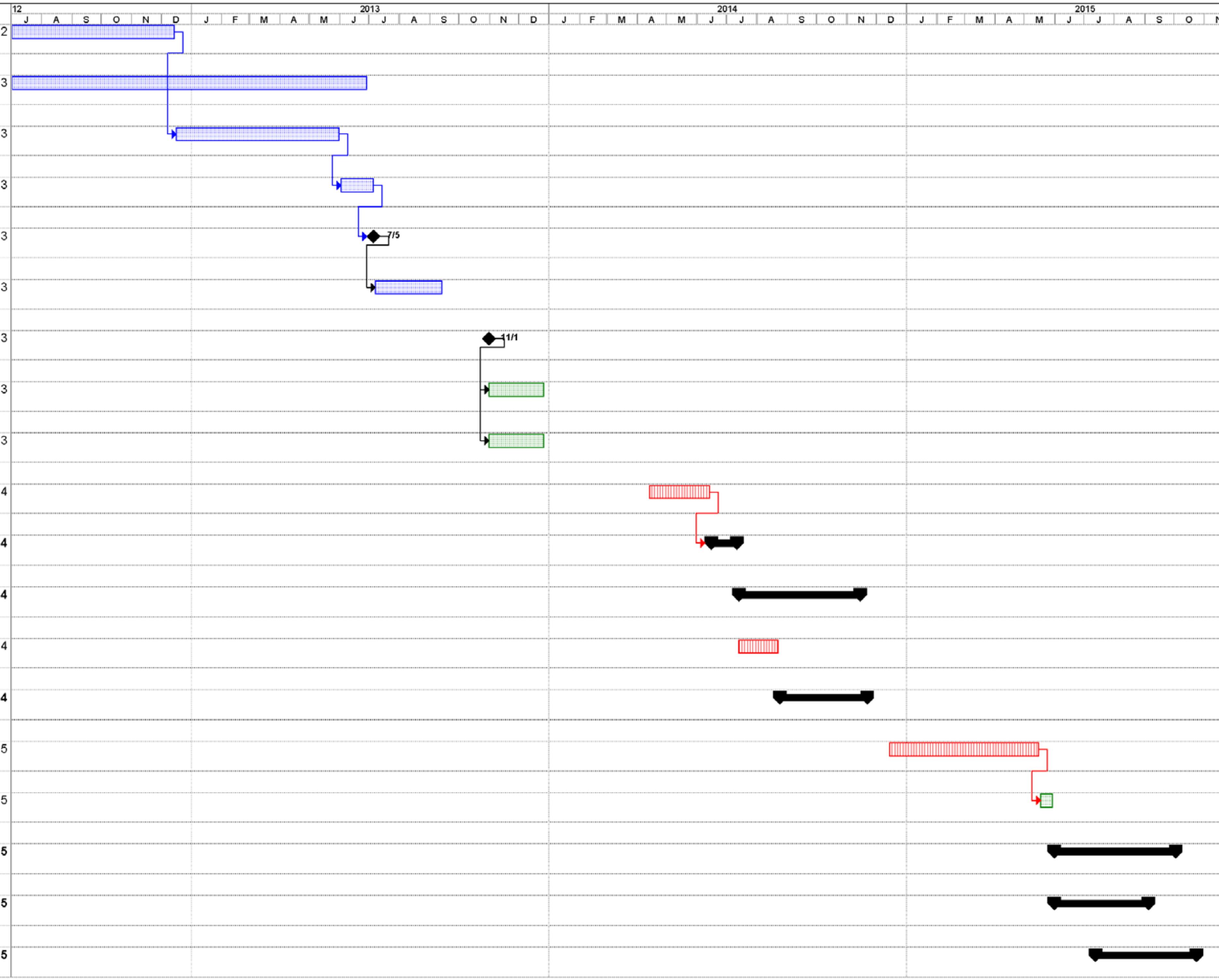
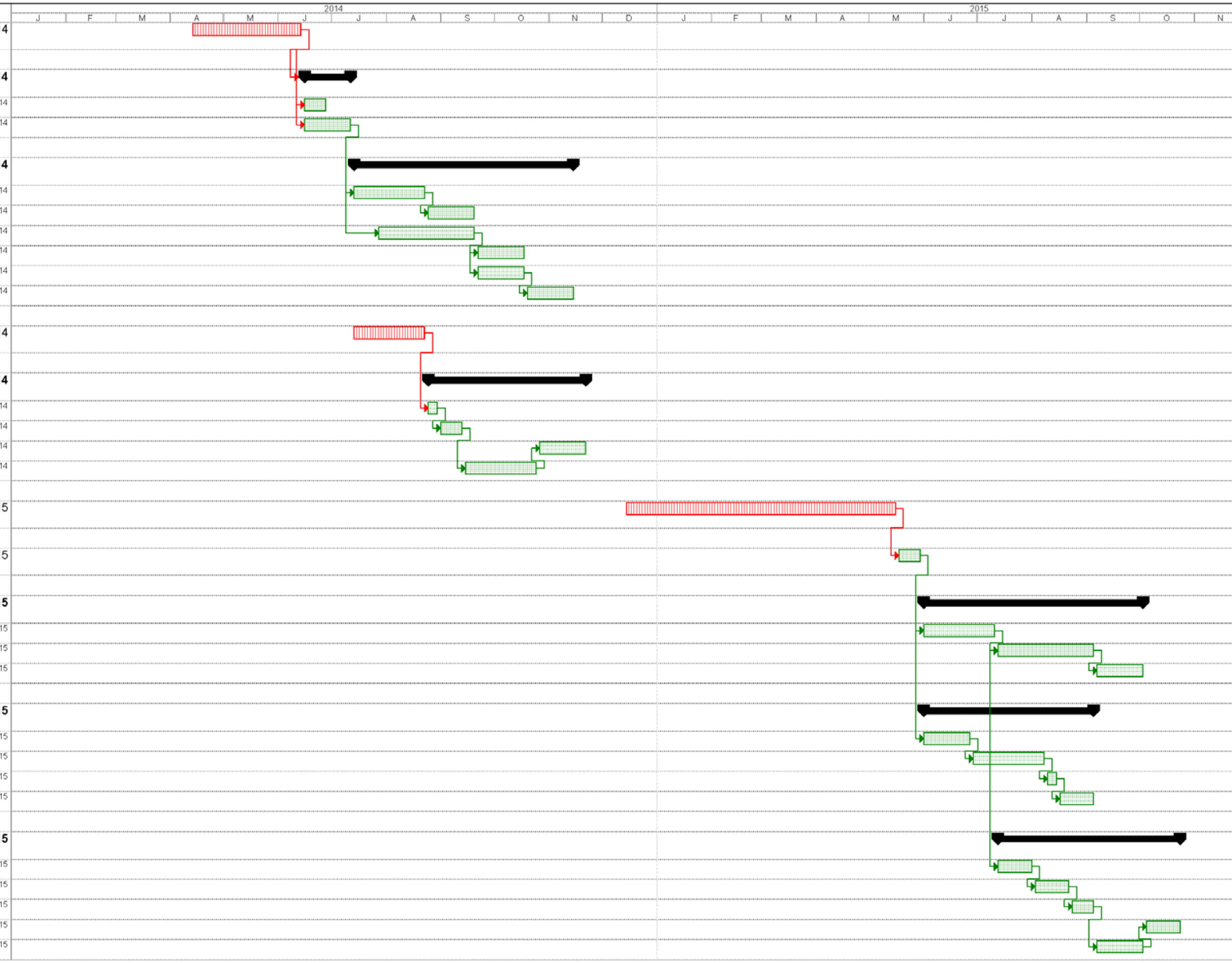


Figure 8-2 Detailed Project Schedule

ID	Task Name	Duration	Start	Finish
1	Spring Runoff	9 wks	Mon 4/14/14	Fri 6/13/14
2				
3	**Mobilize 2014**	**4 wks**	**Mon 6/16/14**	**Fri 7/11/14**
4	Manage Spring Runoff	2 wks	Mon 6/16/14	Fri 6/27/14
5	Develop Site Access	4 wks	Mon 6/16/14	Fri 7/11/14
6				
7	**Left Abutment Downstream Filter Drain System**	**18 wks**	**Mon 7/14/14**	**Fri 11/14/14**
8	Excavation Phase 1	6 wks	Mon 7/14/14	Fri 8/22/14
9	Excavation Phase 2	4 wks	Mon 8/25/14	Fri 9/19/14
10	Filter and Drain	8 wks	Mon 7/28/14	Fri 9/19/14
11	Pipe 24" HDPE	4 wks	Mon 9/22/14	Fri 10/17/14
12	Sheet Piling	4 wks	Mon 9/22/14	Fri 10/17/14
13	Fill	4 wks	Mon 10/20/14	Fri 11/14/14
14				
15	Draw Down Reservoir	6 wks	Mon 7/14/14	Fri 8/22/14
16				
17	**Outlet Works Conduit**	**13 wks**	**Mon 8/25/14**	**Fri 11/21/14**
18	Water Management and Cofferdam	1 wk	Mon 8/25/14	Fri 8/29/14
19	Demolition	2 wks	Mon 9/1/14	Fri 9/12/14
20	Concrete	4 wks	Mon 10/27/14	Fri 11/21/14
21	Steel Conduit and Fittings	6 wks	Mon 9/15/14	Fri 10/24/14
22				
23	Suspend Work for Winter	22 wks	Mon 12/15/14	Fri 5/15/15
24				
25	Mobilize 2015	2 wks	Mon 5/18/15	Fri 5/29/15
26				
27	**Preliminary Spillway Concrete Chute**	**18 wks**	**Mon 6/1/15**	**Fri 10/2/15**
28	Excavation	6 wks	Mon 6/1/15	Fri 7/10/15
29	Concrete	8 wks	Mon 7/13/15	Fri 9/4/15
30	Riprap	4 wks	Mon 9/7/15	Fri 10/2/15
31				
32	**Upstream High Density Polyethylene Liner**	**14 wks**	**Mon 6/1/15**	**Fri 9/4/15**
33	Excavation	4 wks	Mon 6/1/15	Fri 6/26/15
34	Concrete	6 wks	Mon 6/29/15	Fri 8/7/15
35	Liner	1 wk	Mon 8/10/15	Fri 8/14/15
36	Fill	3 wks	Mon 8/17/15	Fri 9/4/15
37				
38	**1914 Landslide Area Filter and Drain System**	**15 wks**	**Mon 7/13/15**	**Fri 10/23/15**
39	Excavation	3 wks	Mon 7/13/15	Fri 7/31/15
40	Filter and Drain	3 wks	Mon 8/3/15	Fri 8/21/15
41	Pipe	2 wks	Mon 8/24/15	Fri 9/4/15
42	Fill	3 wks	Mon 10/5/15	Fri 10/23/15
43	Concrete	4 wks	Mon 9/7/15	Fri 10/2/15



9.1 CONCLUSIONS

Since the Beaver Park reservoir's first filling, there have been incidents involving the rock knob and moraine that form the left abutment of the dam. These incidents have raised concerns about failure as the result of internal erosion. Even after the cribbing of the downstream moraine slope and construction of a drainage tunnel, there were sinkhole incidents in 1938 and again in 2010.

The alternatives developed included consideration of rehabilitation of the existing dam and construction of a new replacement dam downstream of the existing dam. In the development of the dam rehabilitation alternatives different rehabilitation alternative elements (or design components) were required to achieve the different objectives. In fact, for a complete rehabilitation alternative that addresses all five objectives, five different elements are required. The different rehabilitation alternative elements (elements) were developed separately, in part so that CPW could decide whether it wanted to consider rehabilitation to address only some of the elements, for example, rehabilitation that did not include the 3-foot increase in normal maximum pool or the improved reliability of the outlet works. Ultimately, CPW decided in a design review workshop held on September 19, 2011, that the rehabilitation alternative should include all five elements to achieve all five CPW objectives.

The new replacement dam alternative was also developed to achieve all five CPW objectives.

Alternative 1A was judged to be preferred over Alternative 1B, principally because the opinion of probable construction cost for Alternative 1B was estimated to be about 3 times greater than that for Alternative 1A, and the estimated risk reduction provided by Alternative 1B was only slightly greater than that provided by Alternative 1A. The opinions of probable cost associated with each developed alternative are summarized in Table 9-1

Table 9-1 Summary of Developed Alternatives

Alternative	***Opinion of Probable Cost***
Alternative 1A - Left Abutment Filter and Drain System	$ 13,951,000
Alternative 1B - Left Abutment Cutoff Wall	$ 40,641,300
Alternative 2 - New Roller Compacted Concrete Dam	$ 45,120,600

The modifications proposed in Alternative 1A were judged to provide a reduction in the annual probability of failure from PFM 1 of ***1.5×10^{-2} to 6.0×10^{-5}.*** Flood surcharge storage and the potential 3-foot raise in the maximum water surface elevation were considered in estimating this reduction.

9.2 RECOMMENDATIONS

Based on the results of the evaluation of alternatives, URS recommends the following:

- Continue detailed observations of the dam until rehabilitation is completed, to identify changes that could indicate increasing risks.
- Proceed with the engineering design and construction of Alternative 1A to improve the facility's safety and reliability.
- Construct Element 1A and Element 2 concurrently. An additional cost of $1,000,000 would need to be included in the opinion of probable cost for Element 1A to account for dewatering costs and reconstruction of the existing outlet conduit, if it was constructed separately from Element 2. All dewatering costs in the estimated were included in Element 2 and the dewatering is required for both Elements 1A and 2.
- Obtain all required permits for the construction of all of the elements associated with the combined rehabilitation alternative, including Alternative 1A. This includes permits from the USACE and USFS.
- Perform a field investigation of the 1914 landslide area to confirm the dimensions of the filter drain system proposed for that area.
- Perform geotechnical investigations as necessary to confirm assumptions related to Alternative 1A.
- Perform an underwater inspection of the upstream conduit intake structure and conduit to identify dimensions and condition and to identify potential design and construction challenges associated with Element 2.
- Install piezometers in the area between the left abutment crest dike and the 1914 landslide.

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State of Colorado (SEO). 2007. *Rules and Regulations for Dam Safety and Dam Construction* State of Colorado, Department of Natural Resources, Division of Water Resources, Office of the State Engineer. Denver, CO.

State of Colorado (SEO). 2008. *Hydrologic Basin Response, Parameter Estimation Guidelines.* State of Colorado, Department of Natural Resources, Division of Water Resources, Office of the State Engineer. Denver, CO.

State of Colorado Department of Natural Resources Division of Park and Wildlife Dam Operations, (CPW) 2011, *Hydrology Study for Beaver Park Dam, Draft No. 1" July 7, 2011*

U.S. Department of Interior, Bureau of Reclamation (Reclamation), 2003, *Guidelines for Achieving Public Protection in Dam Safety Decision Making*, Technical Service Center, Denver, Colorado, June 15, 2003.

Appendix A

Historical Drawing Datum Comparison

	Historical Drawings											2011 Datum
Feature	1912	1913	1947	1950	1951	1966	1969	1970	1976	1977	1988 - Phase 1	NAVD 88
Dam Crest (1912 Proposed)	190.0	190.0	83.0									8801 - 8808
Dam Crest (1950 Proposed Raise)				95.0	95.0		83.0[1]	8795.0	8795.0	8795.0	8794.0[2]	8817.0
Dam Crest (1988 Proposed Raise - Phase 1)											8811.5	8833.5
Top of Parapet / Concrete Wall (1912 Proposed)	191.0	191.0										8802 - 8809
Top of Parapet / Concrete Wall (1947 Proposed Raise)			88.13									8810.13
Top of Seepage Wall across Spillway (1950)				82.0								8804.00
Spillway Crest (1912 Proposed)	185.0	185.0	74.0	74.0								8796.0
Spillway Crest (1950 Proposed Side Channel)				85.0	85.0	8773.3[3]						8807.0
Spillway Crest (1966 Proposed Ogee Modification)						8770.2[3]		8770.2[4]	8770.2[4]	8770.2[4]	8782.0	8804.0
Tower Invert (1912 Proposed)	104.0		0.0				0.0	0.0	8700.0	8700.0		8722.0
Top of Tower (1912 Proposed)	186.0		84.2									8806.2
Top of Tower (1950 Proposed Raise)				96.0				95.0	8795.0			8817 - 8818
Outlet Works Inlet Invert (Proposed 1947)			1.0							8701.0		8723.0
Outlet Works Outlet Invert (Proposed 1947)	95.0							-4.0	8696.0	8696.0		8718.0
Outlet Works Outlet Invert (Proposed 1977 Extension)										8685.9		8707.9
Tunnel Outlet Invert										?8687?		8709
Freeboard (Dam Crest to Spillway Elevation)	5.0	5.0	9.0	10.0	10.0			[4]	[4]	[4]	29.5	29.5
Adjustment to 2011 Datum (NAVD 88)	8611 - 8618	8611 - 8618	8722	8722	8722		8722		22	22	22	0

[1]The 1969 dam crest elevation appears to be taken in error from the 1947 drawing.

[2]The 1988 existing dam crest elevation appears to take into account one foot of settlement.

[3]The 1966 drawings appear to show the spillway elevations in error.

[4]The spillway elevations appear to be an error propagating from the 1966 historical drawings.

Appendix B

Beaver Park Reservoir - Synthetic Inflow Record Development Memo

COLORADO PARKS & WILDLIFE

711 Independent Avenue • Grand Junction, CO 81505
Phone (970) 255-6100 • FAX (970) 255-6111
wildlife.state.co.us • parks.state.co.us

Memorandum ***Via Email***

To: SLVID – CPW Cooperative Project Participants

From: CPW Water Resources Unit, Collin Robinson

Date: 2011.11.17

Re: Beaver Park Reservoir – Synthetic Inflow Record

In order to evaluate potential benefits of proposed conjunctive operation of Beaver Park Reservoir (BPR) and Rio Grande Reservoir (RGR) which would occur once structural rehabilitation of these facilities is performed, it is necessary to gather information needed for modeling possible operating protocols. Among other pieces of information, reservoir inflow time series data is needed as a matter of primary importance. The Colorado Parks and Wildlife (CPW) Water Resources Unit (WRU) was tasked with providing BPR daily inflow data from 1980 through 2008 to the ad-hoc group representing CPW and the San Luis Valley Irrigation District (SLVID) working on this cooperative project (Coop Project). As there is no inflow gage at BPR, no actual record exists, so a synthetic record was generated. The data are provided herewith as an attachment in Microsoft Excel format. This memo explains the process by which the record was synthesized.

Beaver Park Reservoir Location

BPR (A.K.A. Beaver Creek Reservoir, F.K.A. San Luis Valley Reservoir) is a CPW-owned facility located in the southern portion of Township 39 North, Range 3 East, of the New Mexico Principal Meridian, with its dam near the middle of Section 28. It is an on-channel reservoir, impounding and regulating the flow of Beaver Creek, which is a tributary of the South Fork of the Rio Grande, which is in turn tributary to the Rio Grande main stem.

The Colorado Division of Water Resources (DWR) maintains a streamflow measurement station (gage) on Beaver Creek, below the reservoir. Daily data are available for that location from the period April 30th, 1997 to present day. The data are dubbed admin flow, as they are classified as provisional. The United States Geological Survey (USGS) maintains a gage downstream on the South Fork of the Rio Grande. Daily data are available for that location from the periods prior to 1980 through September of 1995 and October 1998 through September 2007. Some of these values are estimates, which have been approved for publication by USGS. There is also a USGS gage farther downstream on the Rio Grande near Del Norte, which has a lengthy period of record. These gages, BPR, the area's terrain, and local hydrography are shown on the attached vicinity map, Exhibit 1.

Inflow Reconstruction

In addition to streamflow data, DWR also maintains reservoir stage records. Daily stage data for BPR are available on-line for the last year, but not for the historical period of interest. Hard-copy print-outs are, however, maintained at the Water Division 3 office in Alamosa. Copies of these were recently obtained by WRU staff. Monthly records were found for most months up to mid-March of 2003, after which daily data were available through October of that year, and again from March of 2004 through October of 2006. It is possible that additional daily data exists in the hard-copy collection, but it was not found to be readily obtainable within the time constraints existing when the copies were made. If refinement is later determined necessary, a more rigorous records search can be undertaken then.

Given the flow data from the gage below the reservoir and the stage data from the reservoir, it is possible to approximately reconstruct inflow by adjusting outflow according to change in storage. Reservoir evaporation is also a factor. Seepage is assumed essentially instantaneous. It is assumed that any non-tributary seepage (in the limited sense that it does not contribute to gage flow) or variation therein is immaterial to the intent of the effort. Difference in drainage area between the inlet and the gage is assumed negligible. Some time-step artifacts are accepted as contributing to the approximate nature of the results. Basically, synthetic inflow is estimated as outflow (gage flow) plus change in storage plus evaporation. Since gage data below BPR is only available from 1997 to present, inflow reconstruction will not meet the Coop Project's needs, but it is a way to start.

Most of the stage records have corresponding reservoir volumes. The difference between the volumes at any two stages is the change in storage. Based on the records, BPR has been considered to have a relationship between stage and volume that can be described as volume in ac-ft equal to 1.105 times stage in ft plus 0.642 times the square of stage in feet.

Evaporation volume is estimated as loss in feet (per time) times water surface area in acres. Given the stage-storage relationship cited above, and based on the average-end-area method of volume computation, the approximate stage-area relationship was found to be surface area in acres equal to 1.3 times stage in ft, which was used to estimate surface area for evaporation calculations. Due to the imprecise nature of the inflow reconstruction technique with all of its assumptions and the surface area estimation technique which is known to be inexact, general approximation of evaporative loss was deemed adequate for this effort. This was accomplished using Colorado State Engineer's Office Policy 2003-2 distribution of NOAA Technical Report NWS 33 Map 3 Average Annual Evaporation at the site to develop a monthly schedule of evaporation rate. The interpolated Map 3 value was rounded up to the nearest inch (37). The schedule was truncated by snow cover, as estimated using temperature and precipitation information, and assuming a 30-day fall lag in ice-over to roughly account for heat storage. Gross evaporation was then offset by precipitation (100% of incident precip) for the purpose of the flow reconstruction effort. NCDC 1981 – 2010 monthly normal weather data from the Del Norte, Colorado station were used after adjustment for orography and lapse. The resulting monthly loss values were applied evenly across each day of each respective month, for all years and multiplied by estimated surface area to yield very approximate evaporation volumes, which were then converted to average daily flow rates. Since live inflow is believed to be perennial, and larger than the largest equivalent rate of evaporation (~1.4 cfs) thus produced, evaporation was assumed to always be additive to change in storage, as opposed to contributing thereto.

As the Coop Project requires daily data, the monthly stage records are only useful if interpolated to a daily time step. This was done, and used for inflow reconstruction. Additionally, some anomalies (single days of missing data, transposed digits, etc.) in the various time series were manually overwritten to provide a continuous set of matching input. The time series plot attached as Exhibit 2, shows a sample of the results. The small hollow light blue triangles represent the streamflow from the gage below the reservoir, in cfs, plotted against the log scale on the left. The orange line represents reservoir contents (volume, storage), in ac-ft, plotted against the standard scale on the right. The small dark blue squares represent the reconstructed inflow derived from the other two (and evaporation), and are also plotted with respect to the left axis. As can be seen, when the change in storage (slope of the orange line) is positive (value increasing with time), the reconstructed inflow is larger than the gage flow, and visa versa. The steeper the slope, the bigger the difference between the flows (the difference is apparently exaggerated at lower flows by the log scale). These are indications that the reconstruction method is generally functional. The significant discontinuities in the inflow data occurring at the beginning (or end) of each month (and most obvious during the low flow, winter months) are artifacts of the linear interpolation of monthly storage values to daily time step, and the corresponding slope breaks on the orange line. These discontinuities cannot easily be rectified without introducing additional assumptions, errors, and artifacts, and they broach a significant quality consideration with regard to the inflow results.

The attached Exhibit 3 shows a similar sample from the period for which daily stage values were obtained. It is, noticeably and as expected, free from the discontinuities caused by interpolation of storage values. Thus, the inflow estimates reconstructed from true daily data are deemed to be reasonable. Some of the large daily fluctuations are believed to be time-step artifacts (due to computation of daily change in storage from the previous day being added to gage flow in the present day, all on daily average basis as opposed to continuous data), so a second generation reconstruction was developed using a concentric three-day average with backstep to smooth the series. This smoothed data is plotted as a thin red line in the attached Exhibit 4. Gaps in the line reflect periods during which true daily values were not available for all inputs. The smoothed reconstructed inflow values based on true daily inputs are thought to be the best available information. However, with the input data on hand, they could only be derived for 1,160 days from March 22, 2003 through October 31, 2006 (they could also be derived for 2010 – 2011, but the goal for this effort was a record ending with 2008). Exhibit 4 only shows a sample of this data set, since plotting too many time-series values on a single sheet results in a cluttered appearance.

<u>Generation of Synthetic Record</u>

In order to develop an inflow record for the period from 1980 through 2008, it was necessary to extend the smoothed reconstructed inflow values by some means, the most obvious being correlation to some nearby gage with a longer record. The most appropriate gage for this purpose is the South Fork USGS gage shown on Exhibit 1. Exhibit 5 shows data from that gage plotted alongside the smoothed reconstruction of BPR inflow (Beaver Creek above the reservoir). As can be seen, the two series relate well.

Several different types of mathematical descriptions of the relationship between series (power functions, exponential functions, etc.) were tested for goodness of fit, but no single form performed well across the broad range of flows in play, despite some close relationship being evident upon inspection. In order to help sort the matter out, a scatter plot of the values from the two series was prepared, and attached as Exhibit 6. Again, a close relationship is evident, despite a few outliers, anomalies (or artifacts of the reconstruction and smoothing techniques), and clutter. The higher BPR values were found to generally correspond to dates of rising hydrograph, while the lower BPR values were found to generally correspond to dates of falling hydrograph, revealing a pattern similar to classical hysteresis.

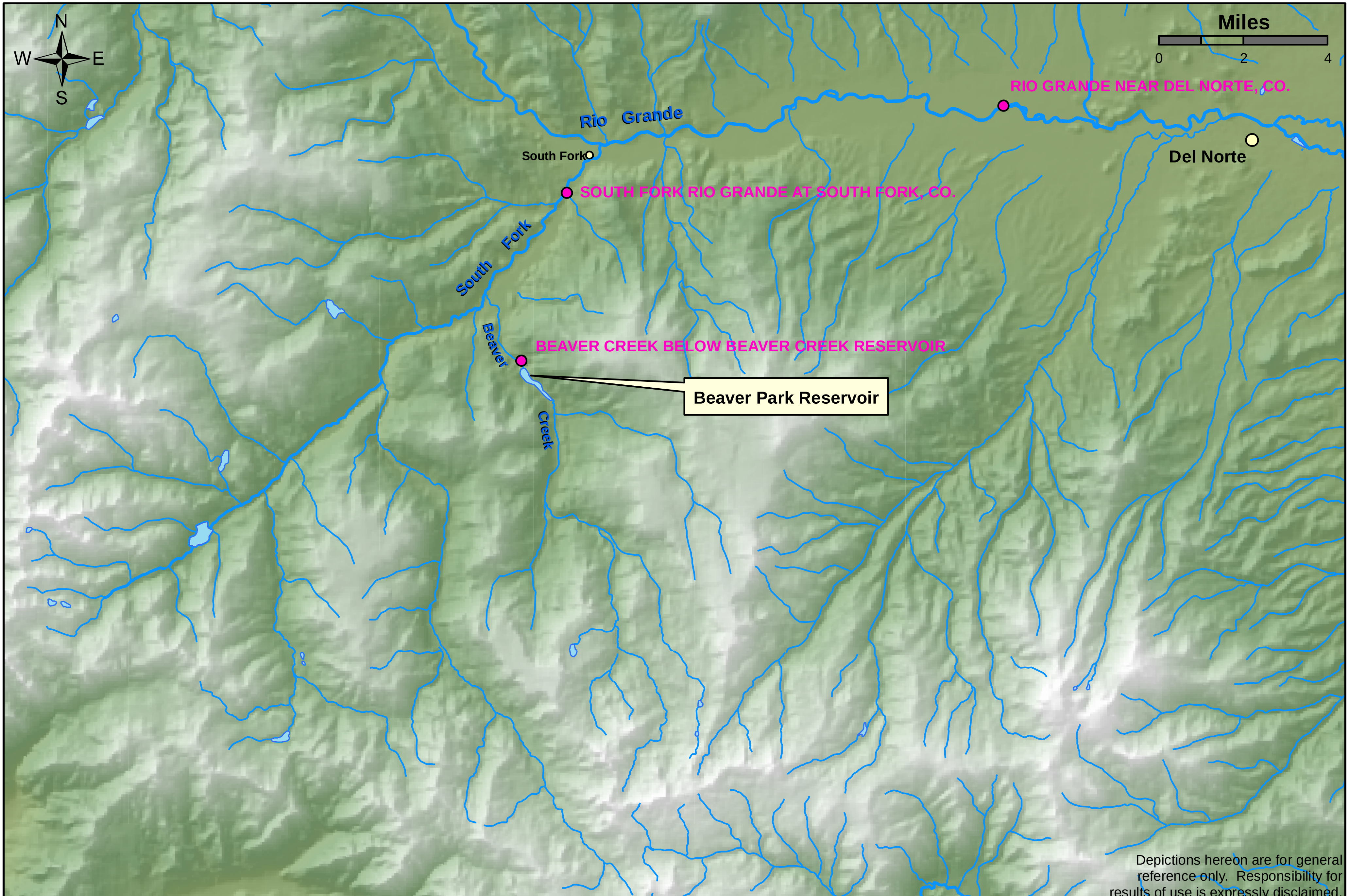
In order to examine this pattern, the scatter plot was divided into two data sets, one consisting of points from dates prior to hydrograph peak and one for points which occurred after peak. The hydrograph peaks represented by smoothed reconstructed inflows based on daily reservoir stage records (2003 – 2006) occurred on days ranging from the 138th to the 142nd of their respective years, with the mean being between day 140 and day 141. Given this, together with the observation that the hydrograph generally exhibits low slope around the new year holiday, points for flows which occurred earlier than the 141st day of their respective year, were plotted as a rising limb data set, and points for flows which occurred later were plotted as a falling limb data set. The result is attached as Exhibit 7, with the rising limb data plotted as solid blue diamonds and the falling limb data plotted as hollow red squares. Again, Several different types of mathematical descriptions of the relationships were tested, with polynomial curves yielding the best result. The solid lines on Exhibit 7 show the curves determined for each data set, and are correspondingly colored, and labeled with their third order polynomial equations and correlation coefficients.

It may be noticed that at very low flows and again at very high flows, the curves intersect (and cross). The intersection is usual for the case of a hysteresis loop approximated by a pair of polynomial functions. Since the range of South Fork gage flows represented on the divided scatter plot and in the curve fits is from 23 to 2,650 cfs, while the range of South Fork gage flows recorded during the period of interest (1980 – 2008) is from 13 to 2,980 cfs, some values on either side of the intersections require treatment. Polynomial curves are good tools for interpolation, as between known data points, here between the intersections, but they are notoriously bad tools for extrapolation outside such ranges, as large relative errors can result from continued curvature of the functions. In order avoid this pitfall, a somewhat lower level of confidence in goodness of fit was accepted (had to be accepted given the data and lack thereof beyond the curve intersections) and linear approximations were developed for extrapolation from the intersections. For the nearly pure extrapolation above the upper intersection, found to occur at a South Fork flow value of roughly 2,632 cfs, the equation of a straight line passing between the points of polynomial curve intersection was employed. It roughly estimates BPR inflow in that uncommon range to be equal to 2.557 plus 0.157 times South Fork flow. For the range below the lower polynomial curve intersection, found to occur at a South Fork flow value of roughly 132 cfs, the equation of a line determined by trial and error to optimize low flow goodness of fit was employed. It estimates BPR inflow in that range to be equal to 8.75 plus 0.11 times South Fork flow. The multi-fit correlation is represented on the attached Exhibit 8 as a set of black lines.

Application of the multi-fit correlation described above to flow data from the South Fork gage results in good matching with the smoothed daily BPR inflow reconstruction data, as can be seen in the attached Exhibit 9, which is the same as exhibit 5, except the reconstruction data has been represented by hollow points instead of a line, and the South Fork gage data has been replaced by the result of applying the correlation equations as appropriate for each day and flow, with the line also made blue, representing the synthetic BPR inflow record. Although imperfect, the agreement is quite good, and shows matching of both high and low flows, on both rising and falling limbs of the hydrograph to be reasonably tight. It is on this basis determined that in the present absence of better information, the synthetic inflow values thus derived are the best data for use in modeling reservoir operations using historical daily flows.

In this way the reconstruction data can be extended to form a synthetic record corresponding to the long periods for which data are available from the South Fork gage. As stated above, the modeling effort is based on flow history from 1980 through 2008. Daily data from the South Fork gage are available throughout this period, with the exception of a break from October 1995 through September of 1998, and excepting the end of the period from October 2007 on. In order to fill in the gaps, further extension is necessary. One means of doing this is by reference to some other nearby gage with a similar period of record that spans the gaps. The Rio Grande Near Del Norte gage is useful for this purpose. Annual flows from the Rio Grande gage were divided by the mean annual flow for the period from 1980 through 2008 in order to develop a series of normalized values representing the ratio of annual flow to mean annual flow for each year. The same was done for the South Fork gage. A scatter plot comparing the ratios is attached as Exhibit 10. As can be seen, the points do not fall along a single line of perfect correlation, but are reasonably well related.

Accepting the imperfection of the correlation, and assuming that the ratios equate, the Rio Grande ratios for the years with missing South Fork data were then matched to ratios from other years of South Fork data to select the nearest proxy years. In this way, it was determined that missing South Fork gage data for dates in 1995 would be filled in with data from the same dates in 1987, 1996 with 1981, 1997 with 2005, 1998 with 2004, 2007 with 1983, and 2008 with 1991. By this method a completed time series with data for every day between January 1, 1980 and December 31, 2008 was compiled for the South Fork gage. It is acknowledged, that not all of the daily values represent actual flows, but they do substantially represent actual conditions. The multi-fit correlation described above was subsequently applied to the completed series to generate a full synthetic record of daily BPR inflow. The output was rounded to the nearest tenth of a cfs. This data is enclosed herewith in a Microsoft Excel spreadsheet email attachment. Although very crowded, the attached Exhibit 11 depicts the data.



Water Resources Unit

Map Title:	Preparation:	Review:	Date:
General Vicinity	SCR	RV, EP, etc.	2011.11.17

Region:	Area:	Property or Process:	Project:
SW	17	Beaver Park Reservoir	Synthetic Inflow Record

Legend:
- Select streamflow stations, CDSS
- Select municipalities, CDSS
- Division 3 Hydrography, CDSS
- Elevation with hillshade, 90m, based on DEM

Exhibit: 1

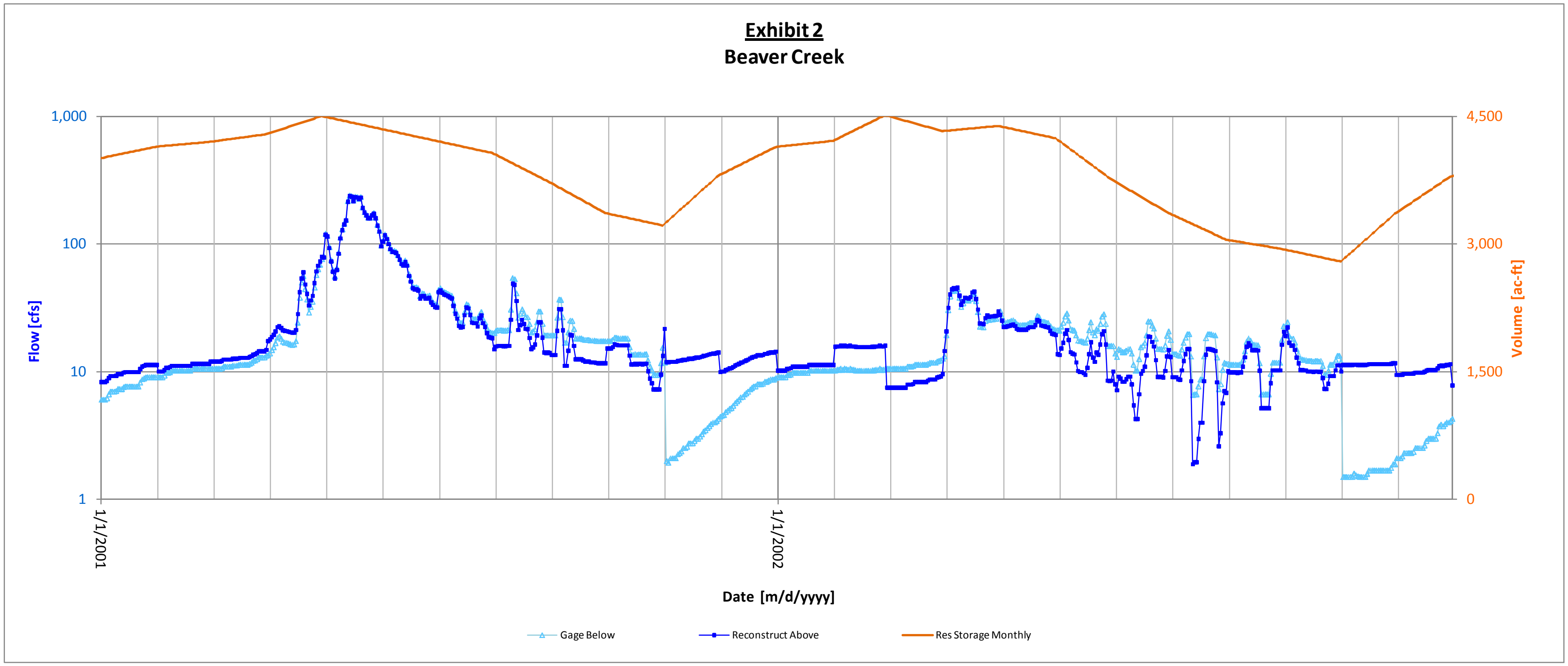


Exhibit 2
Beaver Creek
1,000
100
10
1
Flow [cfs]
4,500
3,000
1,500
0
Volume [ac-ft]
1/1/2001
1/1/2002
Date [m/d/yyyy]
Gage Below
Reconstruct Above
Res Storage Monthly

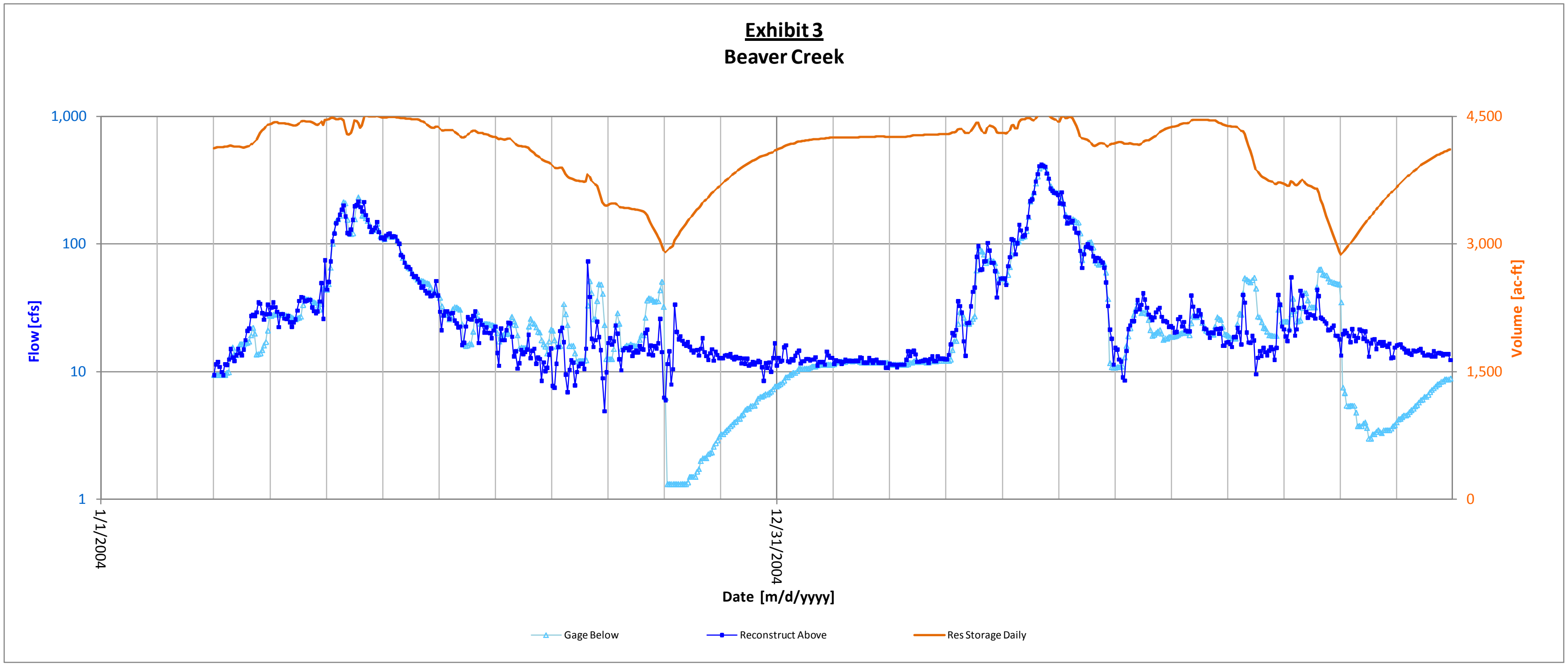


Exhibit 3
Beaver Creek
1,000
100
10
1
Flow [cfs]
4,500
3,000
1,500
0
Volume [ac-ft]
1/1/2004
12/31/2004
Date [m/d/yyyy]
Gage Below
Reconstruct Above
Res Storage Daily

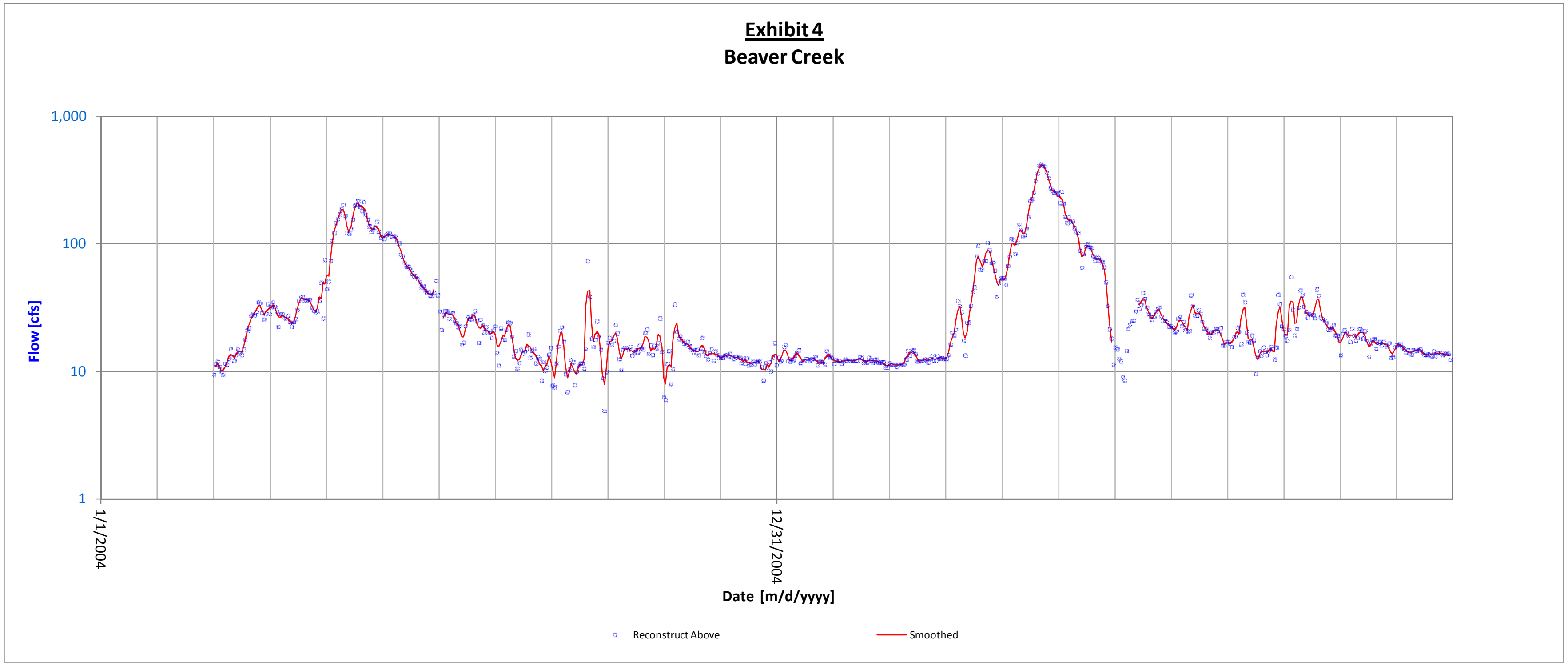


Exhibit 4
Beaver Creek
1,000
100
10
1
Flow [cfs]
1/1/2004
12/31/2004
Date [m/d/yyyy]
Reconstruct Above
Smoothed

Exhibit 5
Gage Comparison

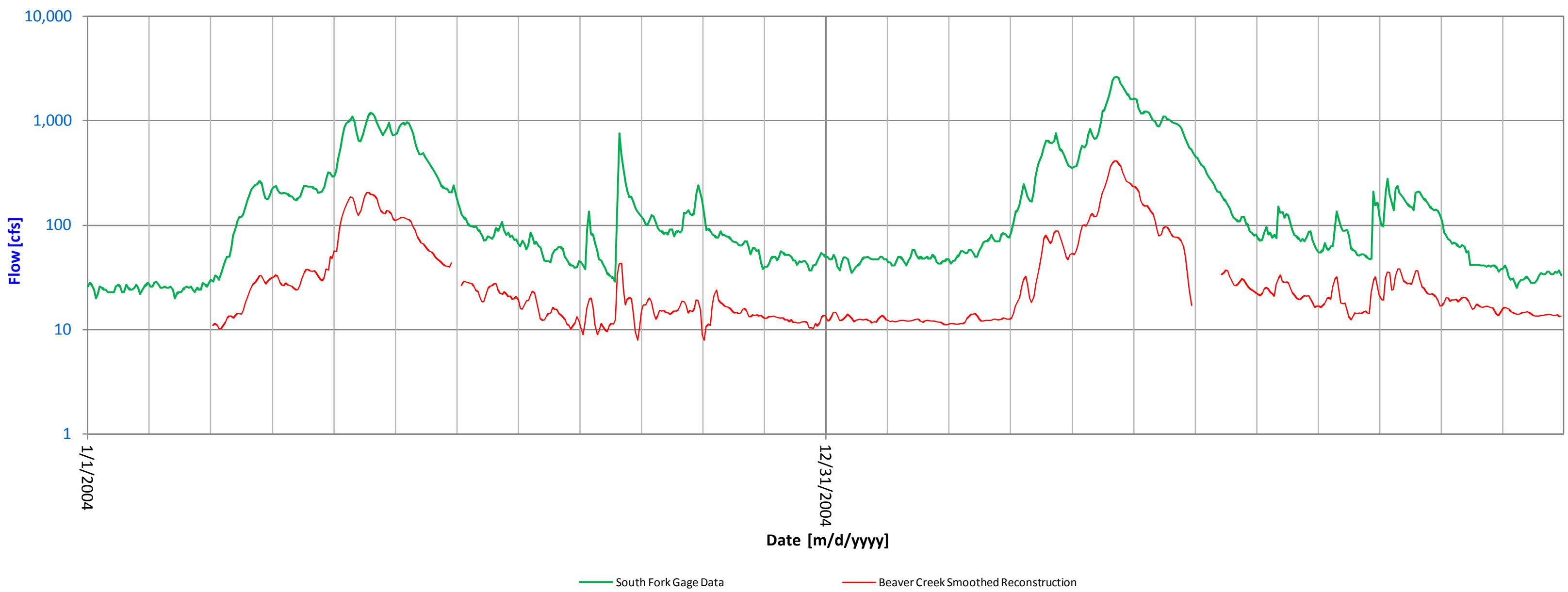
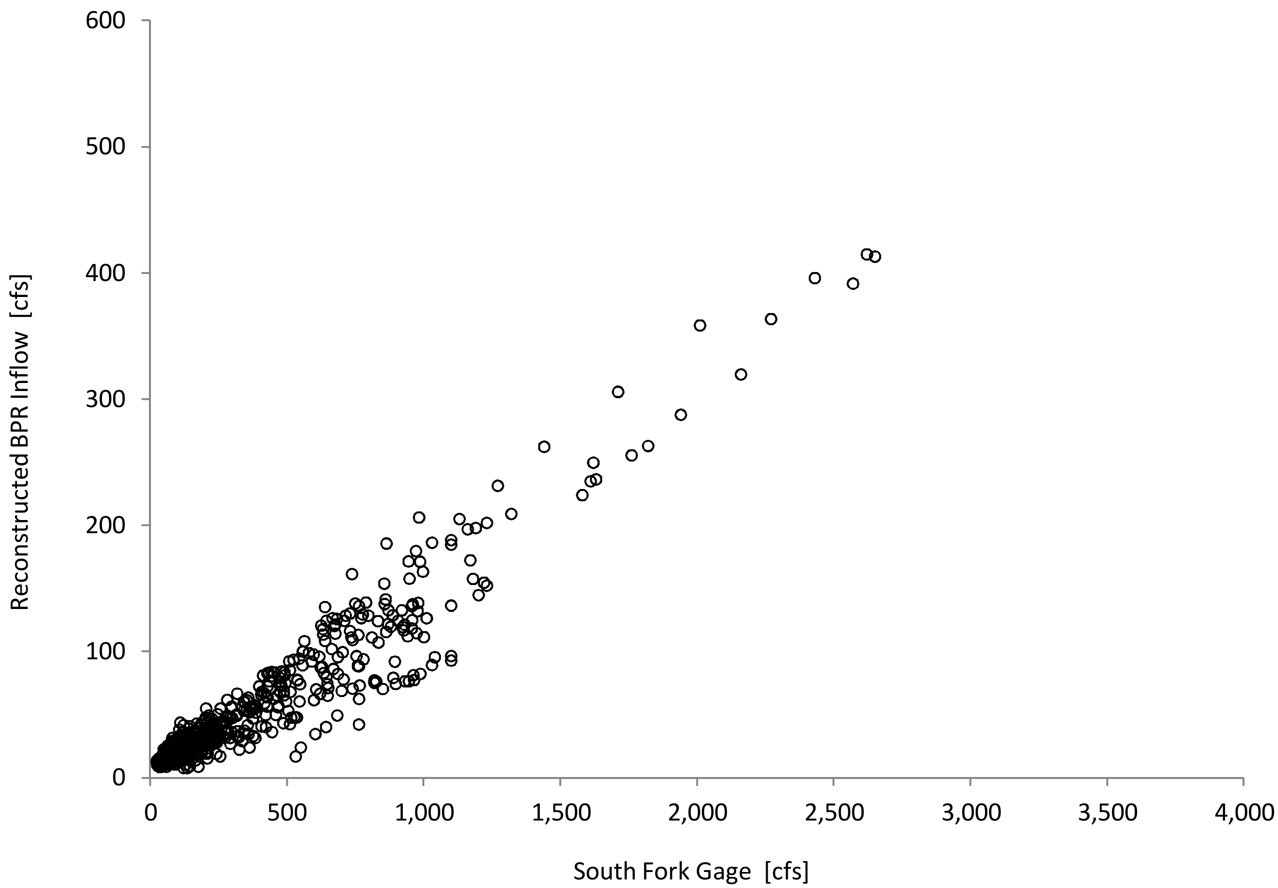


Exhibit 6

Scatter Plot



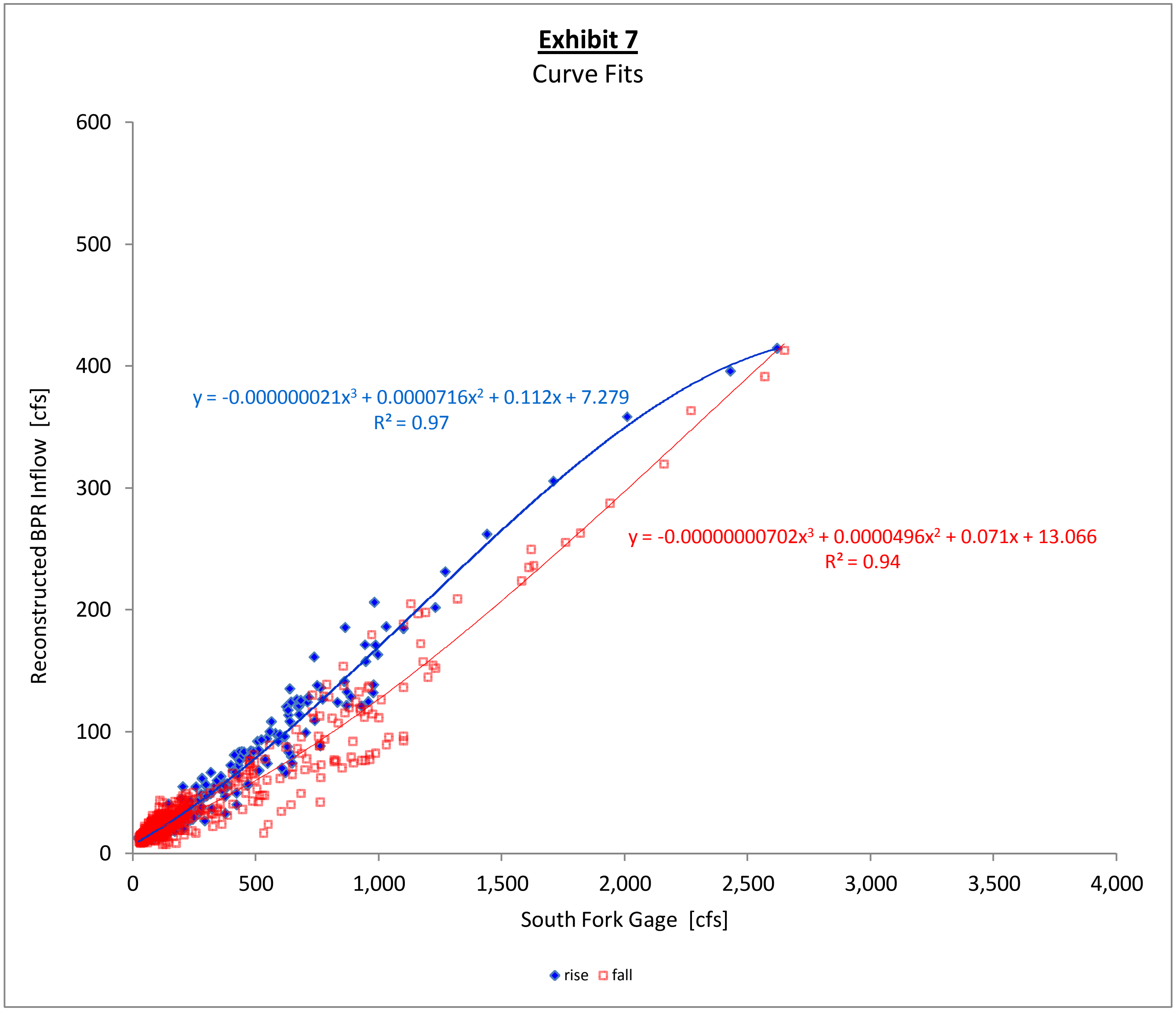


Exhibit 7
Curve Fits
600
500
400
300
200
100
0
Reconstructed BPR Inflow [cfs]
y = -0.000000021x³ + 0.0000716x² + 0.112x + 7.279
R² = 0.97
y = -0.00000000702x³ + 0.0000496x² + 0.071x + 13.066
R² = 0.94
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
South Fork Gage [cfs]
rise
fall

Exhibit 8

Curve Fits

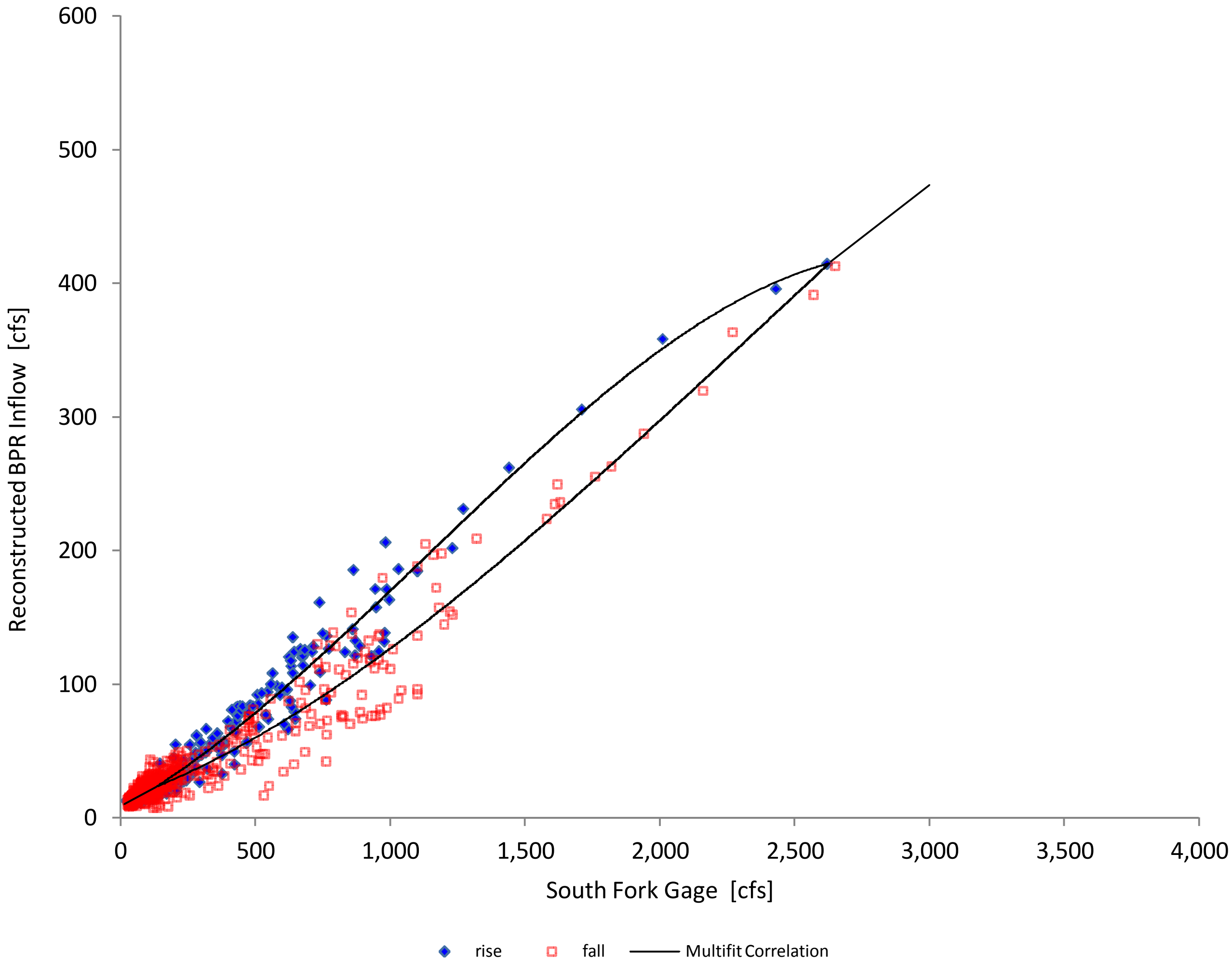
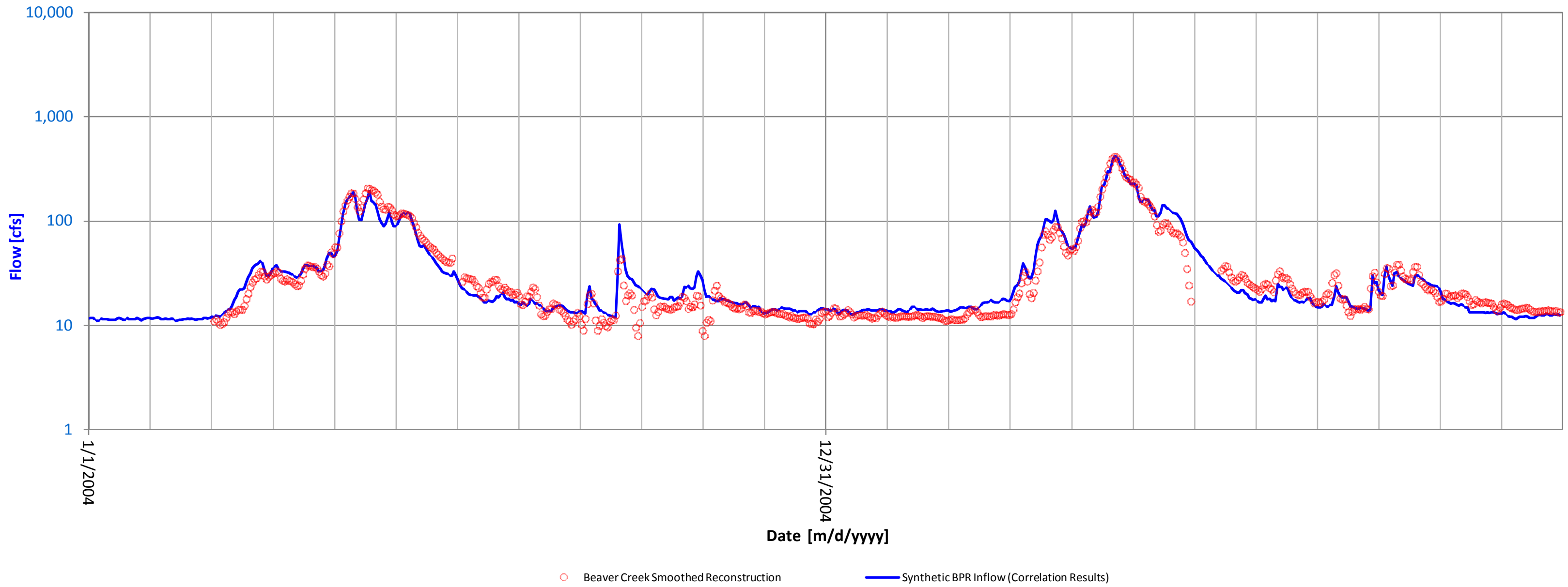


Exhibit 9

Matching Correlation Results



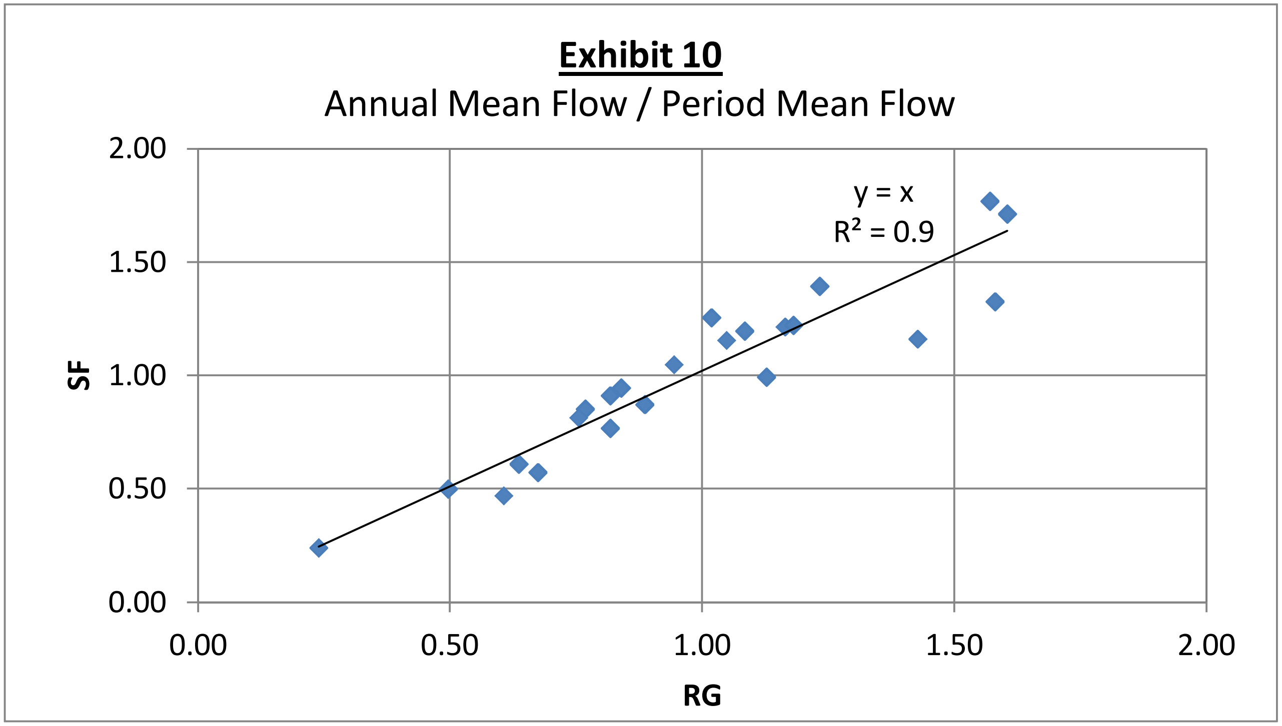


Exhibit 10
Annual Mean Flow / Period Mean Flow
y = x
R² = 0.9
SF
RG
0.00
0.50
1.00
1.50
2.00

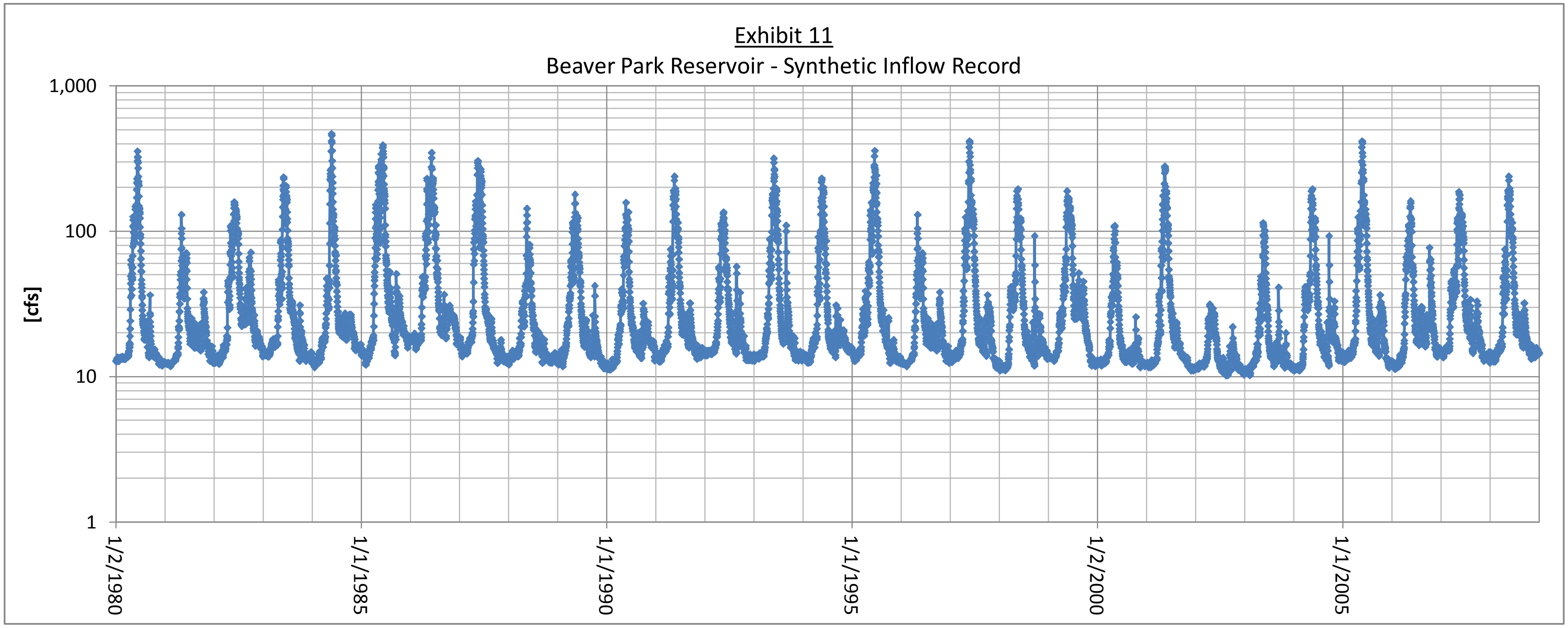


Exhibit 11
Beaver Park Reservoir - Synthetic Inflow Record
1,000
100
10
1
[cfs]
1/2/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005

Appendix C

Seismic Refraction Survey

C.1 INTRODUCTION

Geophysical techniques were utilized as part of the geotechnical evaluation for possible improvements at Beaver Park Dam and reservoir. The objectives of the geophysical survey were to:

- Obtain seismic velocity information to help estimate overburden thickness, and
- aid characterization of the bedrock surface.

This information was sought in selected areas at the site, to help in future design efforts. The geophysical method used to accomplish the above was seismic refraction.

C.2 SEISMIC REFRACTION METHOD

C.2.1 Description of Method

Seismic refraction for engineering applications is most often used to infer geological boundaries as indicated by interfaces with seismic velocity contrasts. Seismic refraction data may be used to characterize:

- The thickness of alluvial or colluvial deposits, or weathered rock
- The depth to the water table and/or competent subsurface layers
- The configuration of the alluvial-bedrock contact
- Relative excavatability (inferred from seismic velocities)

The seismic refraction method consists of transmitting seismic energy into the ground and recording the arrival of direct or refracted sound waves at various distances along the earth's surface. The seismic energy travels within geologic units with a characteristic compressional seismic velocity that is dependent on the density, compressibility, porosity, and fluid content of the geologic layer. The seismic refraction method analyzes the first arrival times of the seismic compression wave (p-wave) versus distance from the source. This analysis is based on Snell's Law which predicts the behavior of a seismic raypath at a geologic interface. By measuring seismic velocities, as inferred from the recorded first-arrival travel times, and by determining seismic velocity contrasts, an interpretation can be made of the configuration and depths of subsurface seismic layers, which often infers or correlates to geologic units. A schematic of the method is shown in Figure C-1.

To record seismic refraction data, a seismic source, cables, geophones, and a seismograph are required. The seismic source may be a sledge hammer, buried explosives or an elastic wave generator, depending on the depth of investigation and attenuation properties of the near-surface material. Geophones implanted in the ground translate vibrations into an electrical signal displayed on the seismograph. The seismic record (seismogram) is a plot of the seismic data with response amplitude versus time recorded. Data can be output on hard copy records and/or saved on personal computer (PC)-compatible disks.

A minimum of three seismic sources are usually used for each seismic refraction setup or spread, one source on each end of the spread, and one in the middle. If the target layer is likely overlain by a relatively thick overburden, offset shots must be used to ensure that sufficient coverage of the target layer is made. However, the use of offset sources can be problematic if topographic or other physical constraints exist in the investigation area.

Data collected can be processed and analyzed with one of several interactive seismic refraction interpretation packages. These packages, which operate on a PC, make all necessary topographic corrections, construct time-distance plots, allow calculation of apparent layer velocities, and calculate depths at each geophone location. From the seismogram, the arrival time of the compression wave for each geophone is selected and plotted versus the distance of each geophone from the seismic source, commonly denoted a time-distance plot. Analysis of the time-distance plot allows calculation of seismic compression wave velocity of the subsurface material(s), or several velocities for multiple layers if they are present. Application of Snell's Law governing the angle of refraction at an interface between layers with different seismic velocities permits calculation of upper layer thicknesses. From the analyses and results, a cross-section of the seismic layers is produced. Cross-sections from individual seismic spreads can be tied to available geologic or geophysical borehole information, as well as to other seismic spreads, to make a final geologic interpretation of the surveyed area.

C.2.2 Method Limitations

There are several potential limitations of the seismic refraction method that must be kept in mind when using it and interpreting the results. One limitation involves the primary assumption made in refraction interpretation that the seismic velocity of the subsurface increases with depth. If the velocity of a layer is less than that of the layer immediately overlying it, the observed travel time due to the deeper layer will be slower and not easily measured, yielding a first-arrival travel time of the faster layer. A decrease in velocity with depth can cause layers to be hidden or shadowed by shallower, faster velocity layers. This can lead to depth estimates that may be in error.

Another limitation of the method is that a target layer must be sufficiently thick to be detected. The thickness required depends on the layer depth, the velocity contrast with overlying and underlying layers, and the field parameters utilized during data acquisition and recording. Additionally, the degree of subsurface weathering can make the interpretation of discrete seismic interfaces more difficult.

In some instances, the seismic refraction method is limited solely by physics. That is, the target geometry can be such that refracted raypaths to a target structure are either limited, or non-existent. For example, consider the scenario of a steep-walled narrow valley and a deep incised channel resulting in a fluctuating, but very deep bedrock surface. Completing a seismic refraction line in light of this scenario, and also considering Figure C-1, one can see that the geometry of the canyon would limit the availability of seismic refraction raypaths to penetrate to the bedrock, resulting in failure to resolve or characterize such a target. Refractions could still occur but would likely be from shallower layers, and not from the desired bedrock interface.

We must also keep in mind that the depth to the refractor identified and mapped in seismic refraction results is the shortest distance between the refractor and the geophones represented by the smallest travel time. For most geometrics, this distance is a true vertical depth. However, for a geometry in which the seismic refraction is conducted along strike of a steeply-dipping interface, the shortest distance to the refractor may not be truly vertical. In such a scenario, the refractor ‘depth” will be a minimum depth, and not necessarily the true depth vertically below the detecting geophones.

C.3 FIELD PROGRAM

Four seismic refraction spreads encompassing two seismic refraction lines (denoted SL1 and SL2) were completed during the field program totaling approximately 1,900 linear feet. The final locations of the seismic refraction spreads were chosen based on the project objectives and to obtain information in specific areas at the project site. Final locations of the seismic refraction spreads are shown in Figure C-2.

Seismic refraction data were collected using a signal-enhancing, Geometrics Strataview Model S24 seismograph. A geophone spacing of 20 feet was used on each seismic spread, resulting in seismic spread lengths of 460 feet. One seismic line (SL1) used one spread, and the second seismic line (SL2) included three overlapping seismic spreads.

Seismic sources were used at both ends of each spread for forward and reverse travel times and at locations within each spread to increase near-surface velocity control. On each of the seismic lines, offset seismic source locations were used to enhance coverage of the bedrock surface. Site topography and site constraints prevented using offsets at some preferred locations. In some instances, this factor coupled with the existence of deep bedrock made complete coverage of the bedrock surface problematic and unachievable along SL1.

For the seismic sources, seismic energy was produced by a small explosive charge. Each explosive charge consisted of approximately ½ to 2 pounds of dynamite initiated by shock tube and a non-electric blasting cap. Overall, the seismic refraction data were of moderate to good quality.

C.4 DATA PROCESSING AND INTERPRETATION

Processing of the seismic refraction data involved the construction of a time-distance plot for each seismic spread. To construct the time-distance plots, the first compressional wave arrival at each geophone location was plotted versus the source-to-geophone distance. Velocities of the seismic layers were calculated based on the slope of the best-fit lines through the plotted compressional-wave time-distance data. The intercept of each velocity slope at time zero was used to calculate depths to particular seismic interfaces.

Interpretation of refraction survey data involved the computation of average velocities over surveyed volumes of subsurface material. For the upper seismic layer in which the direct seismic wave arrived at the geophone first, the velocity observed was the true average velocity between the energy source and the geophone. For deeper interfaces in which the refracted seismic wave arrived at the geophone first, the velocity observed is usually an apparent velocity. If the refractor surface is flat-lying, the apparent velocity will be equal to the true average velocity. If, however, the refractor surface is dipping or has a variable surface, the true average velocity can be estimated utilizing the apparent velocities obtained from the data collected form both the forward and reverse seismic energy sources.

Time-distance plots were constructed and interpreted using the software program GREMIX, an interactive seismic-refraction processing routine developed by Interpex Limited. GREMIX allowed interactive plotting of each travel time plot, selection of velocity slopes, and identifications of forward and reverse shot pairs. With elevation information from each geophone, the program calculated seismic layer thicknesses for each geophone travel time. Final interpretations are displayed for each seismic line in cross-section form in Figures C-3 and C-4.

Each of the seismic refraction plots display three panels. The top panel displays a plot of the arrival time data and is the p-wave arrival at each geophone versus distance along the seismic spread. The middle panel, which is the depth section, is the interpreted subsurface model based on the input arrival time data. The lower panel, which is the velocity section, displays the seismic velocities associated with the interpreted depth section. The depth and velocity sections are plotted in terms of seismic layers, which represent the interfaces at which a distinct velocity contrast is interpreted to exist. These seismic layer interfaces often relate to geologic boundaries as well, depending on the seismic velocity contrast observed in the geologic section. Note that distances portrayed in the plotted panels are slope distances. Table C-1 summarizes the seismic refraction results.

C.5 GEOPHYSICAL RESULTS

The travel time data, resulting cross sections and seismic velocities are plotted for the seismic profiles in Figures C-3 and C-4. The seismic refraction spreads indicate a three-layer model of the subsurface. The near-surface seismic layer has a seismic velocity ranging from 870 feet per second (ft/s) to 1,560 ft/s. The thickness of the near-surface seismic layer varies from about 5 feet to a maximum of about 56 feet. Based on geologic mapping and field observations, this unit probably consists of colluvial overburden materials.

The intermediate seismic layer has a seismic velocity ranging from 1,570 ft/s to 3,390 ft/s, and has a thickness of 10 to 135 feet. Along SL1, a deeper and higher-velocity unit was not detected, so the thickness of seismic layer 2 along that line is unknown. Based on geologic mapping completed, this layer is likely glacial moraine and outwash materials.

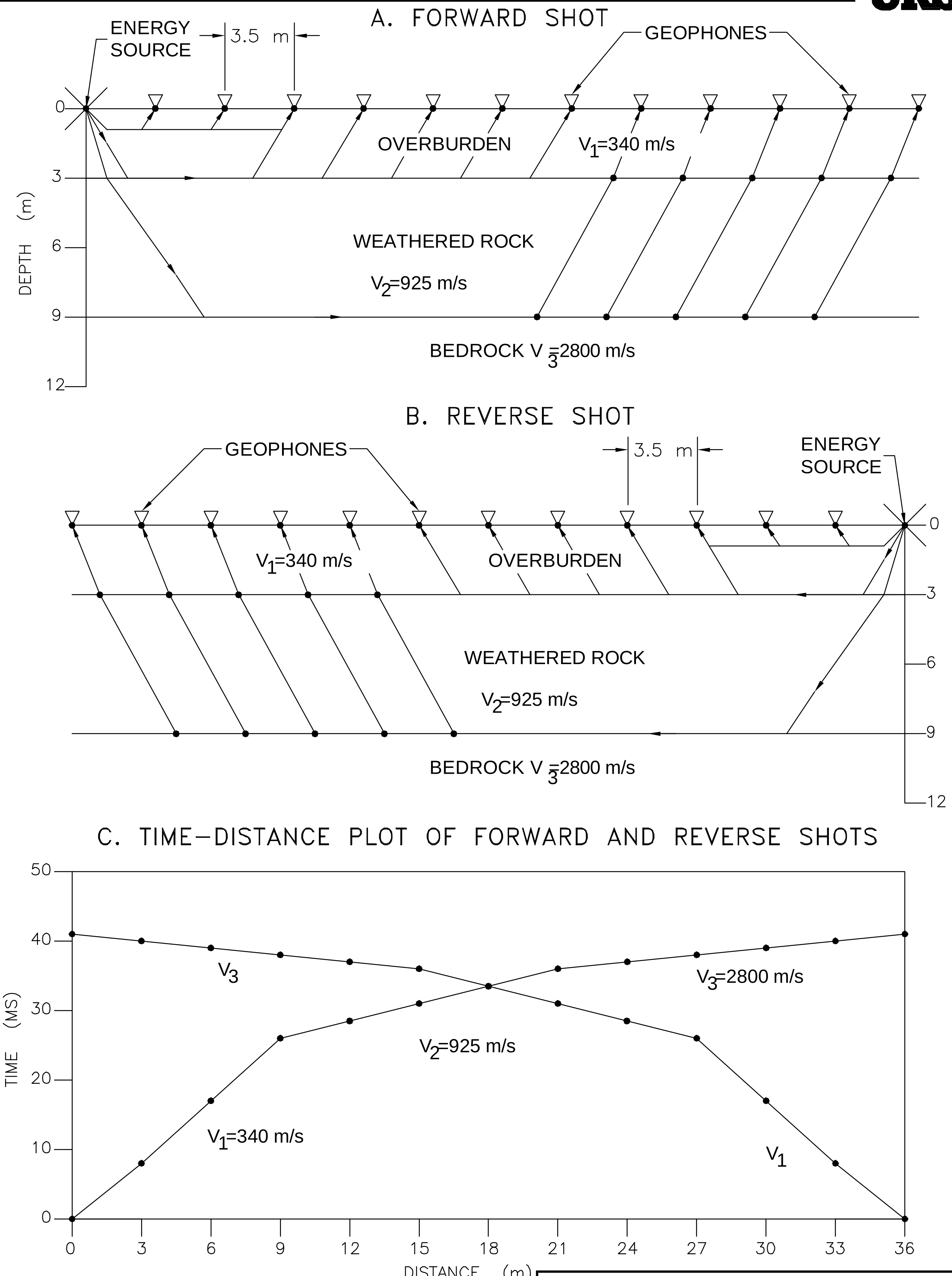
Seismic layer 3, which is probably variably-weathered bedrock, has a seismic velocity ranging from 7,560 ft/s to over 10,320 ft/sec. The depth to this unit varies from 25 feet to over 150 feet on lines SL2 to SL4. On SL1, the bedrock unit was not detected across the seismic line, despite using significant offset seismic sources. A minimum depth to a competent bedrock unit along line SL1 would be from 125 feet to more than 200 feet.

Table C-1
Seismic Refraction Results

Seismic Line	Seismic Layer	Seismic Velocity Range (feet/second)	Thickness of Seismic Layer (feet)	Depth to Most Competent Layer Detected (feet)
1	1	1,250 – 1,970	21 – 56	
	2	2,610 – 3,400	?	
	3	?	?	Greater than 125 feet
2	1	1,000 – 1,010	6 – 8	
	2	3,090 – 3,390	27 – 120	
	3	7,560 – 9,500		35 – 129
3	1	900 – 1,100	5 – 24	
	2	1,570 – 2,200	74 – 135	
	3	8,140 – 10,320		91-150
4	1	870 – 1,560	15 – 52	
	2	2,380 – 2,700	10 – 110	
	3	8,000 – 9,300		25 – 110

C.6 LIMITATIONS

This work was conducted in accordance with reasonable and accepted engineering geophysics practice, and the interpretations and conclusions are rendered in a manner consistent with other consultants in our profession. However, all geophysical techniques have some level of uncertainty and limitations. No other representation to the client is expressed of implied, and no warrant or guarantee is included or intended.



URS

BEAVER CREEK DAM

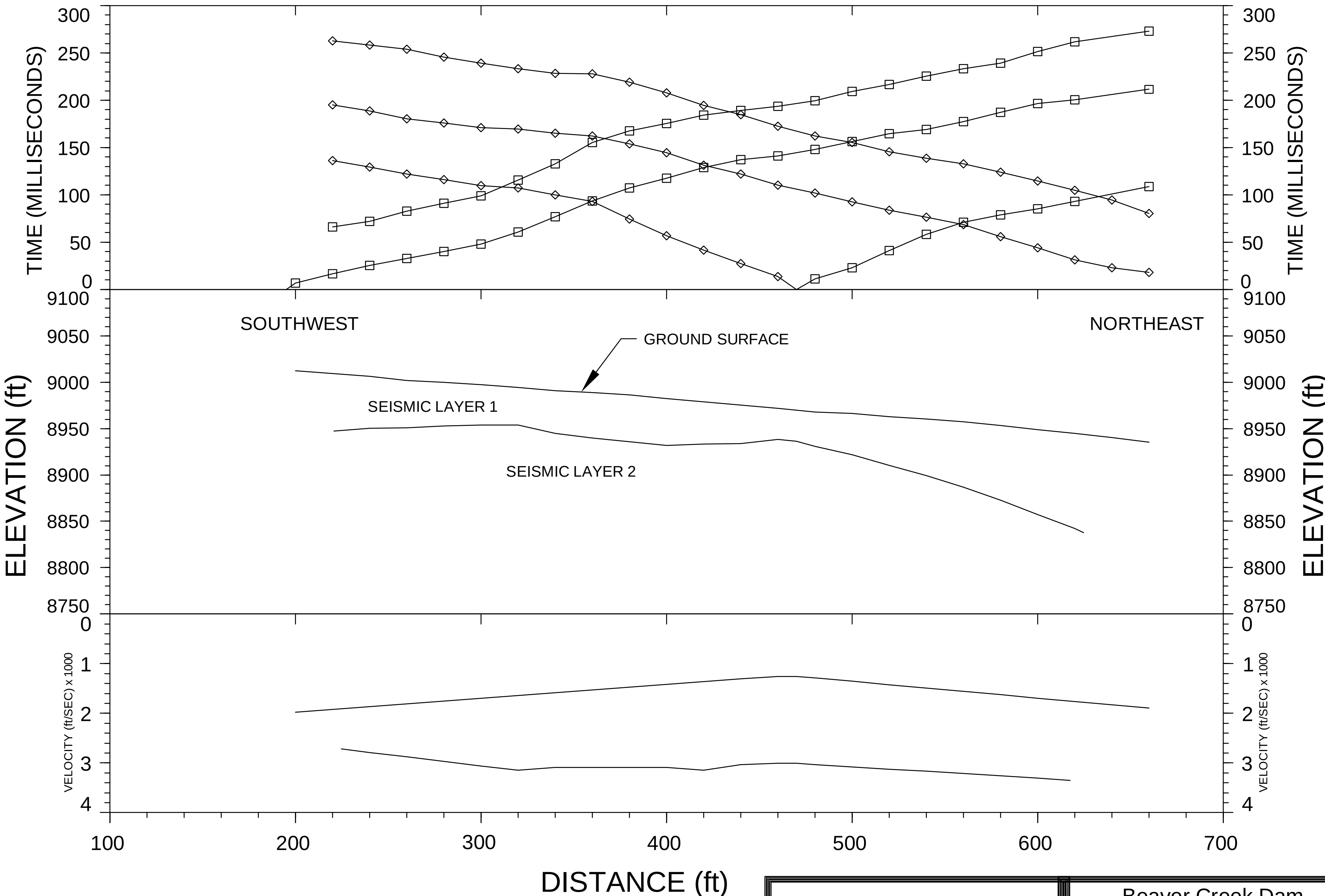
SCHEMATIC OF THE SEISMIC REFRACTION METHOD

Job No. :	22242295
Prepared by :	J.J.N.
Date :	11/16/11

N:\Projects\22242295_Beaver_Park_Alt_Std\Sub_001\7.0_CAD_GIS\SEISMIC SCHEMATIC.dwg

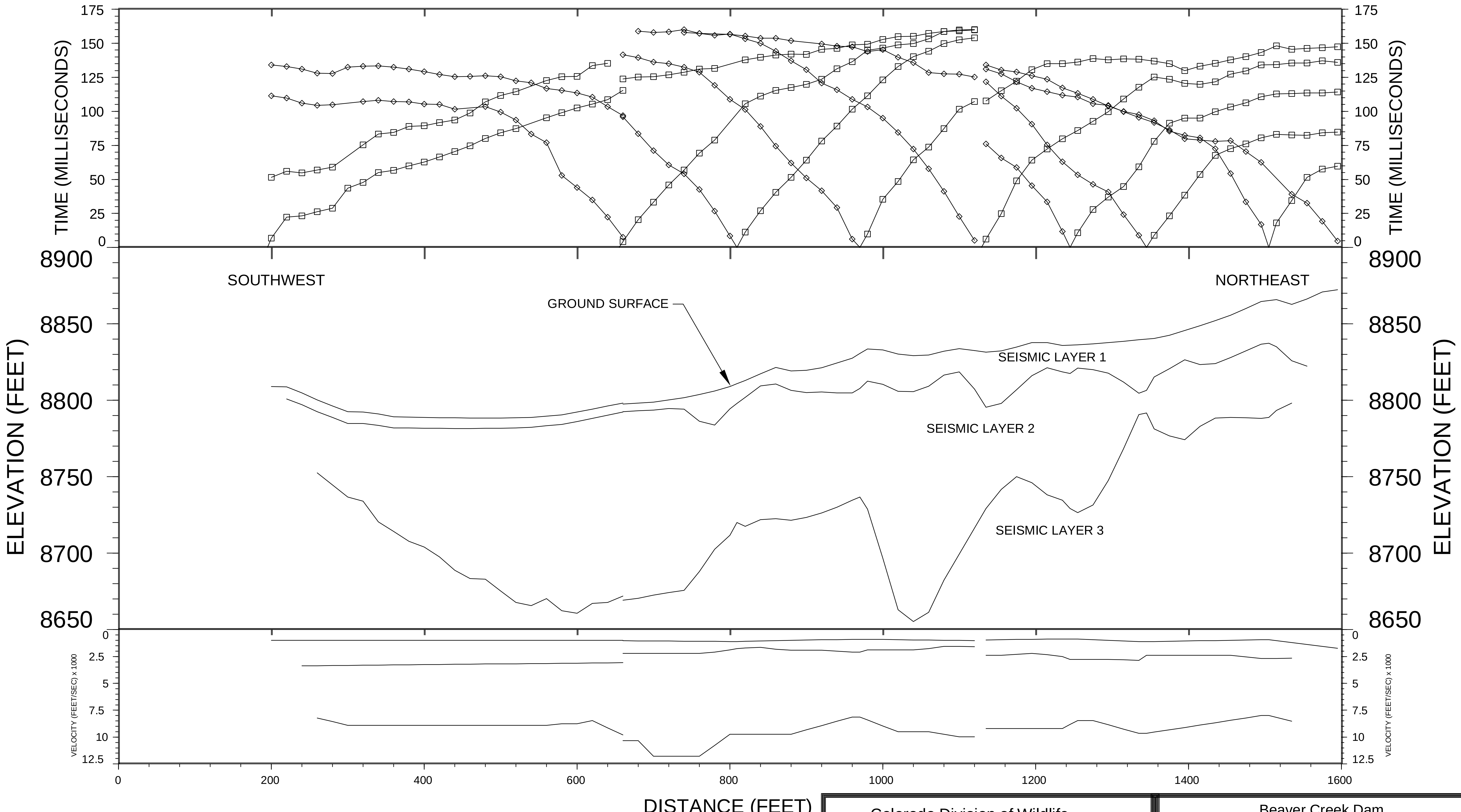
Job No. : 22242295	APPROXIMATE LOCATIONS OF BEAVER PARK SEISMIC LINES
Prepared By : JJN	
Date : 7/14/11	

FIG. C-2



MINIMUM DEPTH TO COMPETENT LAYER,
APPROXIMATELY 125' TO 200'

for: Colorado Division of Wildlife		Beaver Creek Dam
by: URS Corporation		SEISMIC REFRACTION LINE SL1
Data SetSL1	Date: July 10, 2011	
Equipment: Smartseis	Spread: 01	Figure C-3



for: Colorado Division of Wildlife		Beaver Creek Dam
by: URS Corporation		SEISMIC REFRACTION LINES SL2-SL4
Data Set SL2-SL4	Date: July 8,'11	
Equipment: Smartseis	Spread: SL2-SL4	Figure C-4

Appendix D

Alternative Opinion Of Probable Cost Worksheets

Colorado Department of Natural Resources, Division of Parks and Wildlife
Beaver Park Dam
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Summary of Alternative Opinion of Probable Cost

Alternative 1 - Modifications to the Existing Dam	
Alternative 1A: Left Abutment Filter and Drain System (Includes the following elements: Left Abutment Downstream Filter and Drain System (Element No. 1), Outlet Works rehabilitation and gate reconfiguration (Element No. 2), Primary spillway concrete chute (Element No. 3), Crest modifications (Element No. 4), and 1914 landslide area filter and drain system (Element No. 5).	$ 13,951,000
Alternative 1B: Left Abutment Cutoff Wall (Includes the following elements: Left Abutment Cutoff Wall (Element 1B), Outlet Works rehabilitation and gate reconfiguration (Element No. 2), Primary spillway concrete chute (Element No. 3), Crest modifications (Element No. 4), and 1914 landslide area filter and drain system (Element No. 5).	$ 40,641,300
New Dam Alternative Opinion of Probable Cost	
Alternative 2: New Roller Compacted Concrete Dam	$ 45,120,600

Alternative 1 - Element Description and Opinion of Probable Cost Breakdown

Element	Element Description	Cost
1A	Left Abutment Downstream Filter and Drain System (1)	$ 6,392,100
1B	Left Abutment Cutoff Wall	$ 33,082,400
2	Outlet Works Rehabilitation and Gate Reconfiguration	$ 3,892,000
3	Primary Spillway Concrete Chute	$ 1,704,300
4A/B	Crest Modifications (Upstream HDPE Liner or Low Permeability Fill) (2)	$ 1,237,600
5	1914 Landslide Filter and Drain System	$ 725,000

Notes:

(1) The Left abutment filter and drain system and outlet work rehabilitation and gate reconfiguration (Element Nos. 1 and 2) were cost as concurrent activities. An additional cost of approximately $1,000,000 would need to be included in Element No. 1 to account for dewatering costs and reconstruction of the existing outlet conduit.

(2) The Upstream HDPE Liner (Element No. 4A) costs were included in the costs show above for Alternative 1A. The low permeability fill crest modifications (Element No. 4B) has a cost similar to those shown for the Element No. 4A. If the concrete crest wall raise (Element No. 4C) is selected, an additional $600,000 will need to be added to the costs.

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Element 1A - Downstream Filter and Drain System

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	48,652	CY	$ 12.00	$ 583,824
2	Filter Sand	5,405	CY	$ 90.00	$ 486,450
3	Drain Aggregate	6,629	CY	$ 80.00	$ 530,320
4	24-inch Perforated HDPE Pipe	100	LF	$ 250.00	$ 25,000
5	Sheetpile	5,180	SF	$ 40.00	$ 207,200
6	Sheetpile Tie Backs	800	LF	$ 65.00	$ 52,000
7	Fill (on site)	37,974	CY	$ 30.00	$ 1,139,220
SUBTOTAL					**$ 3,025,000**
Unlisted Items 15%					$ 454,000
Mobilization, Demobilization & Preparatory Work 10%					$ 303,000
SUBTOTAL					**$ 3,782,000**
Contingency 30%					$ 1,135,000
CONSTRUCTION TOTAL					**$ 4,917,000**
Engineering 10%					$ 491,700
Construction Engineering 15%					$ 737,550
Permitting 5%					$ 245,850
OPINION OF PROBABLE PROJECT COST					**$ 6,392,100**

Element 1B - Left Abutment Cutoff Wall

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	30,000	CY	$ 12	$ 360,000
2	Cutoff wall	51,000	SF	$ 300	$ 15,300,000
SUBTOTAL					**$ 15,660,000**
Unlisted Items 15%					$ 2,349,000
Mobilization, Demobilization & Preparatory Work 10%					$ 1,566,000
SUBTOTAL					**$ 19,575,000**
Contingency 30%					$ 5,873,000
CONSTRUCTION TOTAL					**$ 25,448,000**
Engineering 10%					$ 2,544,800
Construction Engineering 15%					$ 3,817,200
Permitting 5%					$ 1,272,400
OPINION OF PROBABLE PROJECT COST					**$ 33,082,400**

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Element 2 - Outlet Works Conduit Rehabilitation and Gate Reconfiguration

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Diversion System (including cofferdam and pumping)	1	LS	$ 250,000.00	$ 250,000
2	Demolition of Existing Intake Structure	1	LS	$ 30,000.00	$ 30,000
3	Structural Concrete	268	CY	$ 700.00	$ 187,600
4	Backfill Concrete	60	CY	$ 600.00	$ 36,000
5	60-inch Steel Conduit	135	LF	$ 1,500.00	$ 202,500
6	42-inch Steel Conduit	330	LF	$ 1,100.00	$ 363,000
7	60-inch to 42-inch Steel Transition	1	LS	$ 75,000.00	$ 75,000
8	42-inch to 36-inch Transition	1	LS	$ 50,000.00	$ 50,000
9	30-inch manhole access	1	LS	$ 25,000.00	$ 25,000
10	60-inch X 60-inch Sluice Gate	1	LS	$ 150,000.00	$ 150,000
11	Steel Vent Pipe	210	LF	$ 90.00	$ 18,900
12	SS Hydraulic lines	1,200	LF	$ 250.00	$ 300,000
13	36-inch knife gate	1	LS	$ 110,000.00	$ 110,000
14	Annular Space Grout	80	CY	$ 350.00	$ 28,000
15	Trashrack	1	LS	$ 15,000.00	$ 15,000
SUBTOTAL					**$ 1,841,000**
Unlisted Items 15%					$ 277,000
Mobilization, Demobilization & Preparatory Work 10%					$ 185,000
SUBTOTAL					**$ 2,303,000**
Contingency 30%					$ 691,000
CONSTRUCTION TOTAL					**$ 2,994,000**
Engineering 10%					$ 299,400
Construction Engineering 15%					$ 449,100
Permitting 5%					$ 149,700
OPINION OF PROBABLE PROJECT COST					**$ 3,892,200**

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Element 3 - Primary Spillway Concrete Chute

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	2,773	CY	$ 18.00	$ 49,914
2	Structural Concrete	460	CY	$ 700.00	$ 322,000
3	Ogee Spillway Crest Raise	40	CY	$ 900.00	$ 36,000
4	Grouted Riprap	1,769	CY	$ 225.00	$ 398,025
SUBTOTAL					**$ 806,000**
Unlisted Items 15%					$ 121,000
Mobilization, Demobilization & Preparatory Work 10%					$ 81,000
SUBTOTAL					**$ 1,008,000**
Contingency 30%					$ 303,000
CONSTRUCTION TOTAL					**$ 1,311,000**
Engineering 10%					$ 131,100
Construction Engineering 15%					$ 196,650
Permitting 5%					$ 65,550
OPINION OF PROBABLE PROJECT COST					**$ 1,704,300**

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Element 4A - Upstream High Density Polyethylene Liner

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	6,110	CY	$10.00	$ 61,100
2	Structural Concrete (Left Abutment Rock Knob /MSE Wall Replacement)	415	CY	$700.00	$ 290,500
3	Geotextile	23,254	SF	$0.90	$ 20,929
4	HDPE Liner	11,627	SF	$1.50	$ 17,441
5	Select Fill	922	CY	$30.00	$ 27,660
6	Fill (onsite)	5,558	CY	$30.00	$ 166,740
SUBTOTAL					**$ 585,000**
Unlisted Items 15%					$ 88,000
Mobilization, Demobilization & Preparatory Work 10%					$ 59,000
SUBTOTAL					**$ 732,000**
Contingency 30%					$ 220,000
CONSTRUCTION TOTAL					**$ 952,000**
Engineering 10%					$ 95,200
Construction Engineering 15%					$ 142,800
Permitting 5%					$ 47,600
OPINION OF PROBABLE PROJECT COST					**$ 1,237,600**

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Element 4B - Low Permeability Fill

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	11,272	CY	$10.00	$ 112,720
2	Structural Concrete (Left Abutment Rock Knob /MSE Wall Replacement)	180	CY	$700.00	$ 126,000
3	Filter Sand	1,125	CY	$90.00	$ 101,250
4	Drain Aggregate	1,752	CY	$80.00	$ 140,160
5	24-inch Perforated HDPE Pipe	270	LF	$90.00	$ 24,300
6	Low Permeability Fill	2,000	CY	$50.00	$ 100,000
SUBTOTAL					**$ 605,000**
Unlisted Items 15%					$ 91,000
Mobilization, Demobilization & Preparatory Work 10%					$ 61,000
SUBTOTAL					**$ 757,000**
Contingency 30%					$ 228,000
CONSTRUCTION TOTAL					**$ 985,000**
Engineering 10%					$ 98,500.0
Construction Engineering 15%					$ 147,750.00
Permitting 5%					$ 49,250.00
OPINION OF PROBABLE PROJECT COST					**$ 1,280,500**

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Element 4C - Raised Crest Wall

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	8,683	CY	$10.00	$ 86,830
2	Structural Concrete (Left Abutment Rock Knob /MSE Wall Replacement)	752	CY	$700.00	$ 526,400
3	Fill (onsite)	8,851	CY	$30.00	$ 265,530
SUBTOTAL					**$ 879,000**
Unlisted Items 15%					$ 132,000
Mobilization, Demobilization & Preparatory Work 10%					$ 88,000
SUBTOTAL					**$ 1,099,000**
Contingency 30%					$ 330,000
CONSTRUCTION TOTAL					**$ 1,429,000**
Engineering 10%					$ 142,900
Construction Engineering 15%					$ 214,350
Permitting 5%					$ 71,450
OPINION OF PROBABLE PROJECT COST					**$ 1,857,700**

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Element 5 - 1914 Landslide Area Filter and Drain System

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	4,300	CY	$ 12.00	$ 51,600
2	Filter Sand	750	CY	$ 90.00	$ 67,500
3	Drain Aggregate	750	CY	$ 80.00	$ 60,000
4	Geotextile	7,500	SF	$ 0.90	$ 6,750
5	Solid HDPE Pipe	40	LF	$ 100.00	$ 4,000
6	Perforated HDPE Pipe	60	LF	$ 150.00	$ 9,000
7	Fill (Onsite)	2,800	CY	$ 30.00	$ 84,000
8	Concrete Weir Structure	1	LS	$ 10,000.00	$ 10,000
9	Access Road	1	LS	$ 75,000.00	$ 75,000
SUBTOTAL					**$ 368,000**
Unlisted Items 15%					$ 56,000
Mobilization, Demobilization & Preparatory Work 10%					$ 37,000
SUBTOTAL					**$ 461,000**
Contingency 30%					$ 139,000
CONSTRUCTION TOTAL					**$ 600,000**
Engineering					$ 50,000
Construction Engineering					$ 50,000
Permitting					$ 25,000
OPINION OF PROBABLE PROJECT COST					**$ 725,000**

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Beaver Park Dam
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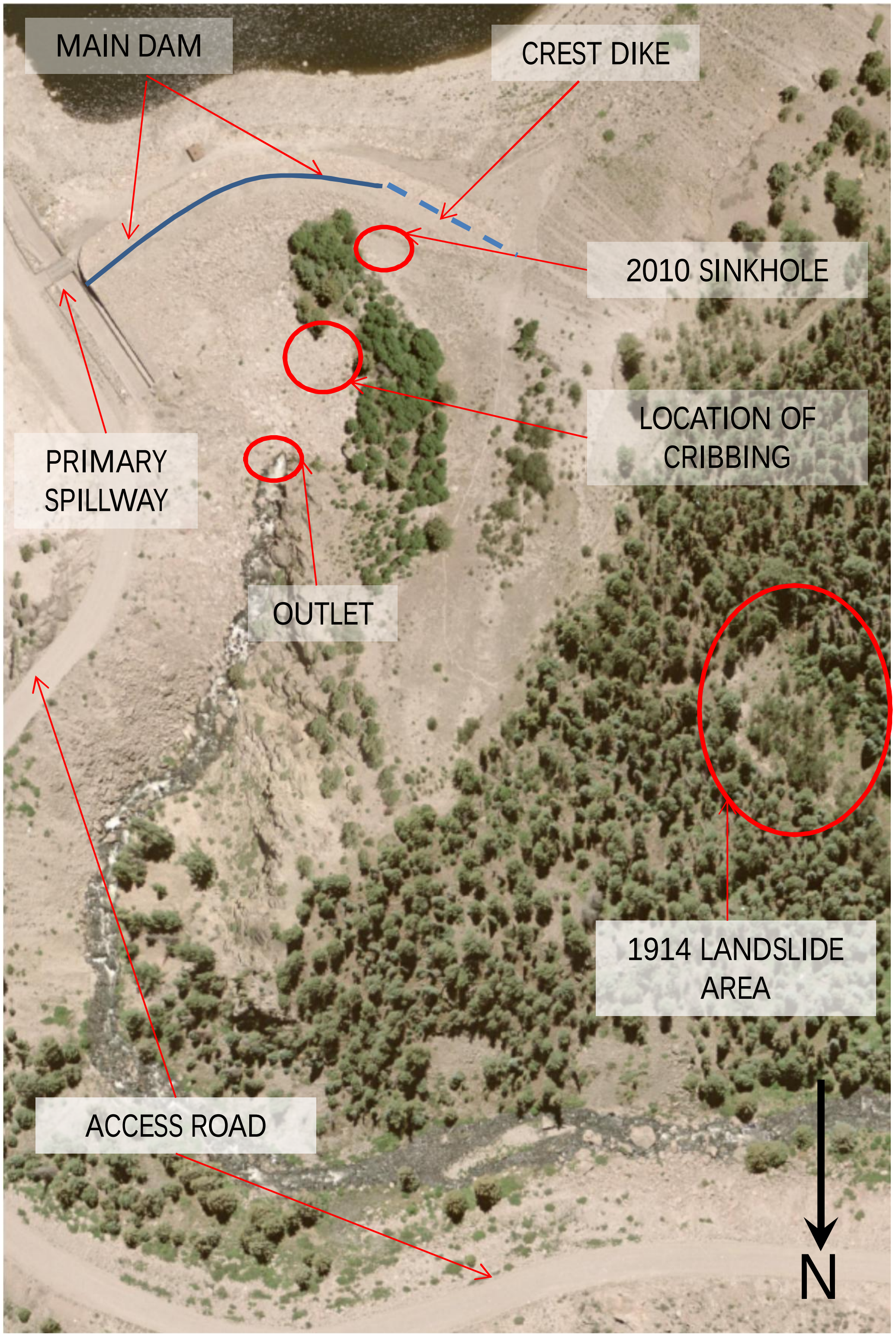
Alternative 2 - New Roller Compacted Concrete Dam

ITEM NO.	ITEM DESCRIPTION	QUANTITY	UNIT	UNIT COST	COST
1	Excavation	5,000	CY	$ 12.00	$ 60,000
2	Dental Concrete	600	CY	$ 350.00	$ 210,000
3	Structural Concrete	2,090	CY	$ 700.00	$ 1,463,000
4	Facing Concrete	1,960	CY	$ 400.00	$ 784,000
5	Rock Anchors	2,340	LF	$ 75.00	$ 175,500
6	Grout Curtain	2,600	LF	$ 160.00	$ 416,000
7	Roller Compacted Concrete	35,200	CY	$ 140.00	$ 4,928,000
8	60-inch Steel Outlet Conduit	120	LF	$ 670.00	$ 80,400
9	30-inch manhole access	1	LS	$ 25,000.00	$ 25,000
10	60-inch X 60-inch Sluice Gate	3	LS	$ 100,000.00	$ 300,000
11	48-inch slide gate	1	LS	$ 140,000.00	$ 140,000
12	Demolition of Existing Dam	1	LS	$ 250,000.00	$ 250,000
13	Diversion and Dewatering Allowance	1	LS	$ 250,000.00	$ 250,000
14	Left Abutment Seepage Control Measures Allowance	1	LS	$10,000,000.00	$ 10,000,000
15	County Road 20 Relocation Allowance	1	LS	$ 750,000.00	$ 750,000
SUBTOTAL					**$ 19,832,000**
Unlisted Items 15%					$ 2,975,000
Mobilization, Demobilization & Preparatory Work 10%					$ 1,984,000
SUBTOTAL					**$ 24,791,000**
Contingency 30%					$ 7,438,000
CONSTRUCTION TOTAL					**$ 32,229,000**
Engineering 10%					$ 3,222,900
Construction Engineering 15%					$ 4,834,350
Permitting 15%					$ 4,834,350
OPINION OF PROBABLE PROJECT COST					**$ 45,120,600**

Appendix E

General Site Photographs



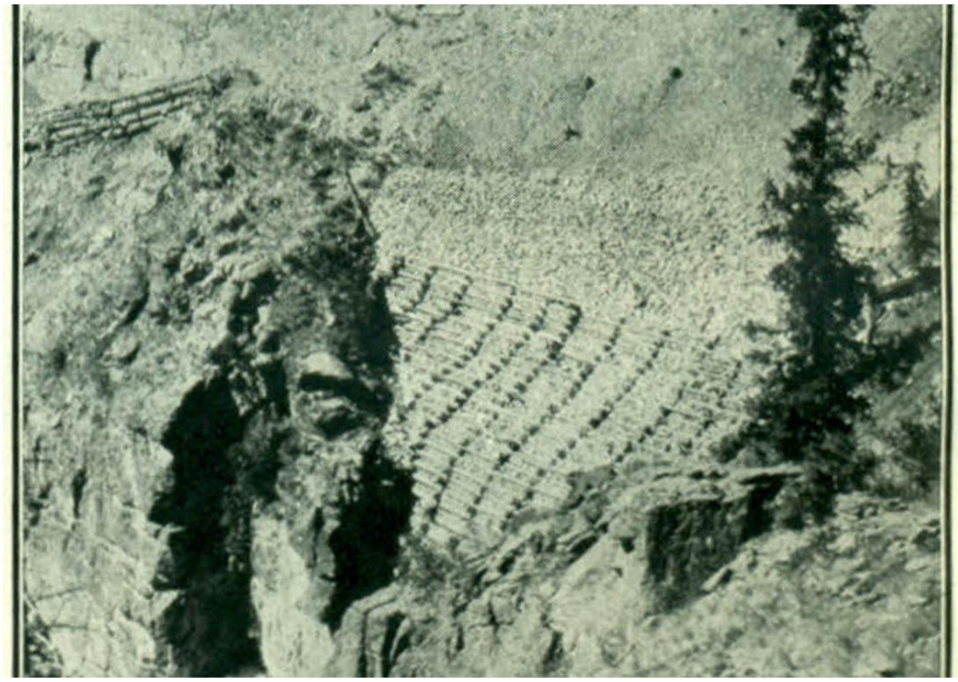
Photograph 1: General plan view of the pertinent features of Beaver Park Dam and surrounding area.

Photograph No. 2 View from the right abutment looking downstream along the primary spillway chute.

Photograph No. 3 View from the access road looking upstream at the downstream end of the primary spillway chute.

Photograph No. 4 View from the right reservoir rim looking downstream at the dam embankment.

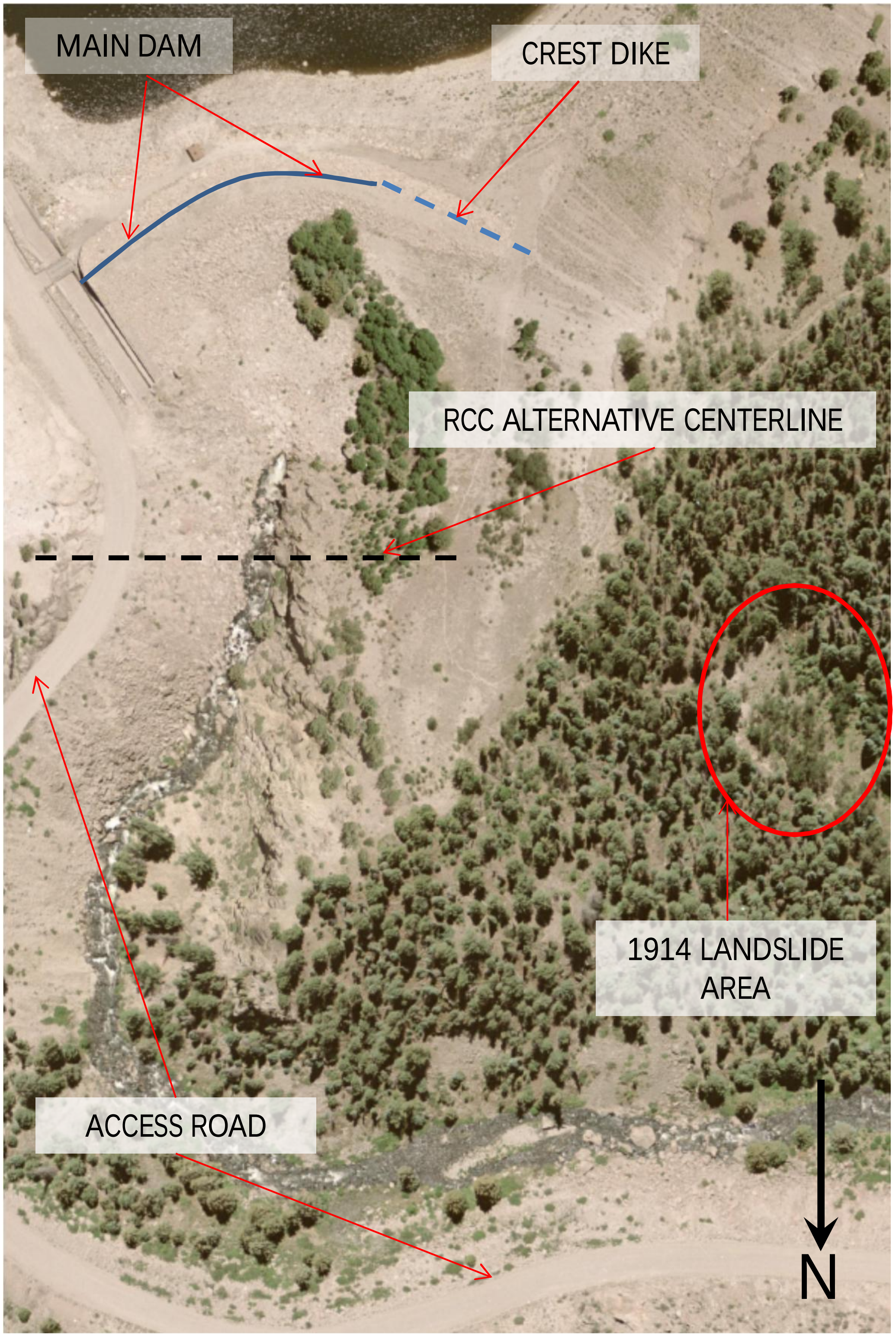
Photograph No. 5 View from existing dam crest looking downstream at the potential location of the RCC Dam alternative. Note existing access road would need to be relocated as part of the RCC dam Alternative.



Photograph No. 6 View of downstream slope cribbing. Cribbing was constructed between 1915 and 1916. Cribbing is not visible today.

Photograph No. 7 View from the left abutment looking right toward the right abutment and primary spillway along the crest dike. Note approximate location of 2010 sinkhole.

Photograph No. 8 View from crest dike looking downstream at the 2010 sinkhole.



Photograph No. 9 General plan view of the approximate location of the RCC Dam alternative.

Appendix F

Scope Of Work

December 16, 2010

Ed Frazar, P.E.
Chief Engineer
Colorado Division of Wildlife
Department of Natural Resources
6060 Broadway
Denver, CO 80216-1000

Subject: Proposal for Alternatives Study for Beaver Park Dam and Reservoir, Near South Fork, Colorado. URS Project Number 22241120.

Dear Ed:

We are pleased to submit this proposal for an alternatives study for Beaver Park Dam and Reservoir near South Fork, Colorado. The purpose of the study will be to develop conceptual level designs and costs to address dam safety issues associated with the dam foundation, spillway erosion and capacity, and outlet works reliability. The study will include alternatives for reestablishing the reservoir to a historical operating level at three feet above the service spillway crest. We understand results of the alternatives study will be used to aid the Colorado Division of Wildlife (DOW) with the alternatives selection.

We propose to prepare conceptual designs to evaluate two alternatives to improve the existing embankment: a *drain and filter system* located downstream of the embankment in the "window" area, and *a cut off wall* located in the left abutment of the existing dam. We will also evaluate a new roller compacted concrete (RCC) dam located downstream of the existing embankment dam. Two spillway alternatives described in this proposal will be evaluated to address the erosion downstream of the existing service spillway chute. The study will also evaluate the adequacy of the auxiliary spillway to safely pass an inflow design flood. Two outlet works alternatives will be evaluated, including a new tunnel outlet works located in the right abutment of the dam, and an upgrade of the existing outlet works. By developing alternatives that include schedule and cost for each of these features separately, a final project configuration can be selected by DOW from a combination of the alternatives. Workshops will be held to ensure DOW involvement at the critical project milestones. We will also include a "do nothing" alternative which will not modify the existing dam but will maintain a storage level below the existing spillway. The following scope of work is divided into ten tasks and is based on discussions we had with Bill McCormick and Hank Koopman in our office on August 17, 2010.

URS Corporation
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Denver, CO 80237
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Offices Worldwide

TASK 1 – TOPOGRAPHIC BASE MAP

We propose to obtain a topographic base map for use in conceptual design of the alternatives. The topography would be developed from LiDAR survey and have a contour interval of two feet. The area of coverage would include the dam site, the area of springs in the moraine downstream of the dam site, and the reservoir. The topography base map would also be suitable for design of the selected alternative. Aerial photo coverage of the reservoir has been included at a small incremental cost. We currently have a topographic map with a five-foot contour interval, made with data obtained from the USGS web site that we have processed using GIS methods. We plan to use this base map in the interim, until the new topography base map is available. Refer to attachment for a survey and mapping scope of work for this task.

TASK 2 – GEOLOGIC MAPPING

We have conducted preliminary geologic mapping (as part of our current contract), using the existing USGS topographic base map. This map locates and describes geologic and geotechnical features, including the bedrock, glacial deposits, alluvium, landslides, springs, erosion gullies, etc. We will transfer information from the existing map to the new topographic base map when the new map becomes available.

Task Leader - Dale Baures, P.E., P.G.

TASK 3 – SEISMIC REFRACTION

We propose to conduct a geophysical investigation using seismic refraction methods to provide an estimate of depth of glacial and alluvial materials within the paleo-valley in the left abutment and downstream of the dam site. We propose to conduct four seismic refraction lines at the approximate locations shown on Figure 1. The survey lines would extend from areas where bedrock is exposed on or near the surface, into the paleo-valley, and be capable of estimating depth to bedrock of approximately 250 feet. We plan to use a 24-channel instrument for the measurements and explosives (ANFO) for an energy source. We plan to subcontract with a company licensed to conduct seismic-related blasting in Colorado.

Task Leader - Dale Baures, P.E., P.G.
Seismologist - John Nicholl, P.G.

TASK 4 – HYDROLOGY/HYDRAULICS

We understand that the DOW will provide URS with flood hydrology prepared for Beaver Creek Dam in the mid 1990s. URS will review the DOW hydrology and will recommend a preliminary flood hydrograph to be used for the spillway design, based upon current State Engineer's Office (SEO) requirements. Preliminary PMF flood hydrology and frequency flood hydrology will be estimated for preliminary flood routings of the spillway alternatives.

Task Leader – Greg Glunz, P.E.

TASK 5 – RISK ANALYSIS

The existing potential failure modes analysis and Risk Analysis Report will be updated to include selected alternative designs. The results of the risk analysis for the alternative embankment and spillway designs will be presented and discussed at the Concept Design Workshop. Alternatives that do not sufficiently reduce risk will be eliminated from the study. The risk analysis will be conducted in an informal meeting format.

Task Leader – John France, P.E. (assisted by John Smart, P.E. and Jennifer Williams, P.E.)

TASK 6 – ALTERNATIVES ANALYSIS

Task 6.1 – Embankment Alternatives

We propose to evaluate two alternatives to address the dam safety issues associated with piping of the existing dam and foundation. The alternative embankment designs are described as follows:

- Drain and Filter System - The drain and filter system is currently envisioned to be located in the area composed of glacial moraine and outwash within the "window" through bedrock exposed downstream of the left abutment. We expect this alternative will include excavating rock fill material at the toe of the dam to expose the glacial materials, flattening the slope composed of glacial materials, installing a drain and drain monitoring features, creating a tie-in of the drain to the existing drainage adit or tunnel, constructing a two or three layer filter over the glacial materials, and rebuilding the rock fill berm and embankment.

- Cut-Off Wall - The cut-off wall alternative would include an evaluation of a deep soil-cement cut-off wall located near the geophysical survey line A-A' shown on Figure 1 and a relatively shallow concrete cut-off wall located near line B-B' shown on Figure 1.

Design criteria for the conceptual design of the embankment, filter and drainage system, and the cut-off wall will be included in this section of the Alternatives Report.

Task Leader – John Smart, P.E.

Task 6.2 – Spillway Alternatives

Two spillway alternatives are proposed to address the erosion at the downstream end of the service spillway chute. The two spillway alternatives to be evaluated are described as follows:

- The first alternative will be to construct a stepped spillway from the downstream end of the existing concrete chute, to the eroded rock surface below the end of the chute. This alternative will also include buttressing the existing rock fill to a flatter, more stable slope.

- The second alternative will extend the existing concrete-lined chute to the river channel. This alternative will include an extended spillway chute and new stilling basin.

Both alternatives will include use of the existing side channel service spillway structure and will include review of the capacity to pass the preliminary flood hydrograph. If necessary, the conceptual design will include modifications to the auxiliary spillway to safely pass the preliminary flood hydrograph.

Design criteria for the conceptual design of the spillway will be included in this section of the Alternatives Report.

Task Leader – Steve Higinbotham, P.E.

Task 6.3 – Outlet Works Alternatives

Two outlet works alternatives will be evaluated to upgrade and improve the reliability of the existing outlet works. The level of effort for these designs will be minimized and based upon URS' experience with designing similar projects.

- Rehabilitation of the Existing Outlet Works – This alternative will replace the existing gate stems with stainless steel stems, and will replace the existing gate operators with modern electric motor operators. The existing cast iron slide gates will remain in place because of the difficulty in their removal and replacement, and because the cast iron slide gates do not corrode with time. We will also evaluate methods of improving the reliability of the downstream steel liner pipe. This work can be accomplished without draining the reservoir.

Although access to the upstream gate stem will be difficult, we understand the DOW has inspected (and made repairs) to the upstream gate stem using divers.

- New Tunnel Outlet Works - This alternative will include a new, steel-lined tunnel outlet works constructed through the right abutment of the dam. (URS recently designed a similar tunnel outlet works at Carter Lake Dam. The reservoir was drained to construct the upstream portion of the new outlet works. This work was accomplished in late fall and winter.)

Design criteria for the conceptual design of the outlet works will be included in this section of the Alternatives Report.

Task Leader - Steve Higinbotham, P.E.

Task 6.4 – New RCC Dam

This alternative will include a new RCC dam. The dam will be located downstream of the existing dam and in the bedrock-walled canyon. Our first task for this alternative will be to conduct a geological fatal flaw analysis for a new dam at this site, due to the geologic conditions within the left abutment. If the RCC dam is found to have a fatal flaw, then this alternative will be dropped with limited engineering effort.

The RCC dam will include a converging stepped spillway, intake tower attached to the upstream face of the dam, and a steel-lined outlet pipe and a valve house located at the downstream toe of the dam. We expect that the RCC dam alternative will require a drain and filter system in the downstream spring area of the glacial deposits and/or a cut-off wall in the left abutment.

Design criteria for the conceptual design of the RCC Dam alternative will be included in this section of the Alternatives Report.

Task Leader – Sal Todaro, P.E.

Task 6.5 – Workshop Meetings

A total of three workshop meetings for project staff are proposed: a Concept Design Workshop, an Interim Design Workshop, and an Alternatives Selection Workshop. The *Concept Design Workshop* will be conducted early during conceptual design of the project to finalize details of the alternatives. (The Concept Design Workshop will include a brief summary of the risk analysis of the alternatives proposed for the embankment and the spillway, so that alternatives that do not sufficiently reduce risk are not brought forward in the study.)

The *Interim Design Workshop* will be conducted when conceptual design is nearing completion, to review the conceptual design and generate information to finalize the conceptual design. *The Alternatives Selection Workshop* will be held after submittal of the Draft Alternatives Report, to solicit input from project personnel for finalizing the Alternatives Report.

Evaluation criteria to be used for evaluation of the alternatives will include: dam safety, risk reduction, water storage economics, and operation and maintenance. All workshops will be held at URS' DTC Office. Meeting minutes will be prepared to document all workshops.

Task Leader – Sal Todaro, P.E.

TASK 7 – DRAWINGS

The conceptual design drawings will be the basis for the construction cost estimates and will be a primary deliverable of the alternatives study. All drawings will be prepared under the direction and review of senior dam engineers experienced with the design of similar structures. The drawings will be prepared using AutoCAD 2010. An annotated list of drawings arranged by project features is included as Table 2.

Task Leader – Sal Todaro, P.E. / Craig Wiltshire, E.I.T.

TASK 8 – CONSTRUCTION SCHEDULE AND COST ESTIMATE

We propose to develop a construction schedule and cost estimate for each alternative, based on the conceptual level design. Results will be presented in tabular format to aid DOW during the alternatives selection process. Construction quantities will be estimated, based upon the conceptual design drawings and updated topography.

Task Leader – Jeff Allen, P.E.

TASK 9 – ALTERNATIVES REPORT

An Alternatives Report will be prepared to present information, analyses, conceptual design drawings, cost estimates, construction schedule, and results for the alternatives study. Design criteria for the conceptual design of the Embankment, Spillway and Outlet Works will be included in the relevant sections of the Alternatives Report. At this time, we envision the report to be organized according to the following outline:

1. Executive Summary
2. Geological Engineering (Geological conditions and impacts of local geology on project design requirements)
3. Results of Seismic Refraction Study
4. Flood Hydrology and Spillway Hydraulics
5. Embankment Alternatives (to address potential piping through dam and abutments)
6. Evaluation of Upper Embankment (to address piping of upper embankment during floods)
7. Spillway Alternatives
8. Outlet Works Alternatives
9. New RCC Dam
10. Drawings (Refer to attached Drawing List)
11. Risk Analysis
12. Alternative Selection (Based upon results of the Alternatives Selection Workshop)
13. Conclusion and Recommendations

Task Manager: Sal Todaro, P.E.

TASK 10 – PROJECT MANAGEMENT

This task will include Project Manager, Sal Todaro's oversight and direction of the study, as well as tracking of the project schedule and budget.

Schedule and Cost

Because the Notice to Proceed will occur during the winter, the LiDAR survey will be delayed until snow has melted from the site. We therefore plan to conduct the survey in mid-April and complete the topographic mapping in mid-May 2011. The schedule of key project milestones are listed below:

Hydrology and Hydraulics	Week of March 28
Preliminary Alternative Sketches (for Risk Workshop)	Week of April 25
Risk Workshop	Week of May 2

Mapping Complete	Week of May 16
Interim Design Workshop	Week of July 18
Alternatives Selection Workshop	Week of August 22
Draft Report	Week of September 19
Final Report	Week of October 17

The estimated cost of the work is $267,581.00 based upon the cost estimate in attached Table 1.Thank you for the opportunity to continue to help you with this important project. Please call should you have questions.

Sincerely,

Sal Todaro, P.E.
Project Manager

John W. France, P.E.
Principal in Charge

cc: Bill McCormick, P.E., P.G., Dam Operations Engineer

Attachments: Table 1 Cost Summary
Table 2 Drawing List
Figure 1 Proposed Seismic Refraction Locations
Figure 2 Organization Chart
Survey and Mapping Scope