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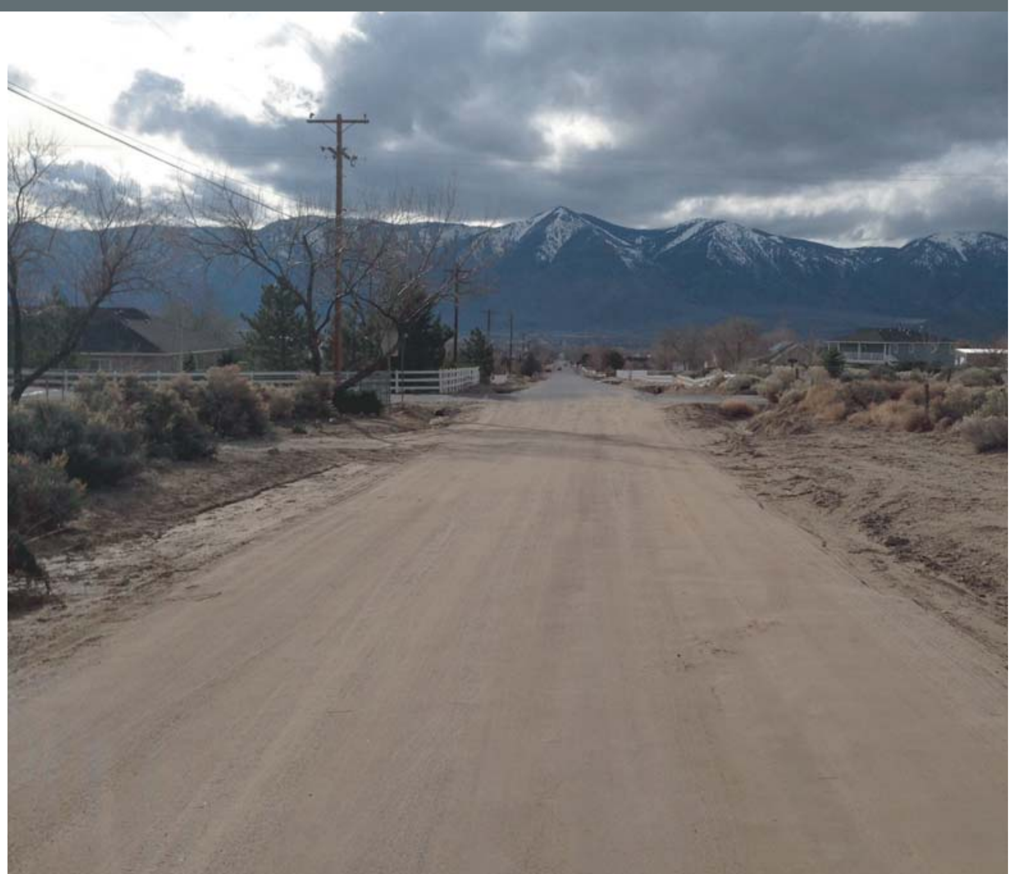
Report

Stephanie Way Flood Control Project
Douglas County, Nevada

Feasibility Engineering Study
for
CARSON WATER SUBCONSERVANCY DISTRICT

Final Report
May 27, 2016



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Douglas County, Nevada

Feasibility Engineering Study

Final Report

May 27, 2016

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1 Background and Introduction

The project area is a relatively small, unmapped watershed located immediately easterly of the east end of Stephanie Way, Douglas County, NV, inclusive of the power substation in this area (Figure 1 – Project Location Map). The contributing watershed of this project area is about 0.65 square miles situated between the Buckbrush Wash and Johnson Lane Wash watersheds

The effective Flood Insurance Rate Maps (FIRMs) issued by the Federal Emergency Management Agency (FEMA), dated January 20, 2010, designate portions of the project area as generally being within shaded Zone 'X' (0.2-percent annual chance flood) and unshaded Zone 'X' (area of minimal flood hazard). Figure 2 – Effective FEMA FIRM depicts the extent of the floodplain boundaries covering the project area and downstream areas. FEMA recently published a revised FIRM for this area having a future date of June 15, 2016. This relationship of the proposed floodplain boundaries and the project site are shown on Figure 3 – Preliminary FEMA FIRM.

These designations suggest that the flooding potential of these areas is expected to be low to moderate. However, this neighborhood has experienced repetitive flooding including heavy sediment deposition in the past. Flood events in 2014 and 2015 resulted in considerable damage to the residential properties and public infrastructure in this area, which has been attributed to this watershed. Douglas County incurred more than $2.3[1] million in cleanup costs resulting from flood-related damages from 2015 flash floods in Johnson Lane and Stephanie Way areas alone.

Consequently, Douglas County partnered with the Carson Water Subconservancy District (CWSD) to explore the feasibility of constructing a flood control facility on BLM property east of Romero Drive to alleviate flood-induced recurring damages in this neighborhood. The following specific tasks were included in the scope of services:

- Collect available topographic data for the study area from U.S. Geological Survey (USGS) National Map; Perform field surveys if needed and construct a work map;

[1] Personal communication from Erik Nilssen, P.E., Douglas County Engineer.

- Delineate contributing watershed boundary for the Stephanie Lane Wash and perform hydrologic modeling to estimate runoff characteristics for the 50-, 10-, 4-, 1-, 0.2-percent annual chance precipitation, and ½ PMP events;
- Locate appropriate location for the proposed flood control reservoir that has minimum impacts on the existing infrastructure;
- Size the flood control reservoir and associated outlet works to detain and attenuate flood flows resulting from the above-mentioned precipitation events, and limit outflow from the reservoir during the occurrence of 1-percent annual chance flood to that of existing 10-percent annual chance peak flow at a minimum;
- Estimate required channel section downstream of the proposed reservoir to safely carry expected outflow from the reservoir;
- Perform earthwork calculations, develop engineer's estimate of probable costs to design, permit and ultimately construct the embankment structure, outlet works and other necessary appurtenances;
- Perform Benefit-Cost Analysis (BCA) conforming to FEMA standards;
- Prepare a draft report with supporting exhibits for CWSD's, and other public agencies' (stakeholders) review and comment;
- Participate in and present the results of this study at the Carson River Coalition River Corridor Working Group Meeting and one general public meeting; and
- Address comments and feedback received from stakeholders and the public and finalize the report.

Section 2 of this report describes criteria used to develop hydrologic model, and also presents the results of hydrologic modeling. Section 3 of the report includes results of hydraulic calculations performed. Section 4 of the report includes a detailed discussion of the basis of design, along with the presentation of the engineer's estimate of probable construction costs for this flood control facility. Section 5 of the report contains the findings and conclusions of this study.

Figure 1 – Project Location Map

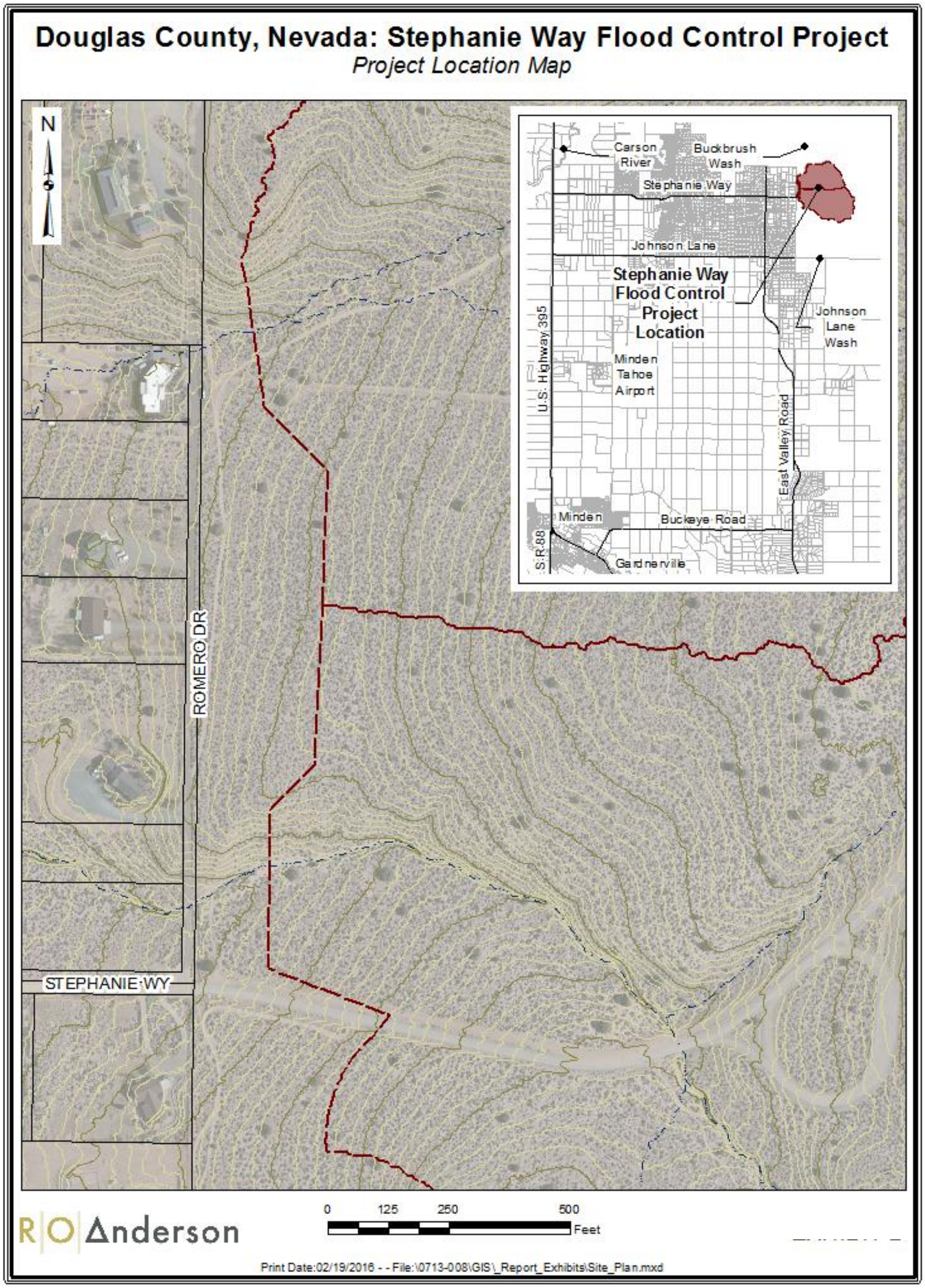


Figure 2 – Effective FEMA FIRM

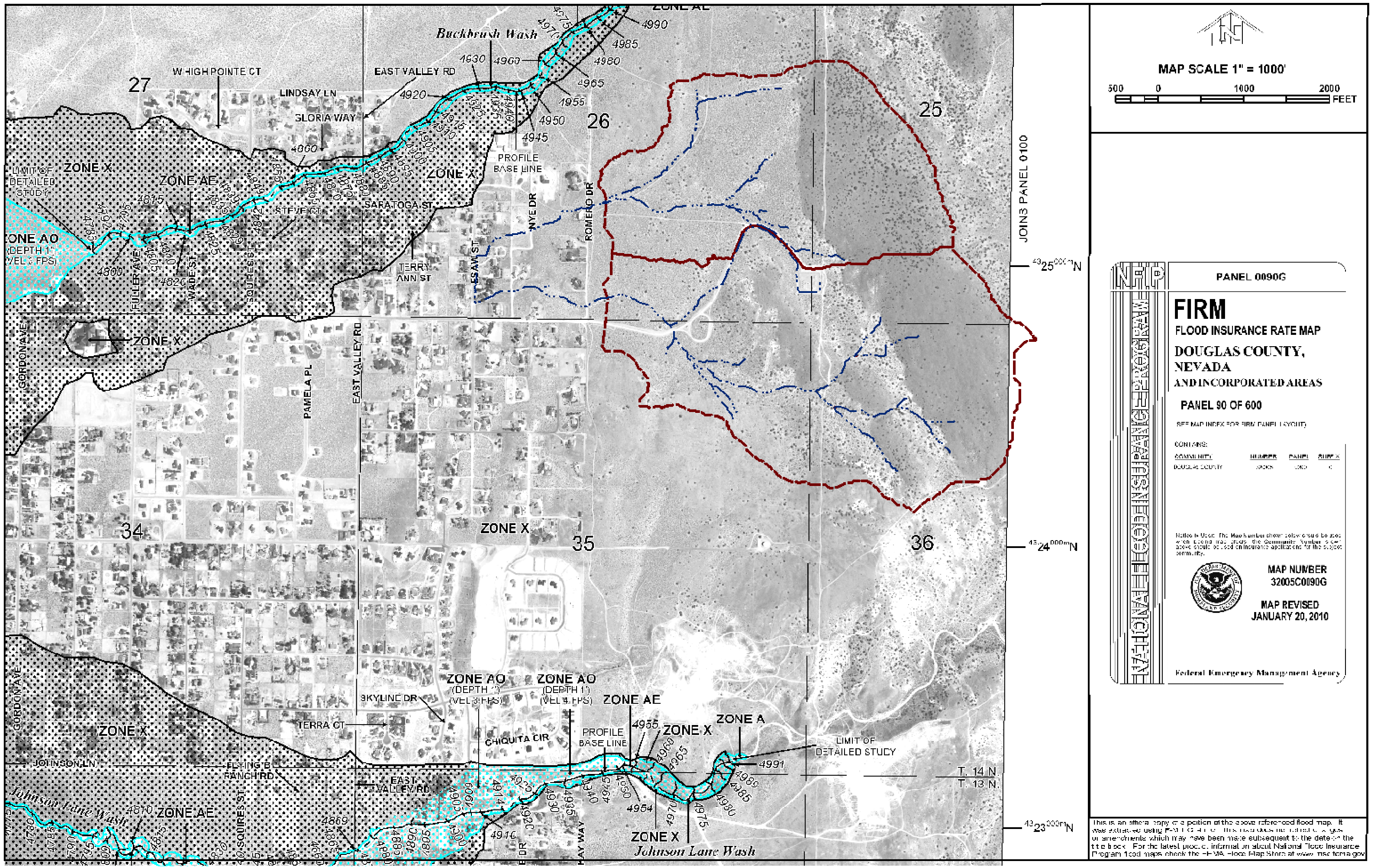
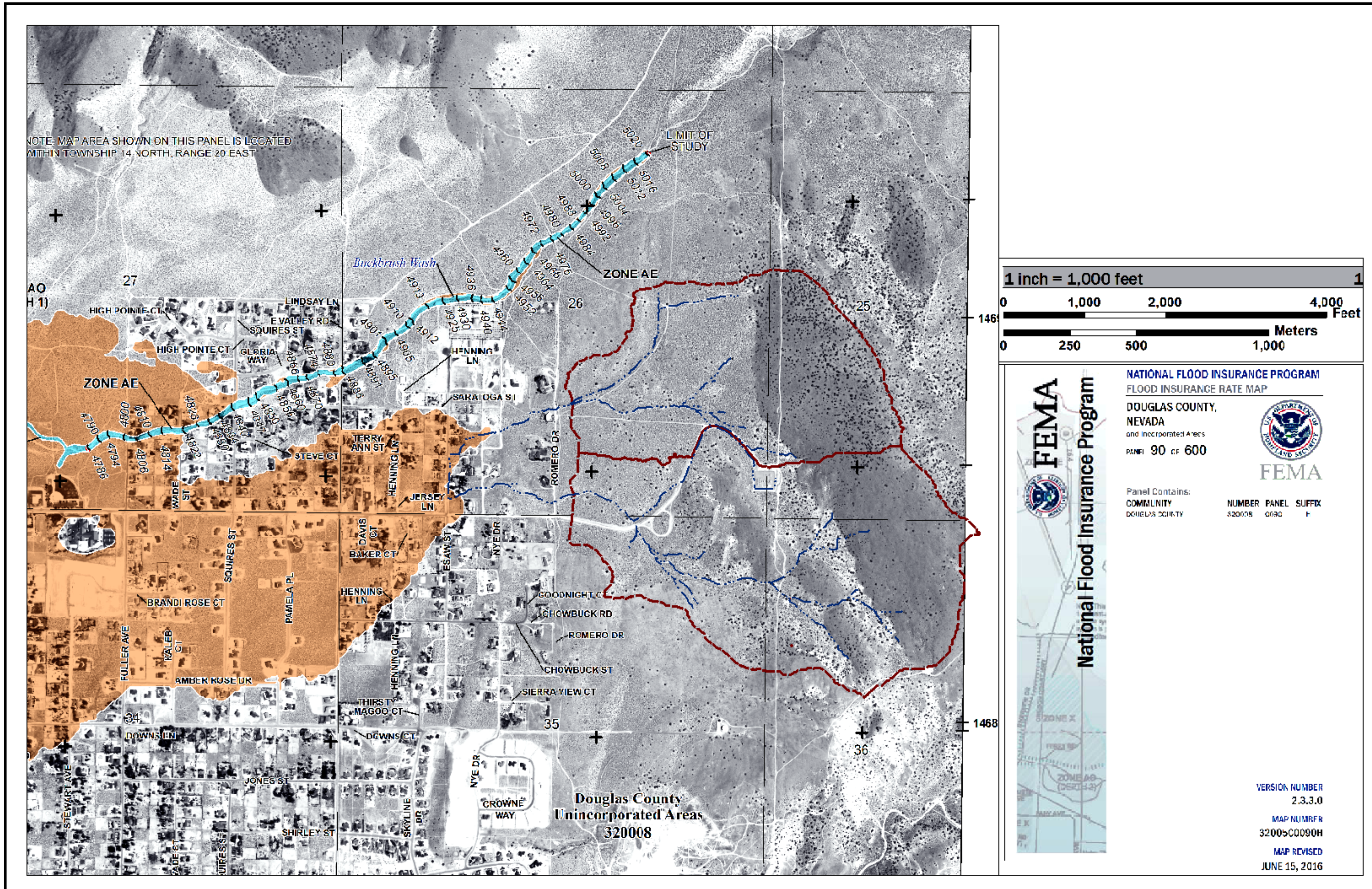


Figure 3 – Preliminary FEMA FIRM



2 Hydrologic Modeling

This section describes procedures and methodology used for the development of watershed model using the U.S. Army Corps of Engineers' (USACE) Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS V 4.0) software. HEC-HMS is the next generation Windows version of the popular HEC-1 program, developed by the USACE. It is capable of modeling various catchments' components such as infiltration / evapotranspiration losses, runoff transformations, and a variety of open channel routing methods. HEC-HMS method provides both peak flow and the total volume of runoff and is appropriate method to use when modeling large watersheds that include large conveyance facilities and storage facilities. The following precipitation return interval events were used while preparing the hydrologic modeling.

- 50-percent annual chance of exceedance (2-year event)
- 10-percent annual chance of exceedance (10-year event)
- 4-percent annual chance of exceedance (25-year event)
- 1-percent annual chance of exceedance (100-year event)
- 0.2-percent-annual-chance of exceedance (500-year event)
- ½-Probable Maximum Precipitation (½-PMP)

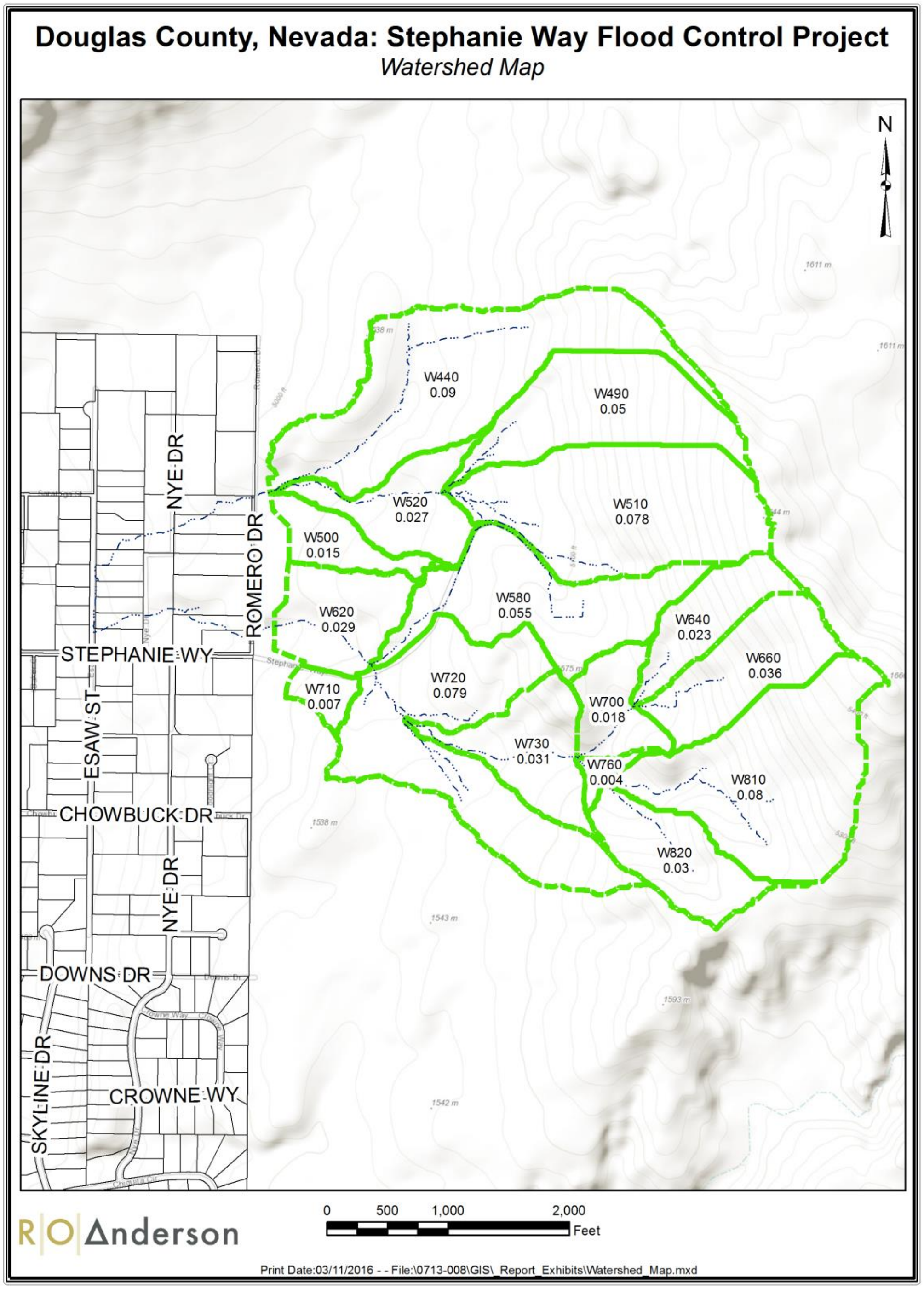
2.1 HEC-HMS Model Setup

The first step in the development of a hydrologic model is to delineate the contributing watershed boundary. A DEM was created from the topographic data and HEC-GeoHMS tools were used in ESRI's ArcGIS environment to delineate contributing watershed. The total drainage area of the contributing watershed is approximately 0.65 square miles.

To perform detailed hydrologic analyses of the study area, the 0.65 square mile drainage area was subdivided into fifteen sub-basins based on distinct topographic characteristics. The runoff from these sub-basins is routed downstream, and the flow is added at the junction of sub-basins as shown in Figure 4 – Watershed Map.

Once the sub-basins were delineated, the next step in the development of the hydrologic model was to estimate the parameters used to build the components of the model. After sub-basin delineation, ArcGIS and HEC-GeoHMS were used to develop modeling input parameters and develop the connectivity schematic for the HEC-HMS model.

Figure 4 – Watershed Map



A HEC-HMS hydrologic model consists of three basic components:

- *A Basin Model*, consisting of a physical representation of watersheds;
- *A Meteorologic Model*, consisting of precipitation, evapotranspiration, and snowmelt data; and
- *A Control Specification*, consisting of information such as hydrologic simulations time span.

2.1.A Basin Model:

In order to estimate excess runoff generated from any particular precipitation event the following input information is entered in the Basin Model of HEC-HMS:

- Loss Rate Parameters;
- Transformation Parameters;
- Base flow Parameters;
- Reach Parameters; and
- Reservoir Parameters, if detention/retention ponds are being modeled.

An assortment of different methods is available in HEC-HMS to physically represent these parameters. The following methodologies were used in developing the hydrologic model for the Stephanie Way watershed:

- Loss Rate: Green-Ampt Method;
- Transformation: Snyder Unit Hydrograph Method;
- Reach Routing: Muskingum-Cunge Method; and
- Reservoir Routing: Outflow Structures.

A detailed description of estimation methods to develop these model parameters are discussed in the subsequent sections of this report.

For this basin, base flow is assumed to be negligible and, therefore, not taken into account in developing these hydrologic models. The other model parameter estimation is described in the subsequent sections of this report.

2.1.A.1 Loss Rate Parameters

Watershed loss or abstraction is a term used to describe the collective precipitation losses throughout the watershed that occur during a storm. These losses play a significant role in rainfall-runoff modeling as they determine the amount of rainfall excess, or direct runoff, produced by the storm within the model. Typical losses abstracted from rainfall include:

- Soil infiltration;
- Landscape interception;
- Depression storage (aka: surface storage);
- Evaporation; and
- Evapotranspiration.

The rainfall volume attributable to these losses is not converted to direct runoff. For this study losses such as evaporation, landscape, interception and evapotranspiration by vegetation are considered minor and were not included.

Depression storage, or initial loss, in a sub-basin is the process by which precipitation is abstracted by being retained in puddles, ditches, interception, and other natural or artificial depressions on the land surface. The water either evaporates or eventually contributes to soil moisture by infiltration. Depression storage, in inches over the sub-basin area or computational cell, is subtracted from rainfall and reduces the contribution to runoff. Land use characteristics are used to help quantify estimates of depression storage.

Infiltration is the process by which precipitation is abstracted by seeping into the soil below the land surface. Soil infiltration was estimated using the Green-Ampt method. The Green - Ampt method applies Darcy's law and principle of conservation of mass to estimate infiltration. The method works under the assumption that water enters the soil as a sharp, vertical wetting front that travels as a function of the hydraulic conductivity.

The Green-Ampt infiltration function (in rate form) is

$$f = K_s\left(1 + \frac{\Psi\theta}{F}\right)$$

Where f is the infiltration rate (capacity, L/T), F is the cumulative infiltration (L), K_s is the saturated hydraulic conductivity (L/T), Ψ is the soil suction at the wetting front (L), and θ is the dimensionless soil moisture deficit of the soil at the beginning of the storm.

Parameters (K_s, Ψ, θ) were determined using the protocol defined by Maricopa County, Arizona (Engineering Division, Flood Control District of Maricopa County, 2010). The basic approach is to estimate a weighted saturated hydraulic conductivity by computing the area-weighted mean logarithm (equivalent to computing the area-weighted geometric mean) and then using that value to enter the table in the Maricopa County manual to choose the suction (Ψ) and soil moisture deficit (θ) parameters. Table 1 – Weighted-Average Green-Ampt Parameters below shows a summary of Green-Ampt parameters calculated for each sub-basin: Figure 5 – Soils Map shows NRCS soils overlaid on the sub watersheds of Stephanie Way watershed.

Table 1 – Weighted-Average Green-Ampt Parameters

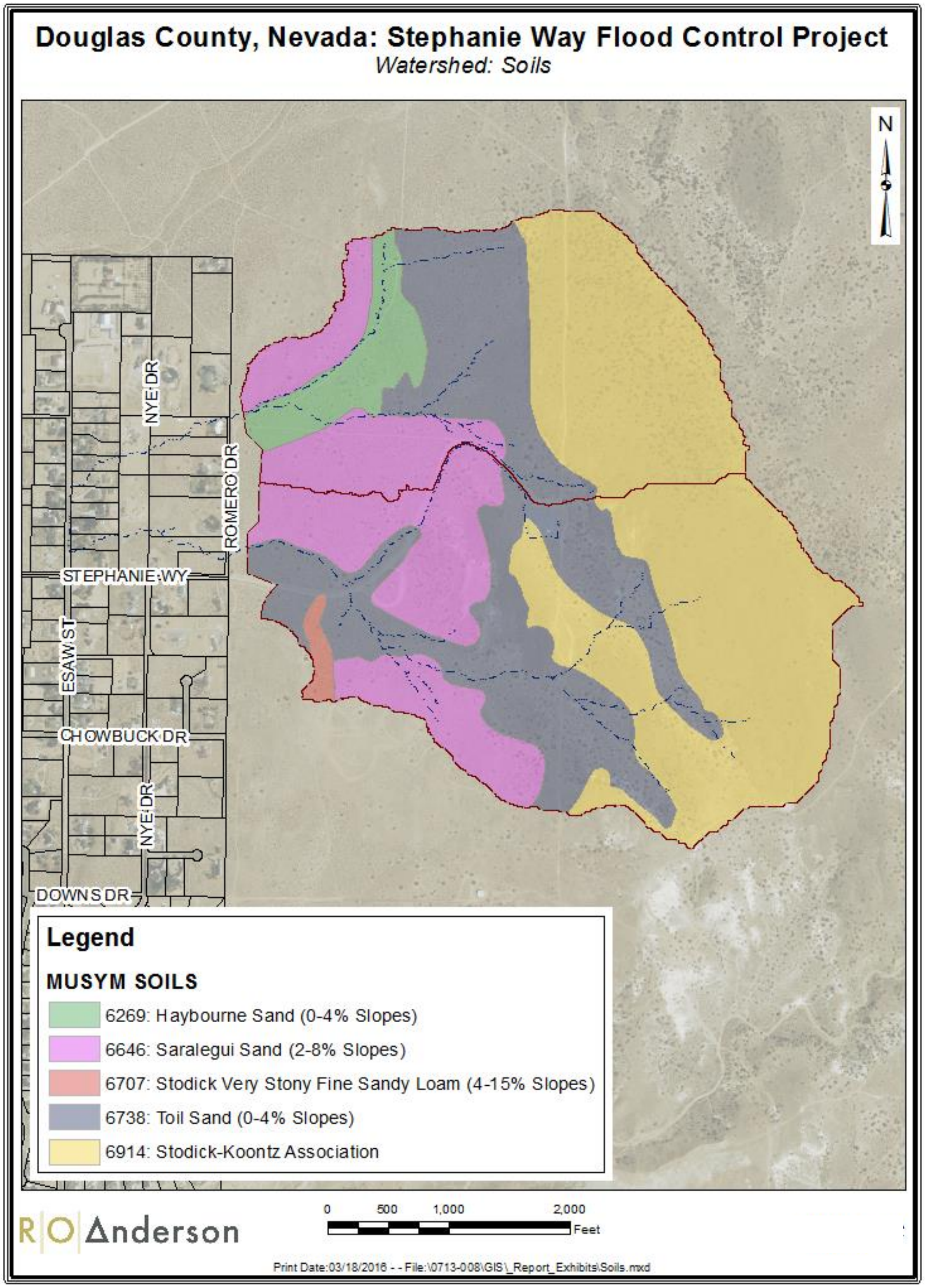
Subwatershed	K_s (in/hr)	Ψ (in)	θ[1]
W440	0.33	5.16	0.33
W490	0.15	7.43	0.28
W500	0.40	4.30	0.35
W510	0.16	7.30	0.28
W520	0.40	4.30	0.35
W580	0.32	5.32	0.33
W620	0.40	4.30	0.35
W640	0.12	7.80	0.27
W660	0.10	8.05	0.26
W700	0.16	7.31	0.28
W710	0.34	5.05	0.33
W720	0.37	4.63	0.34
W730	0.38	4.55	0.34
W760	0.34	5.09	0.33
W810	0.21	6.70	0.29
W820	0.21	6.70	0.29

[1]Dimensionless Parameter

2.1.A.2 Transformation Parameters

Rainfall transformation, as it relates to rainfall-runoff modeling, refers to the process of converting excess rainfall into storm-water runoff – typically in the form of a runoff hydrograph. HEC-HMS has a total of eight different transform methods available. The choices include various unit hydrograph methods, a kinematic wave implementation, and a linear quasi-distributed method. Out of all the available transformation methods within HEC-HMS, Snyder Unit Hydrograph (UH) method was selected to perform runoff transformation calculations. The Snyder UH method was selected because of its

Figure 5 – Soils Map



wide spread use in the mountainous watersheds, and the reliable input parameters available for this particular region. Other available rainfall transformation methods, such as the SCS UH and the Clark UH were considered but known limitations of each made the Snyder UH a better selection.

The Snyder UH method, as proposed by F.F. Snyder in 1938, was developed from studies of basins in the Appalachian Mountain region and uses a synthesized hydrograph approach derived from specific physical watershed measurements (Johnstone, 1949). The method calculates flow values using a Snyder lag time as presented in the following equations:

$$Q_p = \frac{640 C_p A}{L_g}$$

where

Q_p= peak runoff (cfs)

C_p= empirical storage or peaking coefficient,

A = watershed or sub-basin area (mi^2), and

L_g = standard Snyder basin lag time (hr).

and

$$L_g = C_t (L L_c)^{0.3}$$

where

C_t = empirical landform coefficient,

L = length of the watershed main stem from divide to outlet (mi), and

L_c = length along the main stem to a point nearest (perpendicular) to the watershed centroid (mi).

Snyder UH is based on five input parameters – three of which are directly measurable from the watershed. The two remaining parameters (C_p and C_t) are empirically based and usually subjectively derived. It is recommended that values for these two parameters be developed through model calibrations from gaged watersheds. Currently, the Stephanie Way watershed does not contain gages, therefore it was decided to use published values for these parameters, which is discussed later in this section.

Complications with using referenced sources of C_t parameter values have been reduced since the inception of the Snyder UH method. The method has been studied, modified, and regionalized by the USACE, US Bureau of Reclamation (USBR), and others. In 1944, the

Los Angeles District of the USACE introduced a modification to the original Snyder standard basin lag time by including the slope of the longest watercourse – a sixth physical watershed parameter (Cudworth, 1989). Subsequently, the USBR has studied, synthesized, calibrated, and further modified the Snyder standard lag time equation into the form used in this restudy, which is:

$$L_g = 26K_n\left(\frac{LL_c}{\sqrt{S}}\right)^{0.33}$$

where

> K_n = an average Manning's *n* roughness coefficient for the principal watercourse of the watershed set to reflect hydraulic conditions during flood events and
> S = overall or average slope of the longest watercourse of the watershed reflecting average conditions (ft/mi).

The primary modification in this form of the Snyder lag time equation is the conversion of the *Ct* parameter into the factor of 26 times average Manning's *n* roughness coefficient, K_n. Most hydrologic modelers have an intuitive or educated sense of appropriate Manning's *n* values – versus the subjective selection of the widely ranging C_t landform parameter.

Runoff using the Snyder UH method is estimated using the following parameters:

- Empirical storage or peaking coefficient, C_p
- Watershed or sub-basin area (mi^2), *A*
- Length of the watershed main stem from divide to outlet (mi), *L*
- Length along the main stem to a point nearest (perpendicular) to the watershed centroid (mi), L_c
- Average Manning's roughness coefficient for the principal watercourse of the watershed, K_n
- Average slope of the longest watercourse (ft/mi), *S*

Early studies developed from the use of the Snyder UH method produced a fairly narrow band of peaking coefficient, C_p, values, ranging from 0.4 to 0.8 (Bedient, 1992). For this study, peaking coefficients are set near the middle of the published range at 0.50. Watershed area, watercourse lengths and slopes were determined using ArcGIS tools. Table 2 – Snyder Unit Hydrograph Parameters lists estimated model parameters.

Table 2 – Snyder Unit Hydrograph Parameters

Subbasin	Area (sq. mi)	Length of Longest Water Course "L" (mile)	Length Along Longest Water Course Nearest to Centroid "L_c" (mile)	High Point (ft)	Low Point (ft)	Elevation Diff (ft)	Slope of Longest Water Course (ft/mile)	UH Peaking Coefficient (C_t)	Manning's n	USBR Method Lag Time (hrs)
W440	0.090	0.97	0.46	5,350.75	4,979.04	371.72	382.44	0.5	0.11	0.82
W490	0.050	0.60	0.38	5,342.35	5,041.62	300.73	498.89	0.5	0.11	0.63
W500	0.015	0.39	0.12	5,053.36	4,974.00	79.36	203.40	0.5	0.11	0.44
W510	0.078	0.62	0.41	5,393.75	5,041.58	352.17	568.58	0.5	0.11	0.64
W520	0.027	0.45	0.23	5,082.12	4,979.10	103.02	227.32	0.5	0.11	0.55
W580	0.055	0.89	0.50	5,341.46	5,003.48	337.98	381.23	0.5	0.11	0.82
W620	0.029	0.35	0.12	5,044.32	4,966.00	78.32	224.97	0.5	0.11	0.42
W640	0.023	0.41	0.20	5,407.91	5,115.20	292.71	721.15	0.5	0.11	0.42
W660	0.036	0.41	0.23	5,422.81	5,115.14	307.66	753.00	0.5	0.11	0.44
W700	0.018	0.29	0.12	5,170.87	5,080.12	90.76	308.28	0.5	0.11	0.37
W710	0.01	0.13	0.05	5,026.00	4,992.00	34.00	255.62	0.5	0.11	0.23
W720	0.079	0.65	0.23	5,194.21	5,003.49	190.73	291.32	0.5	0.11	0.60
W730	0.031	0.43	0.21	5,144.58	5,023.35	121.23	282.64	0.5	0.11	0.51
W760	0.004	0.17	0.06	5,167.29	5,080.12	87.17	526.81	0.5	0.11	0.22
W810	0.080	0.59	0.33	5,431.29	5,094.21	337.08	568.00	0.5	0.11	0.59
W820	0.030	0.42	0.19	5,235.78	5,093.92	141.86	337.00	0.5	0.11	0.47

2.1.A.3 Reach Parameters

A reach is an element of the watershed with one or more inflow and only one outflow. Inflow comes from other elements in the basin model. Outflow is computed using one of 7 different routing methods that simulate open channel flow. Given the predominantly natural terrain and limited land uses in the study area, the Muskingum-Cunge 8-point routing method was selected, and is appropriate. The Muskingum-Cunge routing method is a combination of the conservation of mass and the iterative diffusion of the conservation of momentum at every time step within the channel (USACE, 2009). The following parameters need to be estimated in order to use Muskingum-Cunge routing method:

- Channel length;
- Channel average slope;
- Manning's *n* roughness coefficient for the channel and overbank areas; and
- Eight-point cross-section of channel and effective overbank flow areas.

ArcGIS was utilized to determine average reach cross-sections, channel lengths, and average slopes for each of the reaches defined in the study area. It is important to note that for the Muskingum-Cunge method, the Manning's *n* values are selected to reflect average conditions throughout the entire routing reach. The Manning's *n* value of 0.037 was selected for the main channels and 0.07 was chosen for overbanks areas. A summary of the estimated Muskingum-Cunge parameters are shown in the Table 3 – Muskingum-Cunge Reach Parametets on the next page:

Table 3 – Muskingum-Cunge Reach Parametets

Reach	Length (ft)	Slope (ft/ft)	Mannig's n (Main Channel)	Mannig's n (Over Banks)
R130	1327.18	0.027	0.037	0.070
R140	1203.8	0.033	0.037	0.070
R150	126.5	0.046	0.037	0.070
R160	322.59	0.025	0.037	0.070
R180	158.18	0.040	0.037	0.070
R220	1570.5	0.040	0.037	0.070
R350	1650.8	0.035	0.037	0.070
R380	324.98	0.042	0.037	0.070
R410	710.37	0.049	0.037	0.070

2.1.A.4 Reservoir Parameters

A reservoir element is added to model storage and resulting attenuation of peak flood flows resulting from various precipitation events. A reservoir element in the HEC-HMS model can be used to model reservoirs, lakes, and ponds, and may have one or more inflow and one computed outflow. Inflow into the reservoir element comes from other elements in the basin model. If there is more than one inflow, all inflow is added together before computing the outflow. It is assumed that the water surface in the reservoir pool is level.

While a reservoir element conceptually represents a natural lake, or a lake behind a dam, as in this case, the actual storage simulation calculations are performed by a reservoir routing method. Four different reservoir routing methods are available in HEC-HMS, and Outflow Structures routing method was chosen for this study. Outflow Structures routing method is designed to model reservoirs with a number of uncontrolled outlet structures. For example, a reservoir may have a spillway and several low-level outlet pipes. Low-level outlet was modeled as a raised structure with an 18-inch RCP culvert that allows for partially full or submerged flow that takes both Inlet and Outlet control conditions into consideration. In addition, a 25-ft wide spillway was included to pass flood flows reaching the reservoir during the occurrence of more extreme events such as 0.2-percent annual chance (500-year) and ½-PMP events. The spillway was modeled as a broad-crested weir with a Discharge Coefficient of 3. The crest of the weir (spillway) was set such that it will only be used (discharge floodwaters) during 0.2-percent-annual-chance, and ½-PMP events. That is, the 1-percent annual chance of exceedance (100-year) event will be detained within the reservoir with the water surface elevation in the reservoir below the crest of the emergency spillway.

Several methods are available for defining the storage properties of the reservoir. Elevation-Area method was used for this study to define the characteristics of the proposed reservoir. The Elevation-Area data was extracted from the topographic data using Autodesk Civil 3D and is graphically shown on Figure 6 – Reservoir Stage – Storage Volume Curve The HEC-HMS automatically transforms provided elevation-area into an elevation-storage curve using the conic formula, and will compute the elevation-area-storage characteristics for each time interval.

In order for HEC-HMS to start reservoir transformation computations, initial conditions must be specified. Out of the two choices HEC-HMS provides to set initial condition, the pool elevation method was chosen, and the bottom of the proposed reservoir was used as the initial pool elevation. Tailwater was assumed to have no effect on the reservoir flow, and was, therefore, ignored. The following table summarizes low-level outlet and spillway characteristics considered:

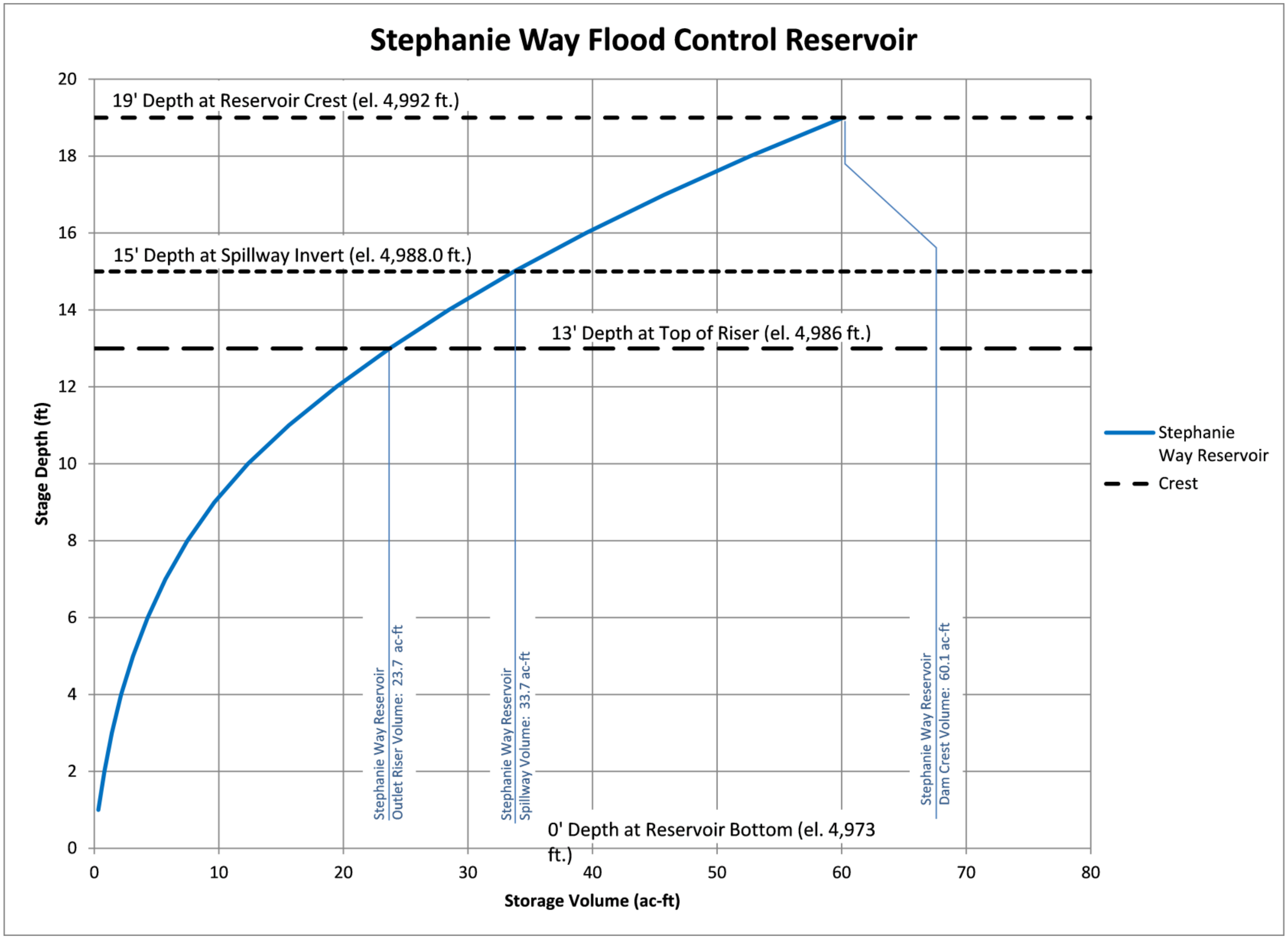
Table 4 – Low-Level Outlet and Spillway Details

Reservoir Outlet Works - Units: feet	
Riser Structure (Circular)	3
Top of Riser	4,986
Orifices (2EA)	0.5
Primary Outlet IE_{IN}	4,973
Primary Outlet IE_{OUT}	4,971
Primary Outlet Size	1.5
Emergency Spillway Crest	4,988.0
Emergency Spillway Width	25
Top of Dam	4,992
Height of Dam	19

2.1.B Meteorologic Model:

In the Meteorologic Model, only the information pertaining to precipitation is entered. Out of several possible methods available to enter precipitation data, Frequency Storm Method was selected for use in developing the Meteorologic Model. A total of six meteorologic models were built to represent 50-, 10-, 4-, 1-, 0.2-percent annual chance precipitation events and ½ PMP precipitation events. Appendix 2 includes a summary table for the original NOAA Atlas 14 rainfall depths.

Figure 6 – Reservoir Stage – Storage Volume Curve



The rainfall depths for the Probable Maximum Precipitation (PMP) were estimated using the protocol presented in Hydrometeorological Report 49 (U.S. Department of Commerce, 1984). The PMP calculations are presented in Table 5 – PMP Calculations and graphically depicted on Figure 7 – General Storm PMP Plot.

2.1.C Control Specifications:

Control specifications are one of the main components of the model, even though they do not contain much parameter data. Control specifications will govern the model simulation time, or the duration of the runoff. The duration of the simulation is defined by the starting date, starting time, ending date, and the ending time in the control specifications. The control specifications are selected so that it exceeds the duration of the rainfall specified in the meteorologic model.

2.2 Hydrologic Modeling Results

The HEC-HMS hydrologic model was used to determine storm-water hydrographs and peak flow rates for the 50-, 10-, 4-, 1-, 0.2-percent-annual-chance, and ½-PMP events under existing land use and watershed conditions for the entire study area. The model is based on the input parameters and modeling methodologies as described in detail in the previous sections of the report. Table 6 summarizes peak flow rates, and associate runoff volume for each rainfall event considered in this study for each sub-basin, including junctions, reaches of the model. Table 7 lists peak storage, peak elevation, along with the available freeboard in the proposed reservoir for each rainfall event modeled. HEC-HMS has limited reservoir routing functionality and does not allow direct modeling of a raiser structure with orifice openings. HydroCAD program has this functionality readily available, and therefore, was used to perform reservoir routing calculations. Detailed printouts of the Hydrologic Modeling results for each storm event are included in Appendix 2.

Table 5 – PMP Calculations

Table 6.1. General -Storm PMP Computations for the Colorado River and Great Basin.

Drainage: Unnamed (Stephanie Way) Area: 0.67 Sq. Miles

Latitude: 39.04310 Longitude: -119.69803 of Basin Center

Month: October

Step				Duration (hrs) 6	12	18	24	48	72	
A	Convergence PMP									
	1. Drainage average value (from one of the figures 2.5 to 2.16)	9.2	Inch							
	2. Reduction for barrier elevation (fig. 2.18)	50	%							
	3. Barrier elevation reduced PMP	4.6	Inch							
	4. Durational Variation (Figs. 2.25 to 2.27 & Table 2.7)			62	82	93	100	119	129	%
	5. Convergence PMP for indicated durations (Steps 3X4)			2.9	3.8	4.3	4.6	5.5	5.9	Inch
	6. Incremental 10 mi^2 PMP (Successive subtraction in step 5)			2.9	0.9	0.5	0.3	0.9	0.4	Inch
	7. Areal reduction (Select from figs 2.28 and 2.29)			100	100	100	100	100	100	%
	8. Areally reduced PMP (Step 6 X Step 7)			2.9	0.9	0.5	0.3	0.9	0.4	Inch
	9. Drainage average PMP (accumulated values of step 8)			2.9	3.8	4.3	4.6	5.5	5.9	Inch
B	Orographic PMP									
	1. Drainage average orographic index (from figure 3.11 a to d)	4	Inch							
	2. Areal reduction (figure 3.20)	100	%							
	3. Adjustment for month (one of figs 3.12 to 3.17)	105	%							
	4. Areally and seasonally adjusted PMP (Steps 1 X 2 X 3)	4.2	Inch							
	5. Durational variation (Table 3.9)			30	57	80	100	157	185	%
	6. Orographic PMP for given durations (Steps 4 X 5)			1.3	2.4	3.4	4.2	6.6	7.8	Inch
C.	Total PMP									
	1. Add Steps A9 and B6			4.2	6.2	7.7	8.8	12.1	13.7	Inch
	2. PMP for other durations from smooth curve fitted to plot of computed data									
	3. Comparison with local-storm PMP (see sec 6.3)									

Reference:
Hydrometeorological Report No. 49 - Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages, Reprinted 1984
U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA), U.S. Department of Army Corps of Engineers

Figure 7 – General Storm PMP Plot

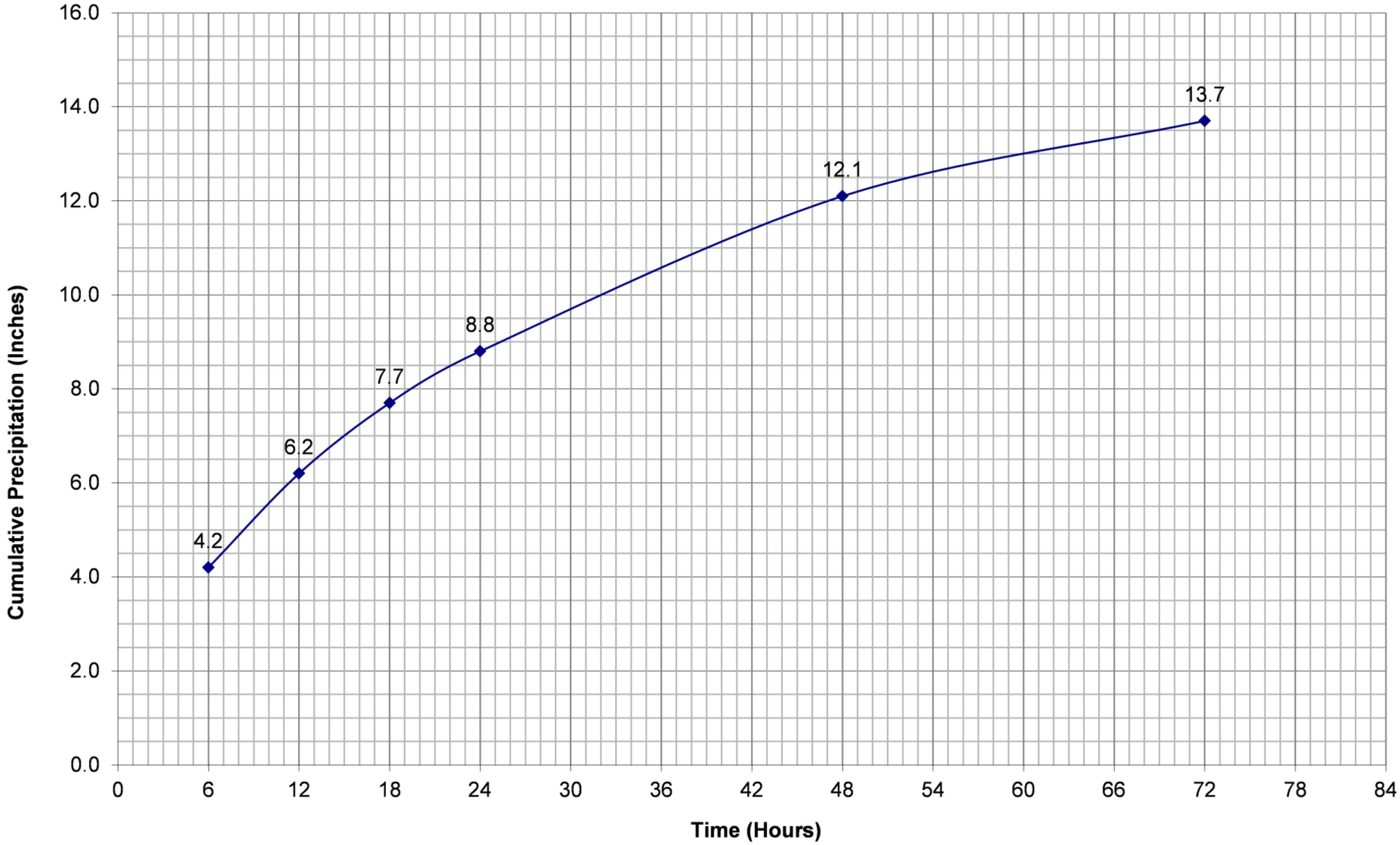


Table 6 – Summary of Hydrologic Modeling Results

		50-percent annual chance		10-percent annual chance		4-percent annual chance		1-percent annual chance		0.2-percent annual chance		Half PMP	
Hydrologic Element	Drainage Area (Sq. miles)	Peak Discharge (cfs)	Volume (Acre-feet)	Peak Discharge (cfs)	Volume (Acre-feet)	Peak Discharge (cfs)	Volume (Acre-feet)	Peak Discharge (cfs)	Volume (Acre-feet)	Peak Discharge (cfs)	Volume (Acre-feet)	Peak Discharge (cfs)	Volume (Acre-feet)
W440	0.090	1.3	0.1	6.1	0.6	10.2	1.1	12.2	1.6	23.8	3.3	26.5	3.6
W490	0.050	3.2	0.3	15.0	1.6	25.3	2.7	22.1	2.2	43.4	4.5	48.6	5.0
W500	0.021	0.0	0.0	1.3	0.1	2.7	0.2	9.9	0.9	19.4	1.7	21.8	1.9
W510	0.078	3.3	0.3	15.1	1.6	25.4	2.7	2.5	0.1	4.7	0.2	5.3	0.2
W520	0.027	2.0	0.2	9.0	1.0	15.3	1.6	10.6	0.7	18.4	1.4	20.3	1.5
W580	0.055	0.2	0.0	3.1	0.4	6.0	0.8	20.3	1.7	33.9	2.9	37.1	3.4
W620	0.035	0.0	0.0	2.6	0.2	5.2	0.4	13.0	1.0	22.0	1.8	24.1	2.0
J164	0.059	3.2	0.2	12.1	0.9	19.1	1.5	33.2	2.7	55.9	4.7	61.2	5.4
R410	0.059	3.2	0.2	12.0	0.9	18.9	1.5	32.9	2.7	55.4	4.7	60.6	5.4
W640	0.023	1.2	0.1	4.6	0.3	7.4	0.5	28.4	2.9	52.4	5.5	57.7	6.0
W660	0.036	2.1	0.2	7.5	0.6	11.8	0.9	13.0	1.1	23.6	2.1	26.1	2.3
J135	0.110	2.2	0.2	12.4	1.2	21.3	2.0	40.9	3.9	75.0	7.6	82.6	8.3
R380	0.110	2.2	0.2	12.4	1.2	21.1	2.0	40.7	3.9	74.6	7.6	82.2	8.3
J138	0.191	5.9	0.5	27.3	2.3	45.1	3.9	82.9	7.5	146.9	13.8	161.1	15.4
R350	0.191	5.9	0.5	27.2	2.3	45.0	3.9	82.9	7.5	146.8	13.8	161.0	15.4
J144	0.222	5.9	0.5	29.6	2.5	49.7	4.3	92.8	8.4	166.3	15.6	182.8	17.3
R180	0.222	5.9	0.5	29.5	2.5	49.5	4.3	92.6	8.4	165.9	15.6	182.5	17.3
J178	0.222	5.9	0.5	29.5	2.5	49.5	4.3	92.6	8.4	165.9	15.6	182.5	17.3
R160	0.222	5.8	0.5	29.2	2.5	49.0	4.3	91.8	8.4	164.9	15.6	181.5	17.3
J176	0.222	5.8	0.5	29.2	2.5	49.0	4.3	91.8	8.4	164.9	15.6	181.5	17.3
R150	0.222	5.8	0.5	29.2	2.5	48.9	4.3	91.4	8.4	164.4	15.6	180.9	17.3
J141	0.356	6.0	0.5	37.3	3.5	64.9	6.2	124.4	12.2	228.5	23.3	251.5	25.9
R140	0.356	6.0	0.5	36.8	3.5	64.4	6.2	123.8	12.2	227.7	23.3	251.1	25.9
W700	0.018	0.8	0.0	3.5	0.2	5.8	0.4	28.3	3.1	50.6	5.8	55.3	6.4
W710	0.008	0.3	0.0	1.6	0.1	2.6	0.2	18.7	2.0	33.1	3.7	36.2	4.2
W720	0.079	0.1	0.0	5.3	0.5	10.6	1.1	47.0	5.2	83.7	9.5	91.5	10.5
W730	0.031	0.0	0.0	2.4	0.2	4.8	0.4	46.8	5.2	83.3	9.5	90.9	10.5
J152	0.128	0.3	0.0	4.9	0.7	9.6	1.3	19.6	2.7	38.5	5.3	42.8	5.9
R220	0.128	0.0	0.0	1.9	0.2	3.9	0.4	8.1	0.8	16.0	1.5	17.9	1.7
J149	0.246	3.5	0.4	21.4	2.4	37.9	4.4	73.0	8.6	135.0	16.3	148.4	18.1
R130	0.246	3.5	0.4	21.3	2.4	37.8	4.4	72.7	8.6	134.1	16.3	147.8	18.1
W760	0.004	0.0	0.0	0.7	0.0	1.3	0.1	10.9	0.8	21.4	1.6	24.1	1.8
W810	0.080	1.5	0.1	8.7	0.8	14.8	1.5	5.6	0.4	11.0	0.9	12.3	1.0
W820	0.030	0.7	0.1	4.0	0.3	6.8	0.5	4.7	0.3	8.2	0.6	9.0	0.7
CP	0.658	9.4	0.9	60.6	6.3	107.3	11.3	207.5	22.4	385.7	42.7	424.2	47.4
Reservoir	0.658	0.0	0.0	0.0	0.0	0.2	0.1	11.0	8.0	41.1	22.0	67.9	26.4

Table 7 – Reservoir Summary

Hydrologic Event	Peak Inflow (cfs)	Peak Discharge (cfs)	Inflow Volume (Acre-Feet)	Discharge Volume (Acre-Feet)	Peak Storage (Acre-Feet)	Peak Elevation (Feet)	Freeboard (feet)
50-percent annual chance	9.4	-	0.9	-	0.9	4,975.2	**16.8**
10-percent annual chance	60.6	-	6.3	-	6.3	4,980.3	**11.7**
4-percent annual chance	107.3	0.2	11.3	0.1	11.3	4,982.6	**9.4**
1-percent annual chance	207.5	11.0	22.4	8.0	19.9	4,985.1	6.9
0.2-percent annual chance	385.7	41.1	42.7	22.0	35.4	4,988.3	3.7
Half PMP	424.2	67.9	47.4	26.4	37.6	4,988.7	3.3

3 Hydraulic Analysis

Two major natural drainage channels traverse through this neighborhood and carry flood flows generated from the upstream watershed. These natural channels are stony, contain considerable amount of weeds with longitudinal slopes ranging from 2 percent to 4 percent along the reach. These channels abruptly end at Esaw Street resulting in heavy sediment build up along Esaw Street and further results in flood waters being dispersed haphazardly.

The proposed flood control dam will detain and attenuate flood flows reducing flows in these natural drainage channels. The outflow from the flood control structure will be discharged into the existing culvert under Romero Drive, just north of Stephanie Way, and ultimately follow the natural drainage course. The other existing natural drainage channel to the north will not receive any flood flows from the upstream watershed with the exception of minor flows generated from the areas lying between the proposed flood control structure and Romero Drive.

In order to analyze the capacity of existing drainage channels and compare anticipated flow depths in the current and proposed conditions (flood-control structure in place), channel cross sections data was obtained at three random locations along the reach of the southern natural drainage way between Romero Drive and Esaw Street. The cross section locations and plot of cross sections data is shown on Figure 8 – Cross Section Map. The extracted channel cross section data and the corresponding channel slope information was then used to perform hydraulic analysis for current and proposed (flood-control structure in place) conditions.

Manning's Roughness Coefficient (Manning's n) is used to calculate energy losses due to channel and overbank characteristics, such as surface roughness, vegetation, channel irregularities, and channel alignment. When corresponding discharge data and water level data are available, Manning's n is calibrated (adjusted) to match observed data. The corresponding data were not available for the study reach; therefore, the Manning's n was estimated from standard engineering references and previous modeling experience. Standard references include Chow (1959) and Barnes (1967). Manning's coefficient of 0.05 was selected that represents natural channels with stones and weeds.

A set of seven hydraulic calculations were performed to represent following conditions:

Current Conditions

- Peak flow resulting from 50-percent annual chance (2-year) storm event (6 cfs)
- Peak flow resulting from 10-percent annual chance (10-year) storm event (40 cfs)
- Peak flow resulting from 4-percent annual chance (25-year) storm event (71 cfs)
- Peak flow resulting from 1-percent annual chance (100-year) storm event (136 cfs)
- Peak flow resulting from 0.2-percent annual chance (500-year) storm event (252 cfs)

Proposed Conditions (Flood-control structure in place)

- Allowed outflow from the proposed flood control reservoir during 1-percent annual chance flood (11 cfs)
- Allowed outflow from the proposed flood control reservoir during 0.2-percent-annual-chance flood (41 cfs)

3.1 Hydraulic Analyses Results

After all the required data were entered in Bentley's FlowMaster software program, Manning's Formula was used to solve for normal depth for a given discharge, and channel slope. The following observations were made after analyzing the hydraulic calculations results:

- During 50-, 10-, and 4-percent annual chance events (2-year, 10-year, and 25-year) flood flows will be completely detained in the flood control reservoir. In comparison, without the proposed flood control structure, estimated flow depths, ranging from 0.4 to 1.9 feet, will occur in the natural drainage channels.
- During the occurrence of 1-percent annual chance event (100-year), the proposed flood control facility limits the outflow from the reservoir to approximately 11 cfs. This outflow is contained within the existing channel and depth of flow from 0.5 to 0.9 feet. In comparison, without the proposed flood control structure an estimated peak flow of 136 cfs will flow through this natural channel with estimated flow depths ranging from 1.7 to 2.6 feet.
- During the occurrence of 0.2-percent annual chance event (500-year), the proposed flood control facility would limit the outflow from the reservoir to approximately 41 cfs. The resulting flow is entirely contained within the channel with estimated flood flow

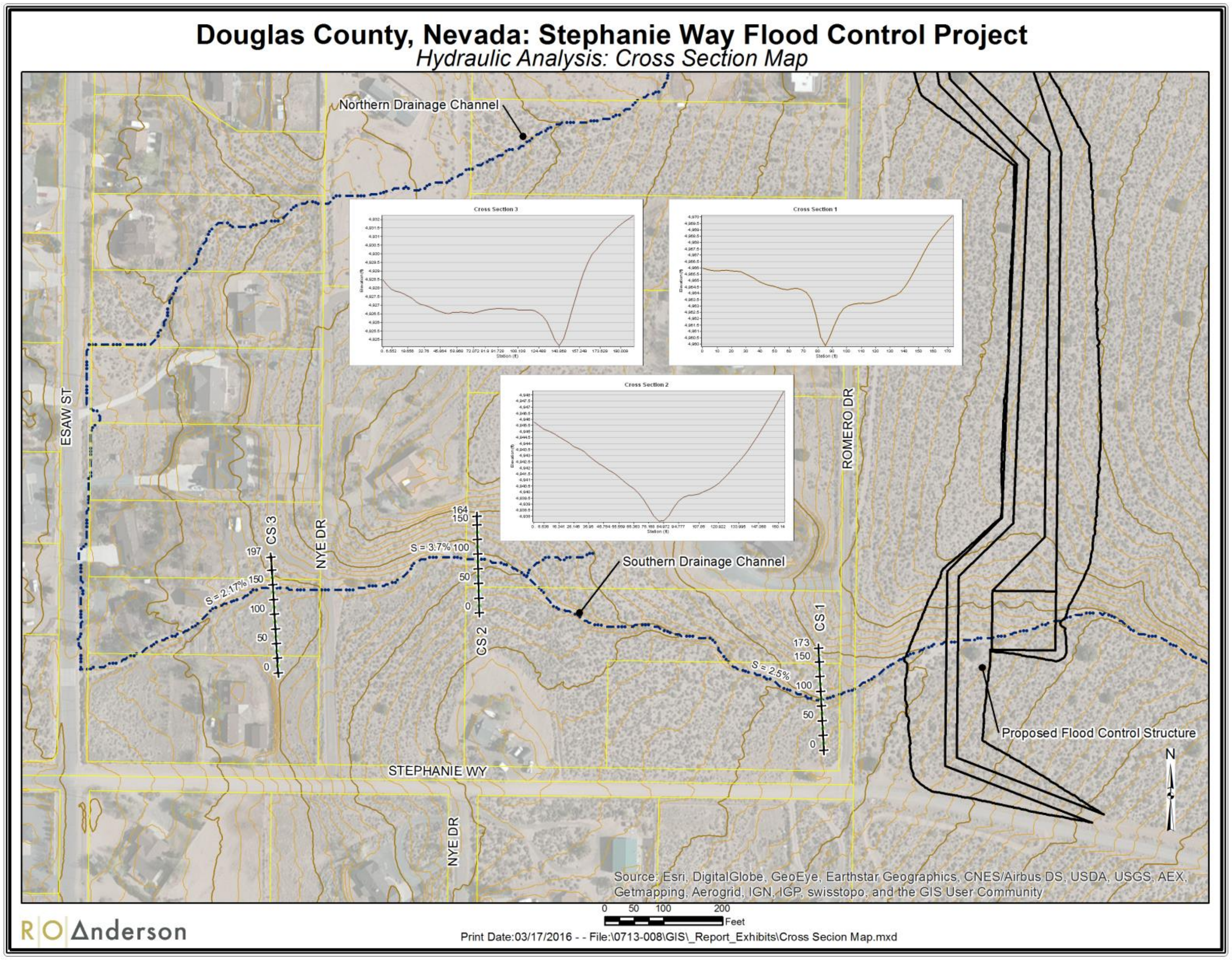
depths ranging from 1 to 1.5 feet. In comparison, without the proposed flood control structure an estimated peak flow of 250 cfs will flow through this natural channel with estimated flood flow depths ranging as high as 2.4 feet to 3.6 feet.

Table 8 – Summary of Hydraulic Analyses Results contains condensed results of hydraulic analyses. Detailed results, including cross section plots are provided in Appendix 3.

Table 8 – Summary of Hydraulic Analyses Results

Storm Event (Year)	Cross Section	Slope (ft/ft)	Current Conditions			Proposed Conditions		
			Peak Flow (cfs)	Normal Depth (ft)	Velocity (ft/sec)	Peak Flow (cfs)	Normal Depth (ft)	Velocity
500	1	0.025	252	3.6	4.81	41	1.5	4.19
	2	0.037	252	2.4	5.82	41	1.0	4.21
	3	0.022	252	2.5	3.21	41	1.3	3.51
100	1	0.025	136	2.6	5.64	11	0.9	2.88
	2	0.037	136	1.7	5.72	11	0.5	2.85
	3	0.022	136	2.3	2.59	11	0.7	2.46
25	1	0.025	71	1.9	4.83	-	-	-
	2	0.037	71	1.3	4.87	-	-	-
	3	0.022	71	1.6	3.96	-	-	-
10	1	0.025	40	1.5	4.16	-	-	-
	2	0.037	40	1.0	4.18	-	-	-
	3	0.022	40	1.3	3.48	-	-	-
2	1	0.025	6	0.7	2.39	-	-	-
	2	0.037	6	0.4	2.36	-	-	-
	3	0.022	6	0.6	2.07	-	-	-

Figure 8 – Cross Section Map



4 Basis of Design, Flood Control Reservoir Layout and Engineer's Estimate of Probable Costs

4.1 Basis of Design

Given the lack of storm water conveyance infrastructure along the Stephanie Way right-of-way, it was determined that the proposed flood control structure should be designed to limit the outflow from the structure to no more than that of 10-year peak flow in current conditions. In addition, representatives of Nevada Division of Water Resources, Bureau of Dam Safety were contacted to confirm the Design Inflow event to safely mitigate and control flood discharges from this watershed. From those discussions, the proposed structure will likely be characterized as a High Hazard Dam. The Design Inflow criteria will, therefore, be the ½-Probable Maximum Precipitation (PMP) event. That is, the proposed dam and its spillway must be sized to safely pass the ½-Probable Maximum Flood (PMF) through the proposed spillway with approximately three feet of freeboard before overtopping the dam structure.

The outlet works and the dam were, therefore, sized to completely detain the inflow from more frequent storm up to 4-percent annual chance (25-year) events. During the occurrence of 1-percent-annual chance (100-year) storm event, a maximum of 11 cfs will be released through the primary outlet structure with water surface elevation in the reservoir well below the crest of the emergency spillway. During the occurrence of 0.2-percent annual chance (500-year) event and ½ PMP events, outflow from the flood control structure will be 41 cfs and 67 cfs, respectively with sufficient freeboard to the top of the dam. The outlet works consists of a low-level outlet with an 18-inch Reinforced Concrete Pipe (RCP) connected to a riser structure. The riser assembly consists of two 12-inch orifice opening on a 3-ft diameter and 13-ft tall concrete structure with aluminum grate on top. During final design, the capacity of the outlet structure and discharge pipe can be reviewed more fully to determine if additional restriction of the outlet discharge is needed.

4.2 Flood Control Reservoir

In order to attenuate peak flood flows and protect the downstream properties in the subdivision, a flood control reservoir was proposed to be constructed east of Romero Drive.

ROA personnel performed site reconnaissance surveys and identified potential location of the proposed flood control reservoir that will have least impact on the existing infrastructure such as access roads, underground and overhead utilities. The identified location was on property managed by the United State Department of the Interior, Bureau of Land Management (BLM). The plausibility of using the identified site for this purpose should be confirmed with representatives of BLM before proceeding with the design of the proposed flood control reservoir. Furthermore, archaeological / paleontological investigation maybe necessary to confirm that there are no cultural resources in this area.

During this feasibility engineering study, the hydrologic modeling of Stephanie Way watershed was undertaken to obtain reasonable estimates of peak flood flows and associated flood volumes for 50-, 10-, 4-, 1-, 0.2-percent annual chance, and ½-PMP storm events. Reservoir stage-area curves were developed using available topographic data to define the reservoirs in HEC-HMS model, and the model was run with the reservoir in place. The modeling results suggested that the proposed reservoir configuration and location is desirable and feasible, primarily because of the available storage area, and reduced cut / fill (earthwork) volumes.

After the reservoir site was selected, HEC-HMS model was further refined by adding detailed information such as low-level outlet works, and spillway information. Detailed discussion of model parameter estimation and the results are presented in Section 2 – Hydrologic Modeling.

The proposed location of the flood control reservoir was compared to the locations of USGS- documented earthquake faults (Quaternary Faults) and is included as Figure 9 – USGS Quaternary Fault Map. It should be noted that there are moderately constrained faults within the reservoir pool area, but no identified faults exists within the limits of the proposed footprints of the flood control structure.

Conceptual layout of the proposed flood control reservoir along with the cross section is displayed on Figure 10 –Reservoir Site Plan. The proposed layout of the flood control structure provides 28.1 acre-feet of storage below the spillway elevation and a dam crest height of 4,992 feet, which is about 19 feet above the channel elevation of the site of the proposed dam.

Figure 9 – USGS Quaternary Fault Map

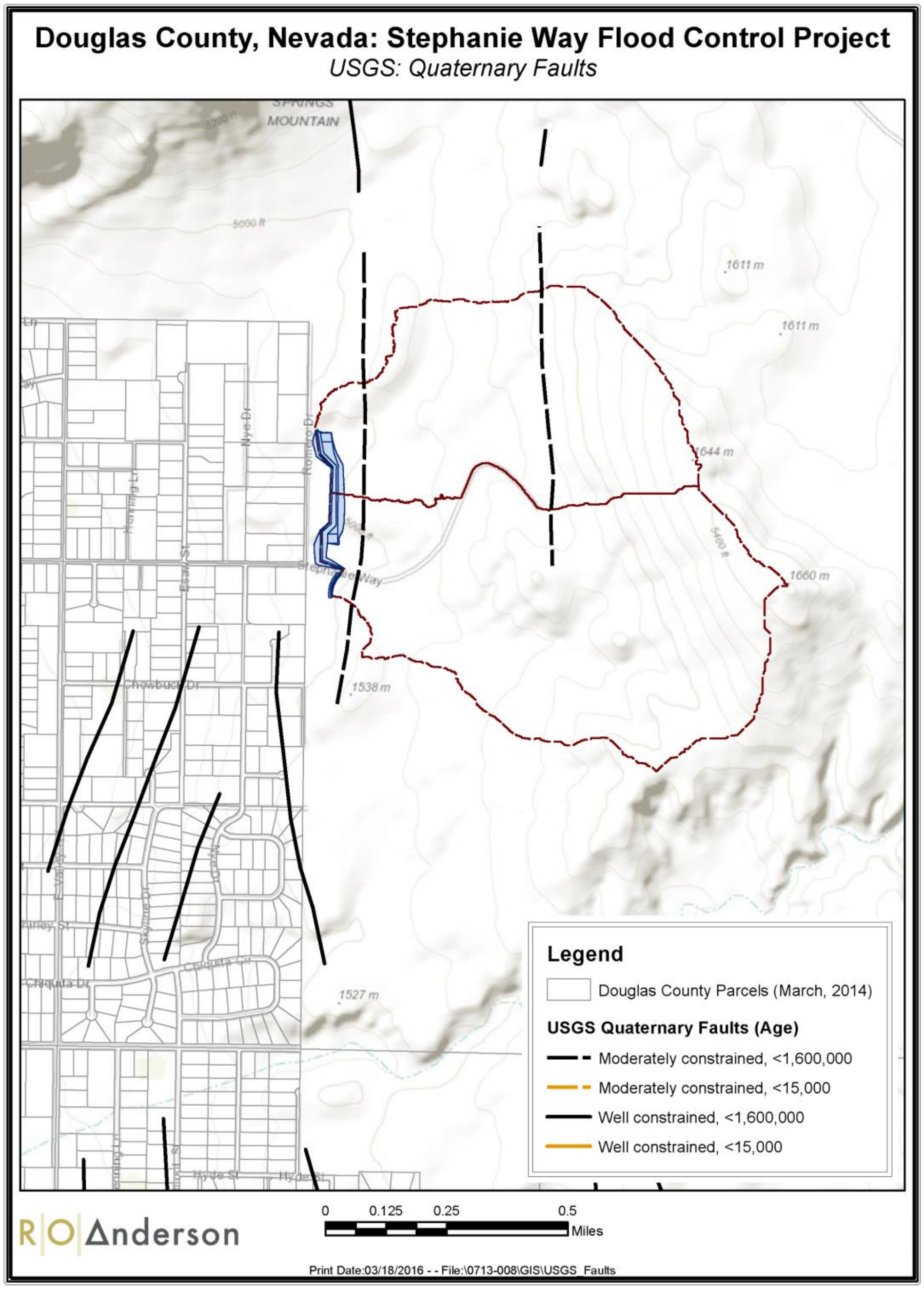
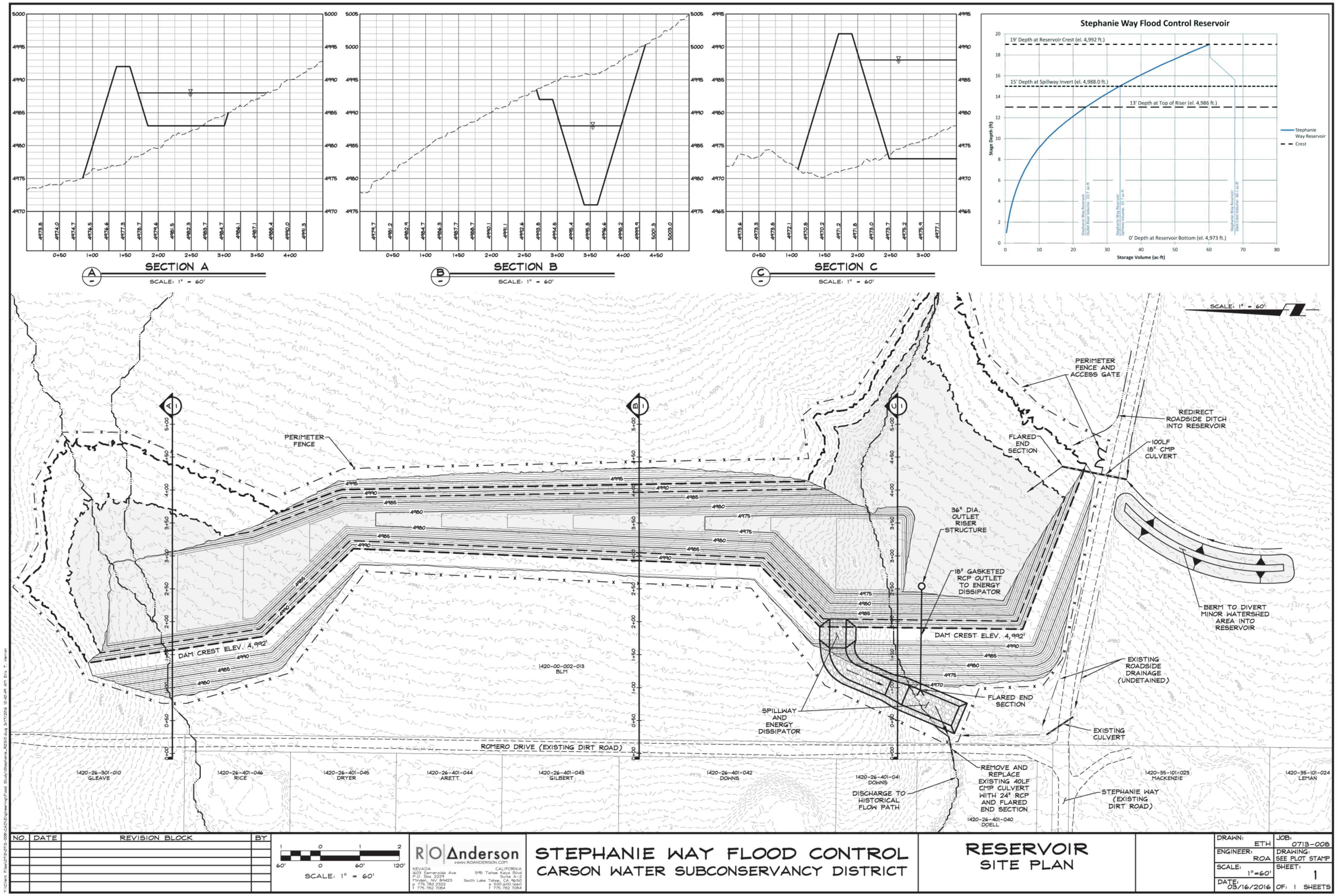


Figure 10 –Reservoir Site Plan



4.3 Engineer's Preliminary Estimate of Probable Cost and Benefit Cost Analyses

Using the schematic design, as shown in Figure 10, an Engineer's Preliminary Estimate of Probable Costs has been developed. The estimates for constructing proposed flood control structure are provided in Table 9 on the following page. The preliminary estimate of probable costs for constructing the proposed flood control structure is estimated to be about $1,337,600. This amount includes: an allowance ($119,700) for construction contingencies at 15% of the estimated probable construction costs; an allowance ($55,000) for land acquisition costs through BLM; an allowance ($185,000) for engineering design and permitting; and, an allowance ($125,000) for construction phase services.

The Stephanie Way Flood Control facility, if constructed will significantly reduce flood induced damages to the residential structures and public infrastructure in this neighborhood, and may be eligible for FEMA's Hazard Mitigation Grant Program. For eligible projects, this program currently provides a 75% grant to complete the design, permitting and construction of proposed flood control facilities. Presuming Douglas County is determined to pursue and was successful in obtaining such a grant, the project cost distribution would be:

- Total Estimated Project Cost: $1,337,600
- Federal HMG Funds (75)%) $1,003,200
- Required Local Match $334,400

Several potential sources for deriving the required local match have been identified including:

- Formation of a Flood Control District specific to Stephanie Way pursuant to NRS 543.170-543.830.
- Formation of a local Assessment District of the benefitted properties.
- A combination of funding from the County and the members of local assessment district.

There may be other grant opportunities available (CDBG, etc.) to assist in achieving the required local match for this project. These opportunities should be researched and considered as the project progresses in to the design and construction phases.

A preliminary BCA was performed using FEMA BCA Version 5.2.1 tool. This tool is a key mechanism for evaluating Hazard Mitigation Grant applications and determining whether mitigation projects are eligible for Federal funding. To be eligible for Federal funding assistance, a BCA should show that the proposed project have a BCA ratio greater than 1.0, and prove that that the proposed project will reduce future damages and losses from natural disasters, such as flooding. FEMA considers reduction in losses or prevention of future damages as benefits of the proposed project, and these benefits should be quantified and at a minimum should outweigh the cost of the proposed project.

Primary input data required for performing BCA is probable construction cost estimate for the proposed flood mitigation project, which was obtained from the engineer's probable cost estimate. The other major element of BCA is quantification of estimated benefits realized from the construction of the proposed flood control reservoir project. The estimated benefits resulting from the implementation of the project can be derived from a variety of sources, such as collection, compilation of documentation of costs associated with expected damage to the structures; loss of use of utilities; loss of roadways and other public infrastructures; and costs incurred by public agencies for debris clean up, and necessary repairs to infrastructure as a direct result of flood damage. Unfortunately, this data is not readily available from the County at this time. Therefore, for the purposes of calculating preliminary BCA, ROA personnel made some simplified assumptions to estimate potential benefits of the project. Examples of some of the simplified assumptions used include cost of cleaning up debris, and sediment buildup, replacement costs of infrastructure such as culverts, roadway, etc. These simplified assumptions are appropriate for use with preliminary estimate of BCA for conceptual level studies. Using this data, FEMA BCA tool was operated and BCA was estimated to be 1.90.

Table 9 – Engineer's Preliminary Estimate of Probable Cost

ENGINEER'S PRELIMINARY ESTIMATE OF PROBABLE COSTS

R|O Anderson

Client:	**CWSD**	**Estimated:**	**ROA**
Project:	**Stephanie Way Regional Flood Control Basin, 33.7 acre-feet**	**Checked:**	
Description:	**Preliminary Estimate of Probable Costs**	**Date:**	**17-Mar-16**
File:	\\1fileprint\clients\Client Files\0713\0713-008\Documents\Cost Estimate\[03_2016 Cost Estimate Worksheet - Stephanie Way.xlsx]Cost Analysis		

ITEM	DESCRIPTION	QUANTITY		UNIT COST		TOTAL
DIVISION 1 - GENERAL REQUIREMENTS						
1	Mobilization, Demobilization, BMPs (15% of construction costs)	1	Lump Sum	$0.00	/LS	$104,100
				SUB TOTAL		**$104,100**
DIVISION 2 - SITE CONSTRUCTION						
1	Clear & Grub	11	Acres	$5,500.00	/Acre	$60,500
2	On-site Earthwork	50,500	Cubic Yards	$5.50	/CYDS	$277,800
3	Rip-Rap - Dam Face	1,536	Cubic Yards	$30.00	/CYDS	$46,100
4	Roller Compacted Concrete - Spillway	2,060	Sq. Feet	$20.00	/SF	$41,200
5	Rip-Rap - Spillway Channel	346	Cubic Yards	$30.00	/CYDS	$10,400
6	Rip-Rap - Energy Dissipation Basin	278	Cubic Yards	$31.00	/CYDS	$8,600
7	Revegetate Disturbed Areas	11	Acres	$2,200.00	/Acre	$24,200
8	Site Fencing & Access Gate	4,640	Lineal Feet	$25.00	/LF	$116,000
9	18" CMP Culvert	100	Lineal Feet	$50.00	/LF	$5,000
10	18" CMP Flared End Section	1	Each	$200.00	/EA	$200
13	Construction Water	1	Lump Sum	$50,000.00	/LS	$50,000
				SUB TOTAL		**$640,000**
DIVISION 3 - CONCRETE						
1	Outlet (36" Dia. 13' Tall Riser with (2) 12" Dia. Orifices)	1	Lump Sum	$5,000.00	/LS	$5,000
2	18" RCP, Gasketed	150	Lineal Feet	$150.00	/LF	$22,500
3	18" RCP Flared End Section	1	Each	$1,500.00	/EA	$1,500
4	24" RCP Culvert	40	Lineal Feet	$200.00	/LF	$8,000
5	24" RCP Flared End Section	1	Each	$1,750.00	/EA	$1,800
				SUB TOTAL		**$38,800**
DIVISION 17 - CONTROLS						
1	Water Level Transducer + SCADA Package	1	Lump Sum	$15,000.00	/LS	$15,000
				SUB TOTAL		**$15,000**

CONSTRUCTION SUB TOTAL	**$797,900**
CONTINGENCY AT 15% OF CONSTRUCTION[1]	**$119,700**
Land Acquisition, BLM Permitting + NEPA	**$55,000**
Engineering Design & Geotechnical Investigation	**$185,000**
Permitting - Dam Safety + Douglas County + NDEP Air Quality + CLOMR/LOMR	**$55,000**
Construction Phase Services including Construction Staking	**$125,000**
ENGINEERS PRELIMINARY ESTIMATE OF PROBABLE PROJECT COSTS (EXCL. FINANCING)	**$1,337,600**

5 Findings and Conclusions

The contributing watershed of the project area is a relatively small, unmapped watershed situated between Buckbrush Wash and Johnson Lane Wash in Carson Valley. Even though the most recent FIRM issued by the FEMA does not identify the project area being in the Special Flood Hazard Areas (SFHA), this neighborhood has experienced repetitive flooding and heavy sediment deposition in the past. For example, floods of 2014 and 2015 resulted in considerable damages to the residential properties and public infrastructure in this area. This study was undertaken to explore the feasibility of constructing a flood control structure on BLM property east of Romero Drive to alleviate flood-induced recurring damages in this neighborhood. The following is the summary of findings and conclusions from this feasibility-level study:

- The effective FIS did not include detailed study of the project area and, therefore, does not contain estimated peak flows from typical storm events. However, as mentioned above, this neighborhood has experienced repetitive flooding in the past, and, therefore, it is appropriate and prudent to evaluate the hydrology of this watershed and estimate runoff peak flows and volumes for various precipitation events.
- The hydrologic study performed by ROA personnel and presented in this report used current NOAA precipitation data, and used Green-Ampt loss rate method that is proven to provide reasonable estimate of runoff peak and volume. The hydrologic modeling estimated runoff peak flows and volumes resulting from six hypothetical precipitation events, namely 50-,10-, 4-, 1-, 0.2-percent annual chance, and ½ PMP storm events.
- General PMP rainfall depths were computed using HMR-49 guidelines, and the resulting rainfall data was used to construct a hyetograph that was applied uniformly over the entire watershed. The resulting hydrograph at the most downstream end of the watershed was taken and the ordinates of this flood hydrograph were divided in half to obtain ½-PMF. The resulting ½-PMF was routed through the proposed flood control reservoir.
- While preparing this feasibility analysis, Nevada Division of Water Resources, Bureau of Dam Safety was contacted to confirm the Design Inflow event that the proposed structure will be required to be designed to safely pass. From those

discussions, the proposed structure will likely be characterized as a High Hazard Dam. The Design Inflow criteria will therefore be the ½-PMP event. That is, the proposed dam and its appurtenances must be sized to pass the ½-PMF through the proposed spillway with approximately three feet of freeboard before overtopping.

- After reviewing the estimated peak flood flows from 1-, 0.2-percent, and ½-PMF events, two alternate flood control basin configurations were considered, and a feasibility analysis was performed, which culminated in the selection of a preferred alternative that avoids conflicts with the existing overhead and underground utilizes located along the dirt access road to the power substation.
- The flood flows from a relatively small watershed south of the dirt access road to the power substation will be detained by constructing a 5-ft high berm and conveyed into the proposed flood control reservoir by an 18-inch CMP crossing.
- The embankment of the preferred alternative flood control structure is about 19 feet above natural grade with a normal storage capacity of 28.1 acre-feet, and a storage capacity of 61.3 acre-feet at dam crest.
- The proposed flood control structure outlet works consists of a 3-ft diameter concrete riser structure that is 13-ft tall with two 12-inch orifice openings, an aluminum grate on top and an18-inch RCP outlet pipe connected to the riser structure. The emergency spillway consists of a 25-ft wide Roller Compacted Concrete (RCC) structure discharging into a riprap energy dissipator.
- The primary and emergency outlet works were sized such that flood flows resulting from precipitation events up to 4-percent annual chance (25-year) storm events are completely detained and, during the 1-percent annual chance flood, the outflow discharge is limited to release just 11 cfs through the primary outlet. During 0.2-percent annual-chance flood and ½-PMF events, the emergency spillway was sized to safely convey incoming flood flows with sufficient freeboard and some attenuation.
- The existing 24-inch CMP culvert under Romero Drive, just north of Stephanie Way will be replaced with a 24-inch RCP with flared end sections. The outflow from the proposed flood control structure will be routed to the new 24-inch RCP culvert and perpetuated in the existing natural channel and follows the historic flow path.
- Hydraulic calculations of receiving downstream natural drainage channel below the proposed flood control facility were performed using Manning's Formula. A set of seven hydraulic calculations were performed that represent discharges from the

current and proposed (flood-control structure in place) conditions and compared the resulting flood depths within the channel for current and proposed conditions. It was observed that the flood control structure drastically reduces peak flows resulting in reduced flow depth in the existing natural channel. This ultimately translates to reduced sediment loading, deposition and more manageable storm water flows in the natural channels.

- Constructing a flood control basin east of Romero Drive on BLM property with an estimated cost of $1.3 million dollars results in direct and substantial benefit to the residents in this area. The project provides additional indirect benefits to the residents of Douglas County by reducing potential damage to public infrastructure such as roads and drainage structures in this area.
- The Engineer's Preliminary Estimate of Probable Costs for preferred alternative is $1,337,600, which amount includes allowances for construction contingencies, land acquisition, engineering design, permitting and construction phase services.
- The Stephanie Way Flood Control project may be eligible for FEMA's Hazard Mitigation Grant Program that currently provides 75% grants for qualified projects.
- If successful in obtaining a Hazard Mitigation Grant for this project, the required local match to complete improvements is estimated to be $334,400.
- Preliminary BCA shows a BCA of 1.90 for the proposed improvements.
- The proposed location of the flood control structure footprint and the extents of reservoir area were compared to the locations of USGS- documented earthquake faults (Quaternary Faults). There are no identified active faults within the foot prints of the proposed structure.
- From these investigations, we conclude that the project is eminently feasible and worthy of pursuing further.

6 References

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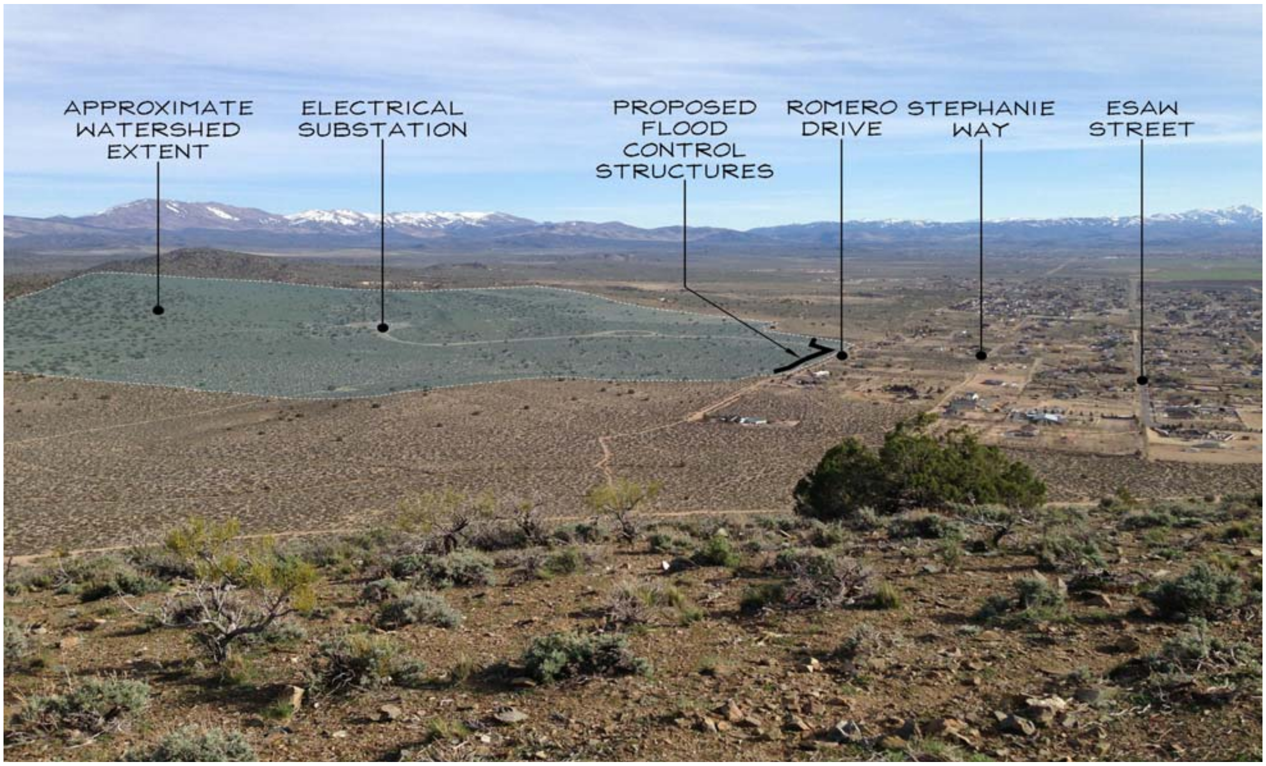
7 Appendices

Appendix 1: March 9, 2016 Site Reconnaissance Pictures

Appendix 2: Hydrologic Modeling Results

Appendix 3: Hydraulic Analyses Results

APPENDIX 1
PHOTOGRAPHS – MARCH 9, 2016



Title:	Project Watershed Area	File:	Image1_Watershed.jpg	Date:	3/9/2016
Desc:	Looking South from Hot Springs Mountain area to Stephanie Way Study Area Watershed.			Slide:	1

Title:	Stephanie Way	File:	Image2_Stephanie.JPG	Date:	3/9/2016 16:57
Desc:	Looking East along Stephanie Way toward Romero Drive Intersection.			Slide:	2

Title:	Romero Drive	File:	Image3_Romero.JPG	Date:	3/9/2016 17:24
Desc:	Looking North along Romero Drive from Southerly Culvert Crossing.			Slide:	3

Title:	Romero Drive Southerly Culvert Crossing	File:	Image4_Romero.JPG	Date:	3/9/2016 17:20
Desc:	Looking West at Southerly Romero Drive 24" CMP Culvert Crossing Inlet.			Slide:	4

Title:	Romero Drive Southerly Culvert Crossing Outlet Channel	File:	Image5_Outlet.JPG	Date:	3/17/2016 14:43
Desc:	Looking West at drainage channel from Southerly Romero Drive 24" CMP Culvert Crossing Outlet.			Slide:	5

Title:	Romero Drive Northerly Culvert Crossing Outlet	File:	Image6_Romero.JPG	Date:	3/9/2016 17:26
Desc:	Looking West at drainage channel from Northerly Romero Drive Culvert Crossing Outlet.			Slide:	6

Title:	Southerly Embankment Location	File:	Image7_Reservoir.JPG	Date:	3/9/2016 17:04
Desc:	Looking North from Stephanie Way East of Romero Drive at approximate proposed Southerly Dam location.			Slide:	7

APPENDIX 2
HYDROLOGIC MODELING RESULTS

Project Notes

Project # 0713-008_Stephanie Way Flood Control Project
Client: Carson Water Subconservancy District
Flow routing through proposed flood control reservoir with a raiser structure and emergency spillway as primary and secondary outlets. The inflow hydrographs into the proposed reservoir was obtained from HEC-HMS analysis.

Summary for Pond 1P: Stephanie Way Reservoir

Inflow =	9.29 cfs @	12.86 hrs,	Volume=	0.906 af
Outflow =	0.00 cfs @	10.00 hrs,	Volume=	0.000 af, Atten= 100%, Lag= 0.0 min
Primary =	0.00 cfs @	10.00 hrs,	Volume=	0.000 af
Secondary =	0.00 cfs @	10.00 hrs,	Volume=	0.000 af

Routing by Stor-Ind method, Time Span= 10.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 4,975.18' @ 17.10 hrs Surf.Area= 0.000 ac Storage= 0.906 af

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

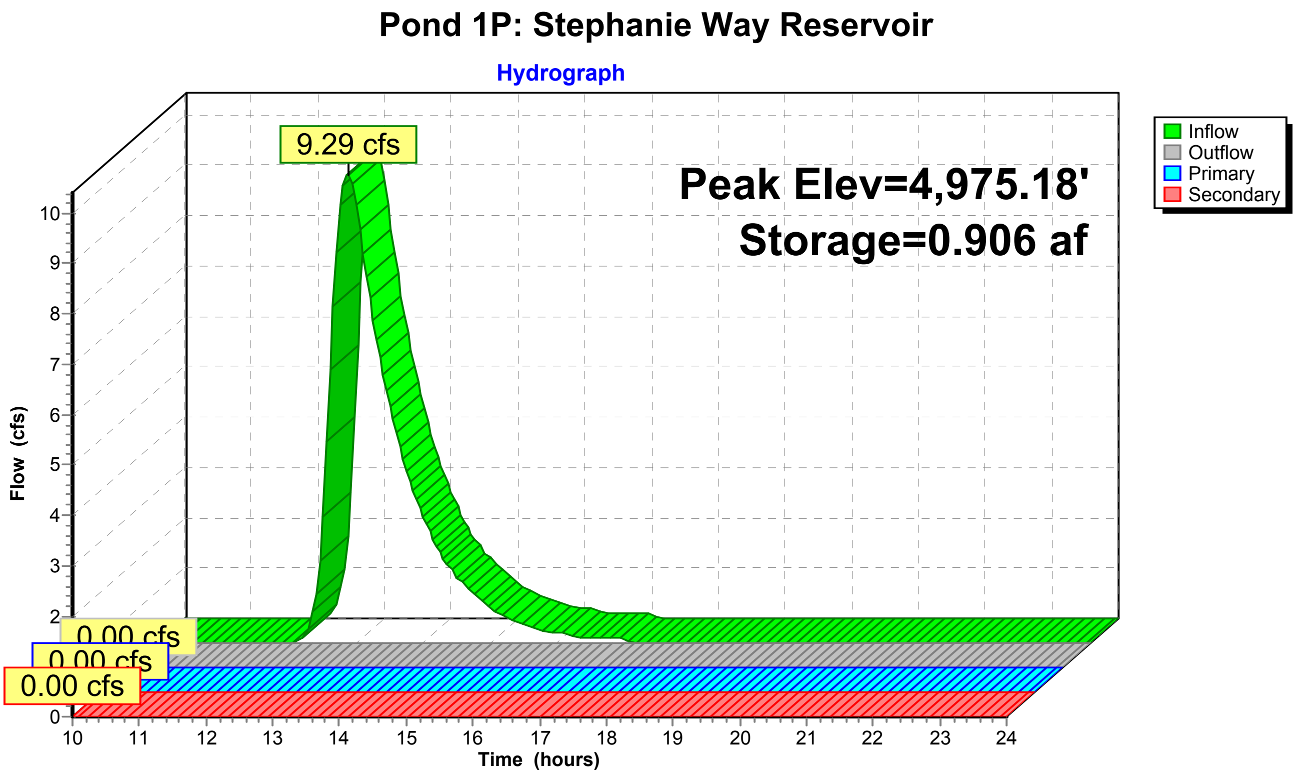
Volume	Invert	Avail.Storage	Storage Description
#1	4,973.00'	60.100 af	**Custom Stage Data** Listed below

Elevation (feet)	Cum.Store (acre-feet)
4,973.00	0.000
4,974.00	0.300
4,975.00	0.800
4,976.00	1.400
4,977.00	2.200
4,978.00	3.100
4,979.00	4.300
4,980.00	5.700
4,981.00	7.500
4,982.00	9.600
4,983.00	12.300
4,984.00	15.600
4,985.00	19.400
4,986.00	23.700
4,987.00	28.400
4,988.00	33.700
4,989.00	39.500
4,990.00	45.800
4,991.00	52.600
4,992.00	60.100

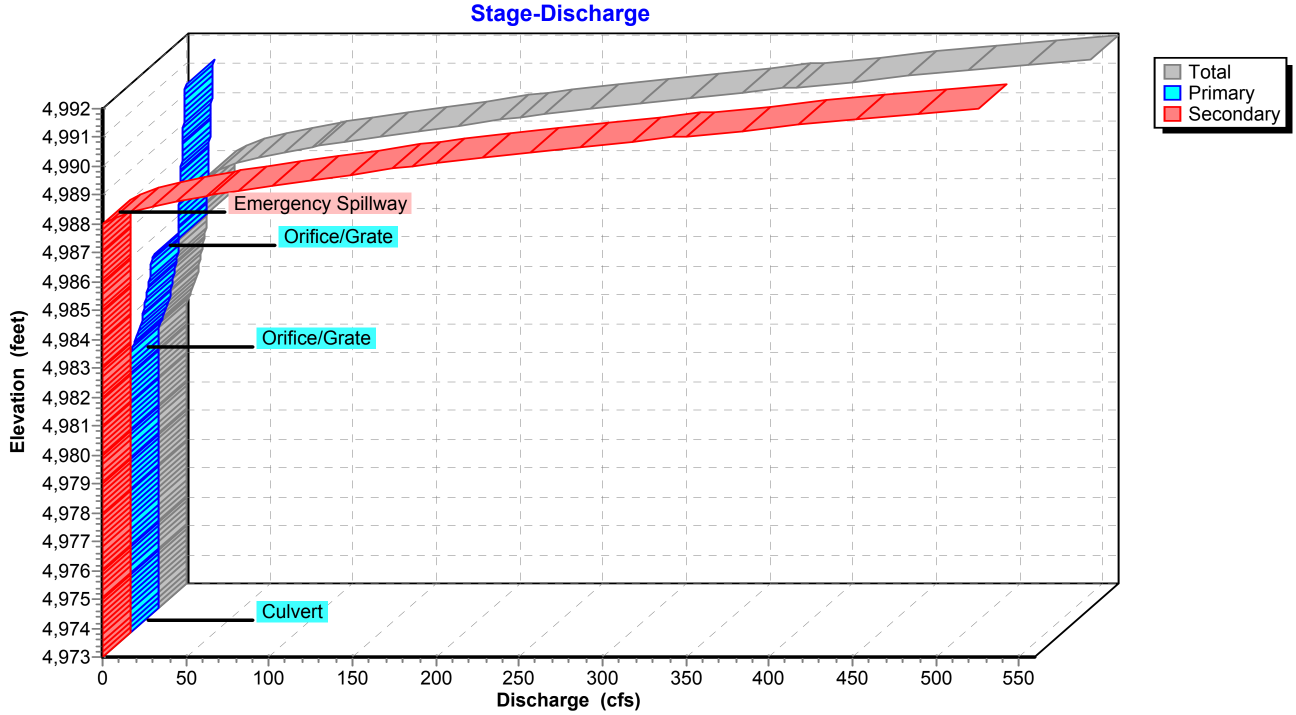
Device	Routing	Invert	Outlet Devices
#1	Primary	4,973.00'	**18.0" Round Culvert** L= 130.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 4,973.00' / 4,971.00' S= 0.0154 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Secondary	4,988.00'	**25.0' long x 20.0' breadth Emergency Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#3	Device 1	4,982.50'	**12.0" Vert. Orifice/Grate X 2.00** C= 0.600
#4	Device 1	4,986.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.00 cfs @ 10.00 hrs HW=4,973.00' (Free Discharge)
└─**1=Culvert** (Controls 0.00 cfs)
  ├─**3=Orifice/Grate** (Controls 0.00 cfs)
  └─**4=Orifice/Grate** (Controls 0.00 cfs)

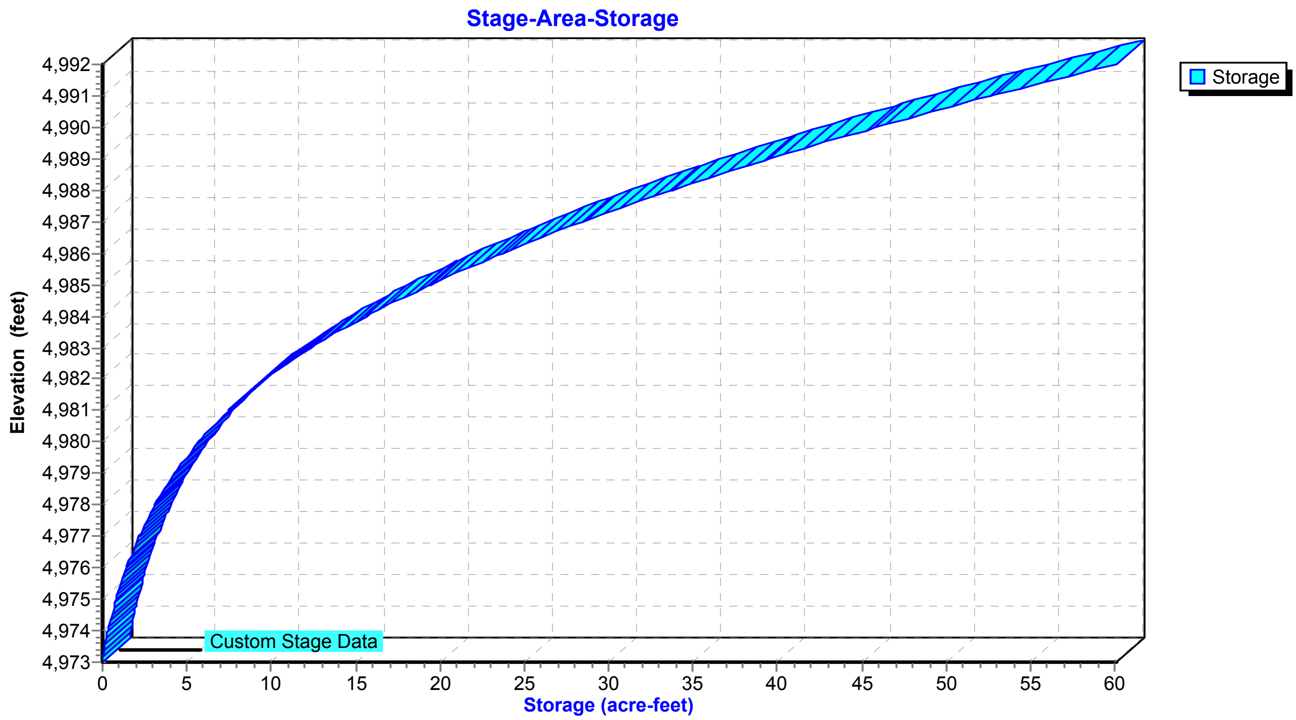
Secondary OutFlow Max=0.00 cfs @ 10.00 hrs HW=4,973.00' (Free Discharge)
└─**2=Emergency Spillway** (Controls 0.00 cfs)



Pond 1P: Stephanie Way Reservoir



Pond 1P: Stephanie Way Reservoir



Summary for Pond 1P: Stephanie Way Reservoir

Inflow = 60.10 cfs @ 12.75 hrs, Volume= 6.229 af
Outflow = 0.00 cfs @ 10.00 hrs, Volume= 0.000 af, Atten= 100%, Lag= 0.0 min
Primary = 0.00 cfs @ 10.00 hrs, Volume= 0.000 af
Secondary = 0.00 cfs @ 10.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 10.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 4,980.29' @ 18.60 hrs Surf.Area= 0.000 ac Storage= 6.229 af

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

Volume	Invert	Avail.Storage	Storage Description
#1	4,973.00'	60.100 af	**Custom Stage Data** Listed below

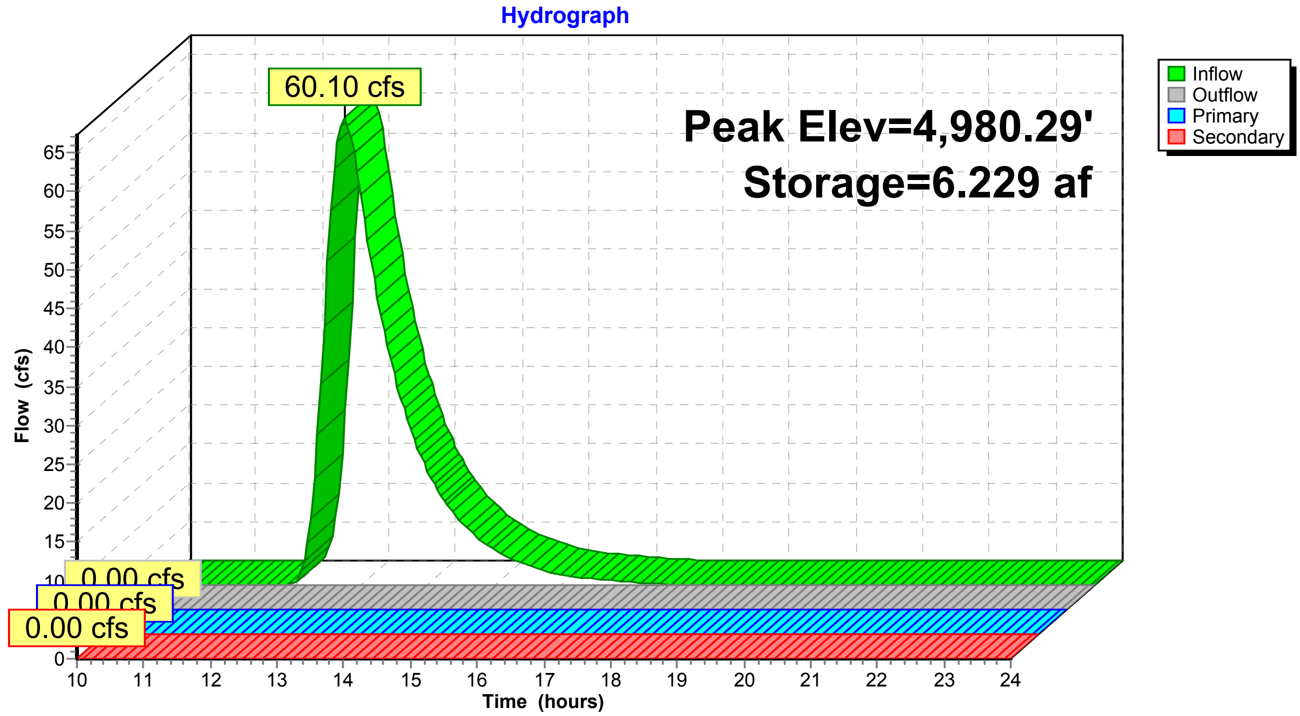
Elevation (feet)	Cum.Store (acre-feet)
4,973.00	0.000
4,974.00	0.300
4,975.00	0.800
4,976.00	1.400
4,977.00	2.200
4,978.00	3.100
4,979.00	4.300
4,980.00	5.700
4,981.00	7.500
4,982.00	9.600
4,983.00	12.300
4,984.00	15.600
4,985.00	19.400
4,986.00	23.700
4,987.00	28.400
4,988.00	33.700
4,989.00	39.500
4,990.00	45.800
4,991.00	52.600
4,992.00	60.100

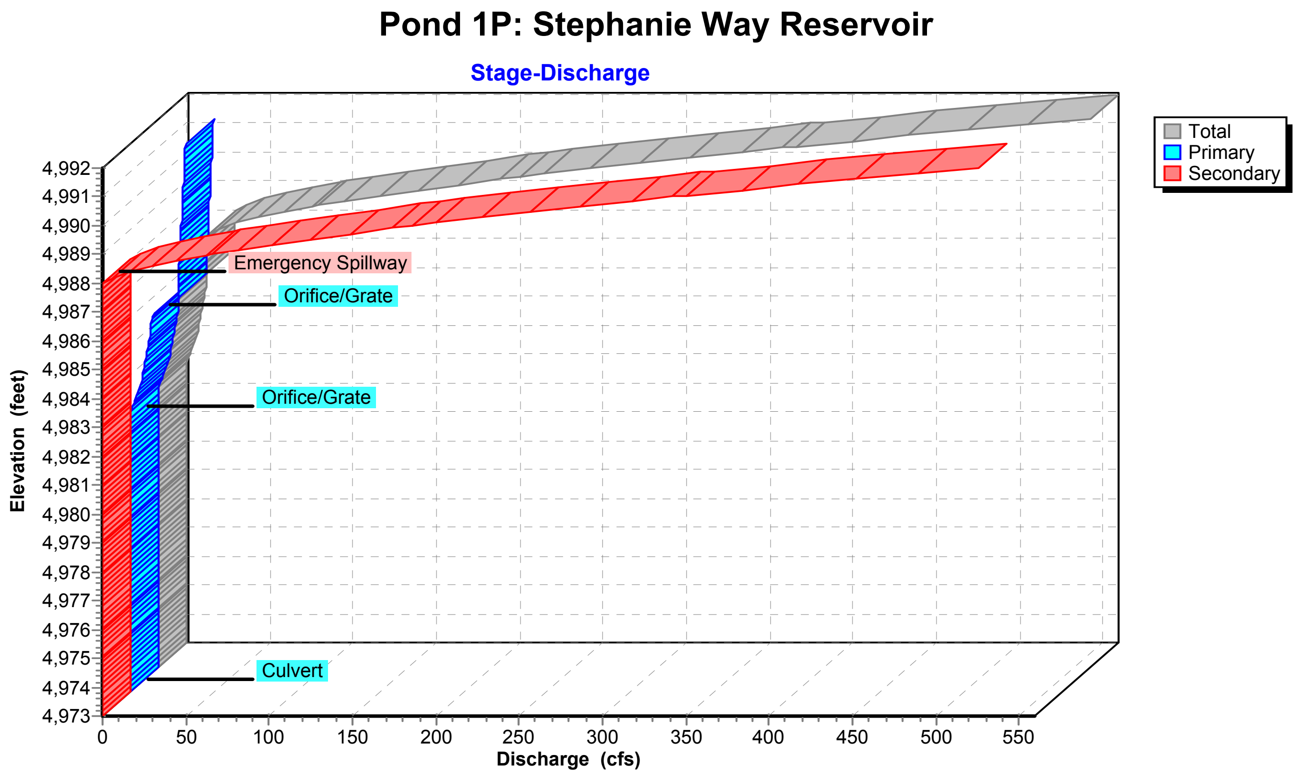
Device	Routing	Invert	Outlet Devices
#1	Primary	4,973.00'	**18.0" Round Culvert** L= 130.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 4,973.00' / 4,971.00' S= 0.0154 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Secondary	4,988.00'	**25.0' long x 20.0' breadth Emergency Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#3	Device 1	4,982.50'	**12.0" Vert. Orifice/Grate X 2.00** C= 0.600
#4	Device 1	4,986.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

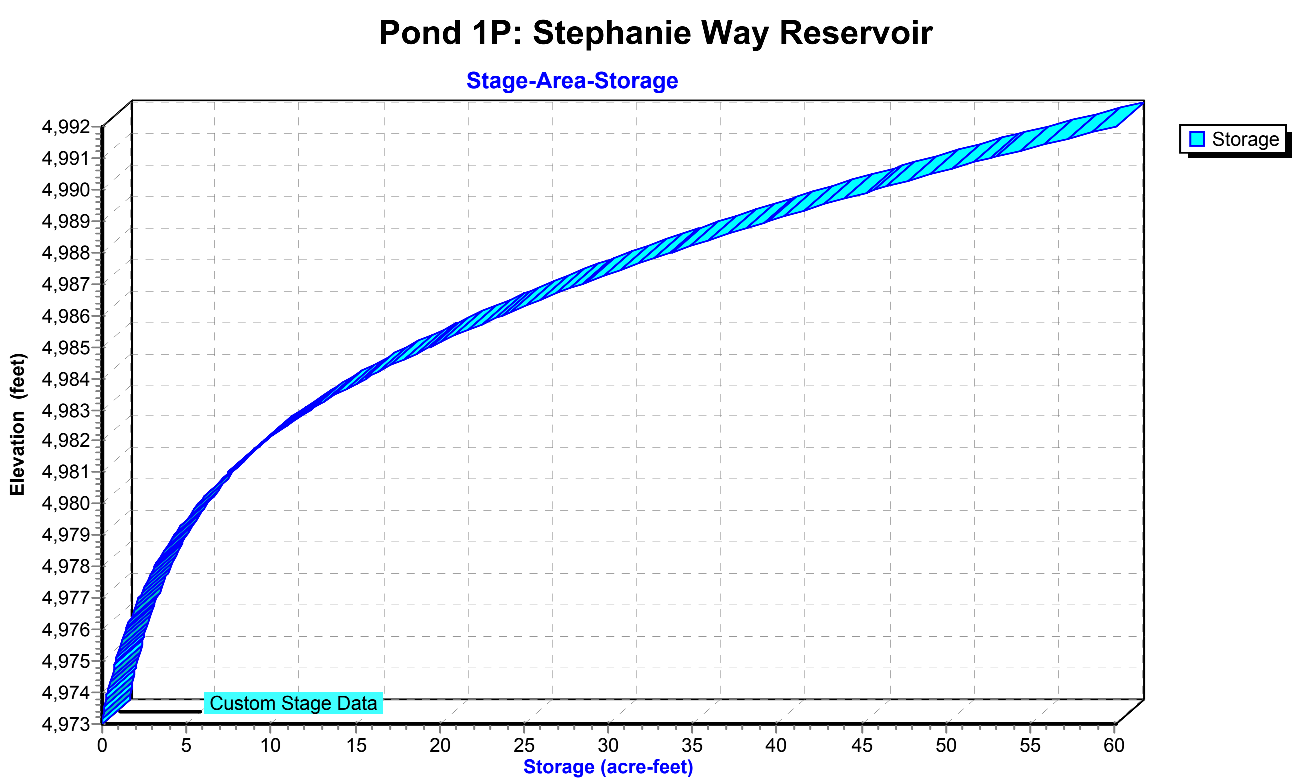
Primary OutFlow Max=0.00 cfs @ 10.00 hrs HW=4,973.00' (Free Discharge)
└─**1=Culvert** (Controls 0.00 cfs)
  ├─**3=Orifice/Grate** (Controls 0.00 cfs)
  └─**4=Orifice/Grate** (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 10.00 hrs HW=4,973.00' (Free Discharge)
└─**2=Emergency Spillway** (Controls 0.00 cfs)

Pond 1P: Stephanie Way Reservoir




Pond 1P: Stephanie Way Reservoir
Stage-Discharge
Total
Primary
Secondary
Emergency Spillway
Orifice/Grate
Orifice/Grate
Culvert
Elevation (feet)
4,992
4,991
4,990
4,989
4,988
4,987
4,986
4,985
4,984
4,983
4,982
4,981
4,980
4,979
4,978
4,977
4,976
4,975
4,974
4,973
0 50 100 150 200 250 300 350 400 450 500 550
Discharge (cfs)


Pond 1P: Stephanie Way Reservoir
Stage-Area-Storage
Storage
Custom Stage Data
Elevation (feet)
4,992
4,991
4,990
4,989
4,988
4,987
4,986
4,985
4,984
4,983
4,982
4,981
4,980
4,979
4,978
4,977
4,976
4,975
4,974
4,973
0 5 10 15 20 25 30 35 40 45 50 55 60
Storage (acre-feet)

Summary for Pond 1P: Stephanie Way Reservoir

Inflow = 9.29 cfs @ 12.86 hrs, Volume= 0.906 af
Outflow = 0.00 cfs @ 10.00 hrs, Volume= 0.000 af, Atten= 100%, Lag= 0.0 min
Primary = 0.00 cfs @ 10.00 hrs, Volume= 0.000 af
Secondary = 0.00 cfs @ 10.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 10.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 4,975.18' @ 17.10 hrs Surf.Area= 0.000 ac Storage= 0.906 af

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

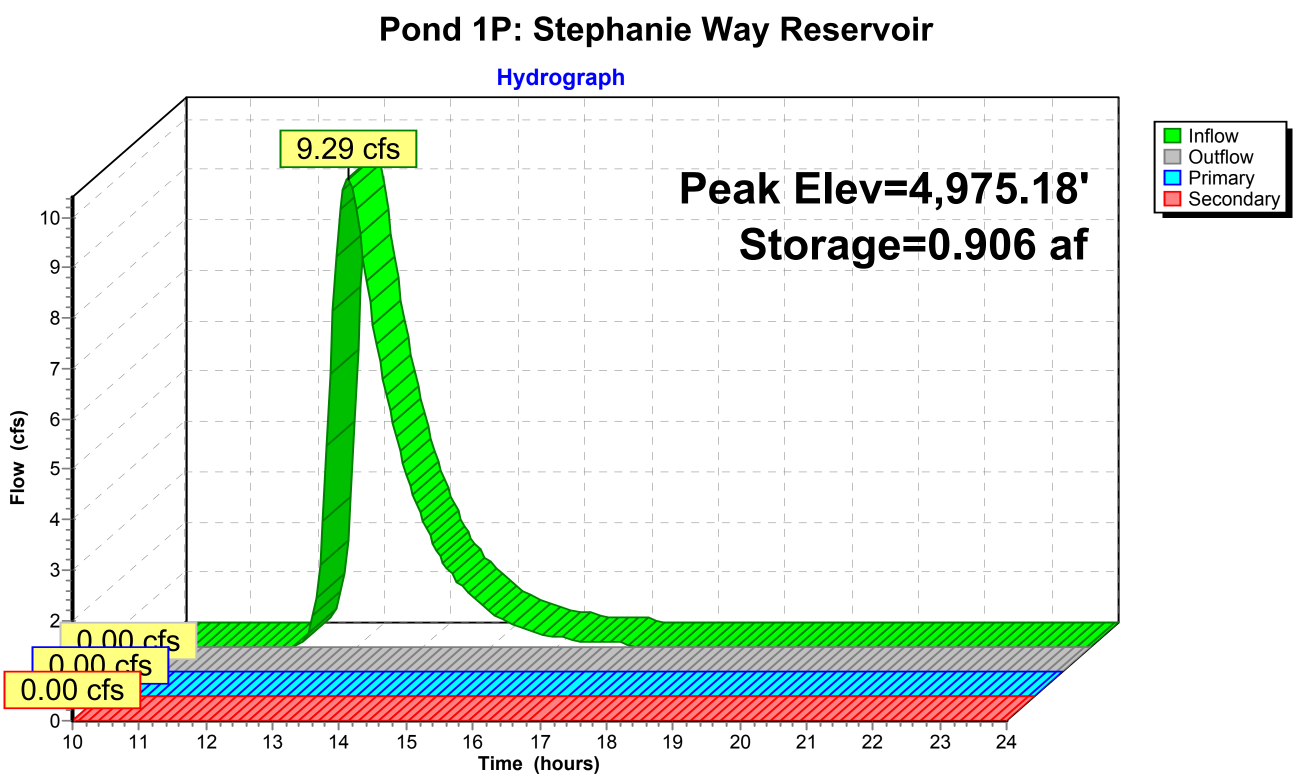
Volume	Invert	Avail.Storage	Storage Description
#1	4,973.00'	60.100 af	**Custom Stage Data** Listed below

Elevation (feet)	Cum.Store (acre-feet)
4,973.00	0.000
4,974.00	0.300
4,975.00	0.800
4,976.00	1.400
4,977.00	2.200
4,978.00	3.100
4,979.00	4.300
4,980.00	5.700
4,981.00	7.500
4,982.00	9.600
4,983.00	12.300
4,984.00	15.600
4,985.00	19.400
4,986.00	23.700
4,987.00	28.400
4,988.00	33.700
4,989.00	39.500
4,990.00	45.800
4,991.00	52.600
4,992.00	60.100

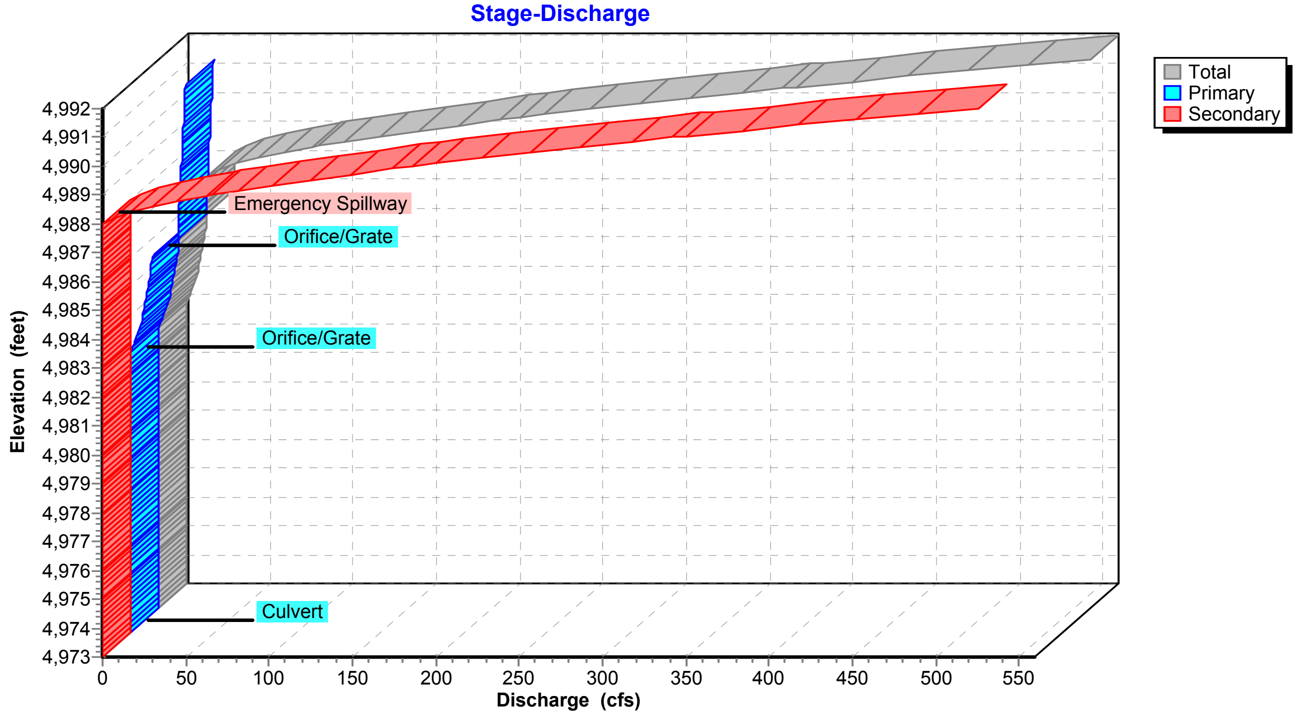
Device	Routing	Invert	Outlet Devices
#1	Primary	4,973.00'	**18.0" Round Culvert** L= 130.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 4,973.00' / 4,971.00' S= 0.0154 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Secondary	4,988.00'	**25.0' long x 20.0' breadth Emergency Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#3	Device 1	4,982.50'	**12.0" Vert. Orifice/Grate X 2.00** C= 0.600
#4	Device 1	4,986.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.00 cfs @ 10.00 hrs HW=4,973.00' (Free Discharge)
└─**1=Culvert** (Controls 0.00 cfs)
  ├─**3=Orifice/Grate** (Controls 0.00 cfs)
  └─**4=Orifice/Grate** (Controls 0.00 cfs)

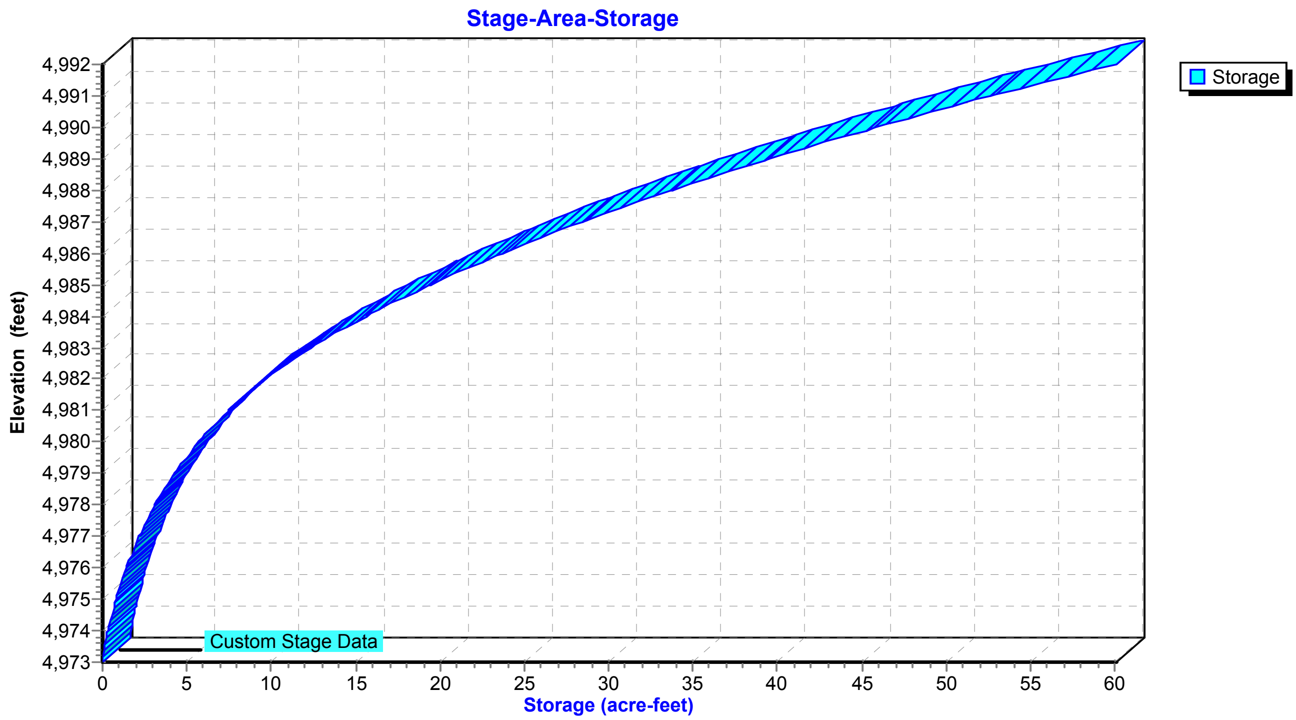
Secondary OutFlow Max=0.00 cfs @ 10.00 hrs HW=4,973.00' (Free Discharge)
└─**2=Emergency Spillway** (Controls 0.00 cfs)



Pond 1P: Stephanie Way Reservoir



Pond 1P: Stephanie Way Reservoir



Summary for Pond 1P: Stephanie Way Reservoir

Inflow	=	206.52 cfs @	12.73 hrs,	Volume=	22.278 af
Outflow	=	10.96 cfs @	15.31 hrs,	Volume=	7.982 af, Atten= 95%, Lag= 155.1 min
Primary	=	10.96 cfs @	15.31 hrs,	Volume=	7.982 af
Secondary	=	0.00 cfs @	10.00 hrs,	Volume=	0.000 af

Routing by Stor-Ind method, Time Span= 10.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 4,985.10' @ 15.31 hrs Surf.Area= 0.000 ac Storage= 19.834 af

Plug-Flow detention time= 336.0 min calculated for 7.954 af (36% of inflow)
Center-of-Mass det. time= 295.1 min (1,091.1 - 796.1)

Volume	Invert	Avail.Storage	Storage Description
#1	4,973.00'	60.100 af	**Custom Stage Data** Listed below

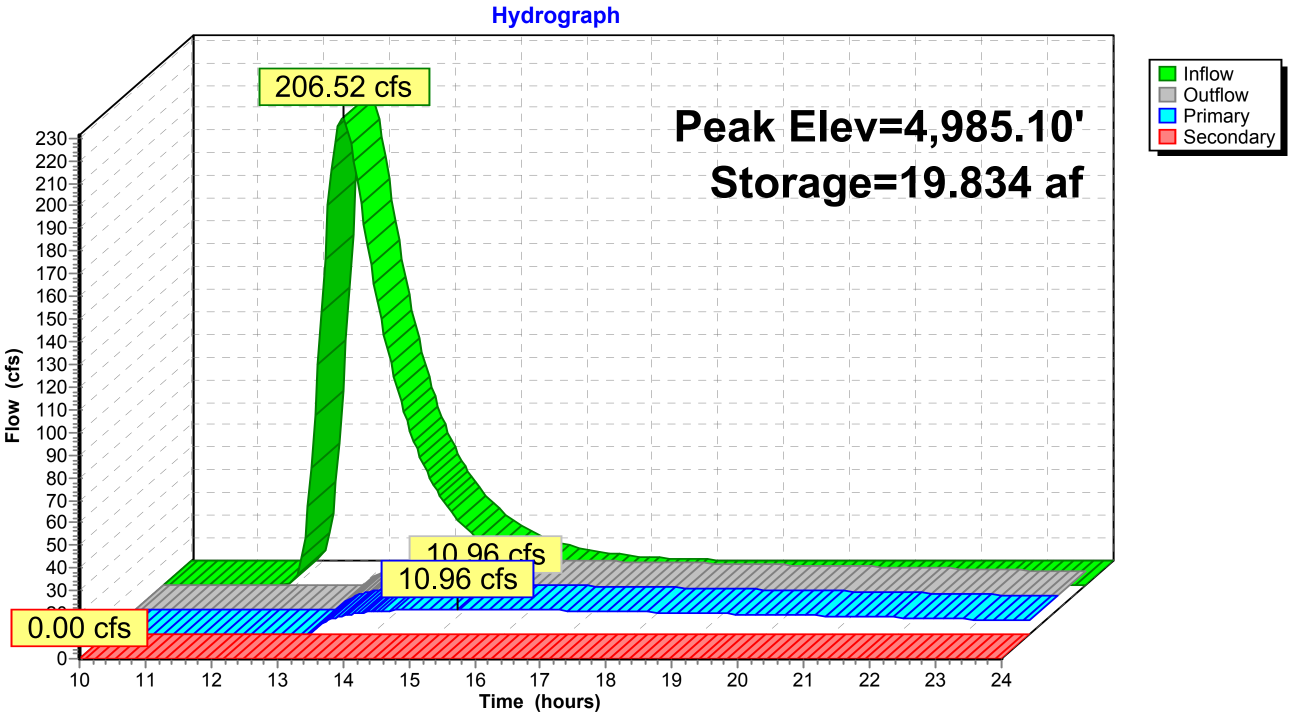
Elevation (feet)	Cum.Store (acre-feet)
4,973.00	0.000
4,974.00	0.300
4,975.00	0.800
4,976.00	1.400
4,977.00	2.200
4,978.00	3.100
4,979.00	4.300
4,980.00	5.700
4,981.00	7.500
4,982.00	9.600
4,983.00	12.300
4,984.00	15.600
4,985.00	19.400
4,986.00	23.700
4,987.00	28.400
4,988.00	33.700
4,989.00	39.500
4,990.00	45.800
4,991.00	52.600
4,992.00	60.100

Device	Routing	Invert	Outlet Devices
#1	Primary	4,973.00'	**18.0" Round Culvert** L= 130.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 4,973.00' / 4,971.00' S= 0.0154 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Secondary	4,988.00'	**25.0' long x 20.0' breadth Emergency Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#3	Device 1	4,982.50'	**12.0" Vert. Orifice/Grate X 2.00** C= 0.600
#4	Device 1	4,986.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

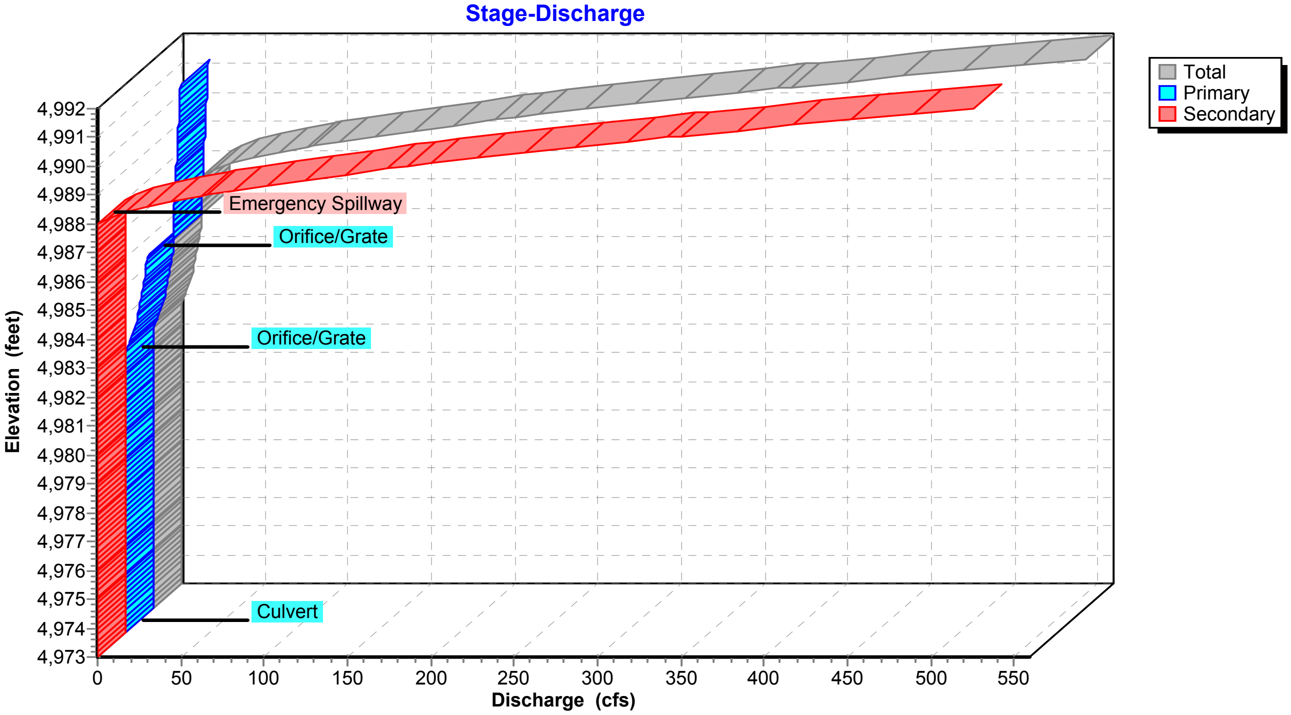
Primary OutFlow Max=10.96 cfs @ 15.31 hrs HW=4,985.10' (Free Discharge)
└─**1=Culvert** (Passes 10.96 cfs of 26.62 cfs potential flow)
  ├─**3=Orifice/Grate** (Orifice Controls 10.96 cfs @ 6.98 fps)
  └─**4=Orifice/Grate** (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 10.00 hrs HW=4,973.00' (Free Discharge)
└─**2=Emergency Spillway** (Controls 0.00 cfs)

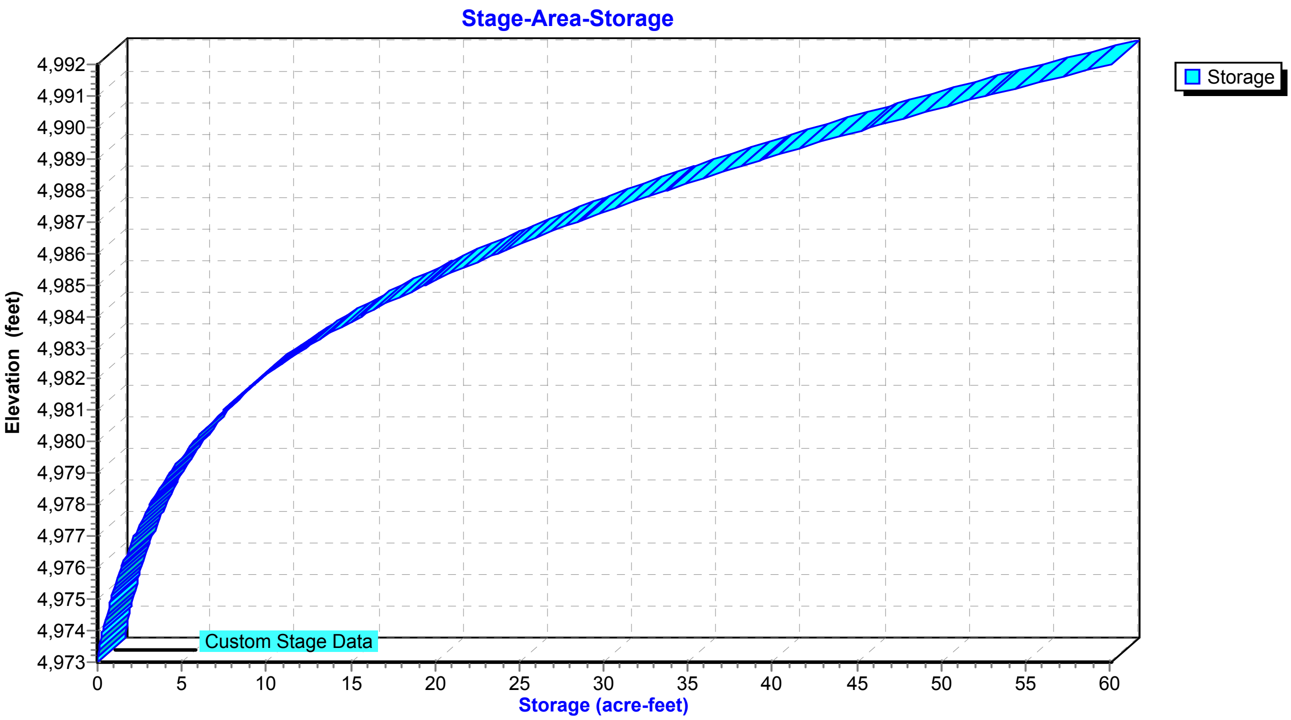
Pond 1P: Stephanie Way Reservoir



Pond 1P: Stephanie Way Reservoir



Pond 1P: Stephanie Way Reservoir



Summary for Pond 1P: Stephanie Way Reservoir

Inflow	=	383.34 cfs @	12.72 hrs,	Volume=	42.563 af
Outflow	=	39.78 cfs @	14.74 hrs,	Volume=	21.780 af, Atten= 90%, Lag= 121.4 min
Primary	=	29.78 cfs @	14.74 hrs,	Volume=	20.807 af
Secondary	=	10.00 cfs @	14.74 hrs,	Volume=	0.973 af

Routing by Stor-Ind method, Time Span= 10.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 4,988.27' @ 14.74 hrs Surf.Area= 0.000 ac Storage= 35.283 af

Plug-Flow detention time= 288.8 min calculated for 21.780 af (51% of inflow)
Center-of-Mass det. time= 253.0 min (1,048.1 - 795.1)

Volume	Invert	Avail.Storage	Storage Description
#1	4,973.00'	60.100 af	**Custom Stage Data** Listed below

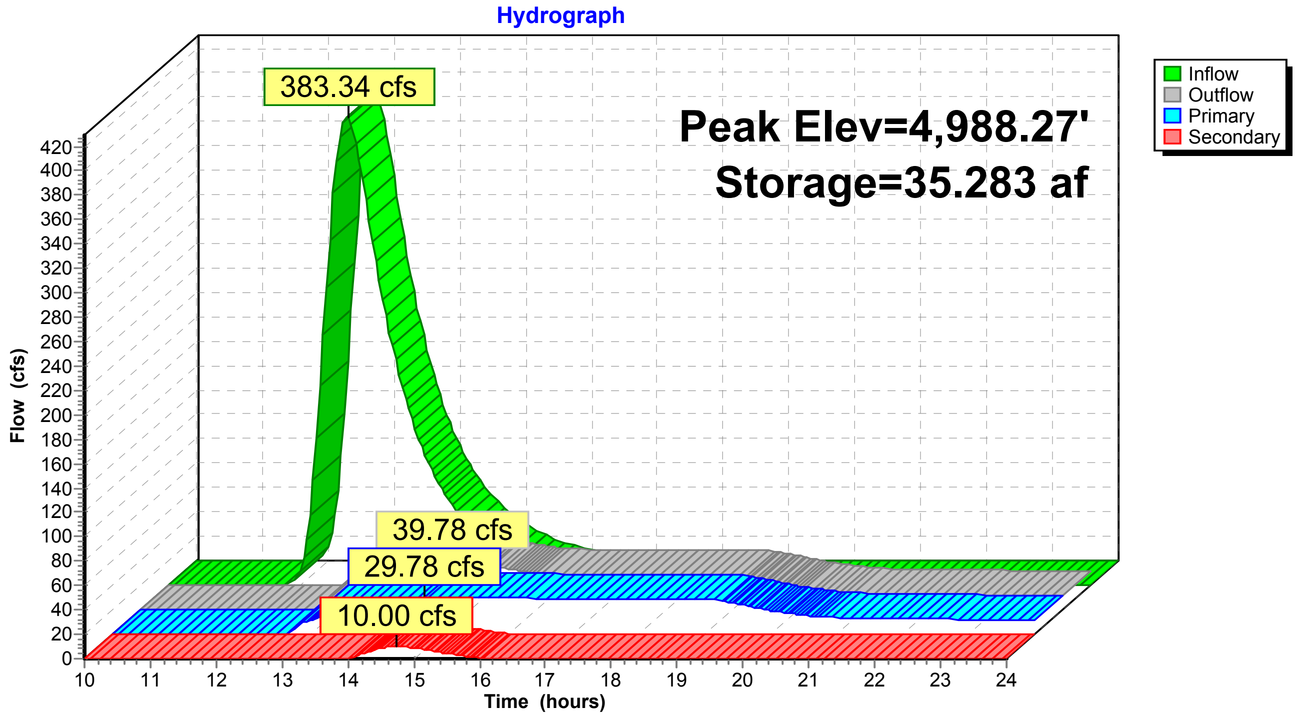
Elevation (feet)	Cum.Store (acre-feet)
4,973.00	0.000
4,974.00	0.300
4,975.00	0.800
4,976.00	1.400
4,977.00	2.200
4,978.00	3.100
4,979.00	4.300
4,980.00	5.700
4,981.00	7.500
4,982.00	9.600
4,983.00	12.300
4,984.00	15.600
4,985.00	19.400
4,986.00	23.700
4,987.00	28.400
4,988.00	33.700
4,989.00	39.500
4,990.00	45.800
4,991.00	52.600
4,992.00	60.100

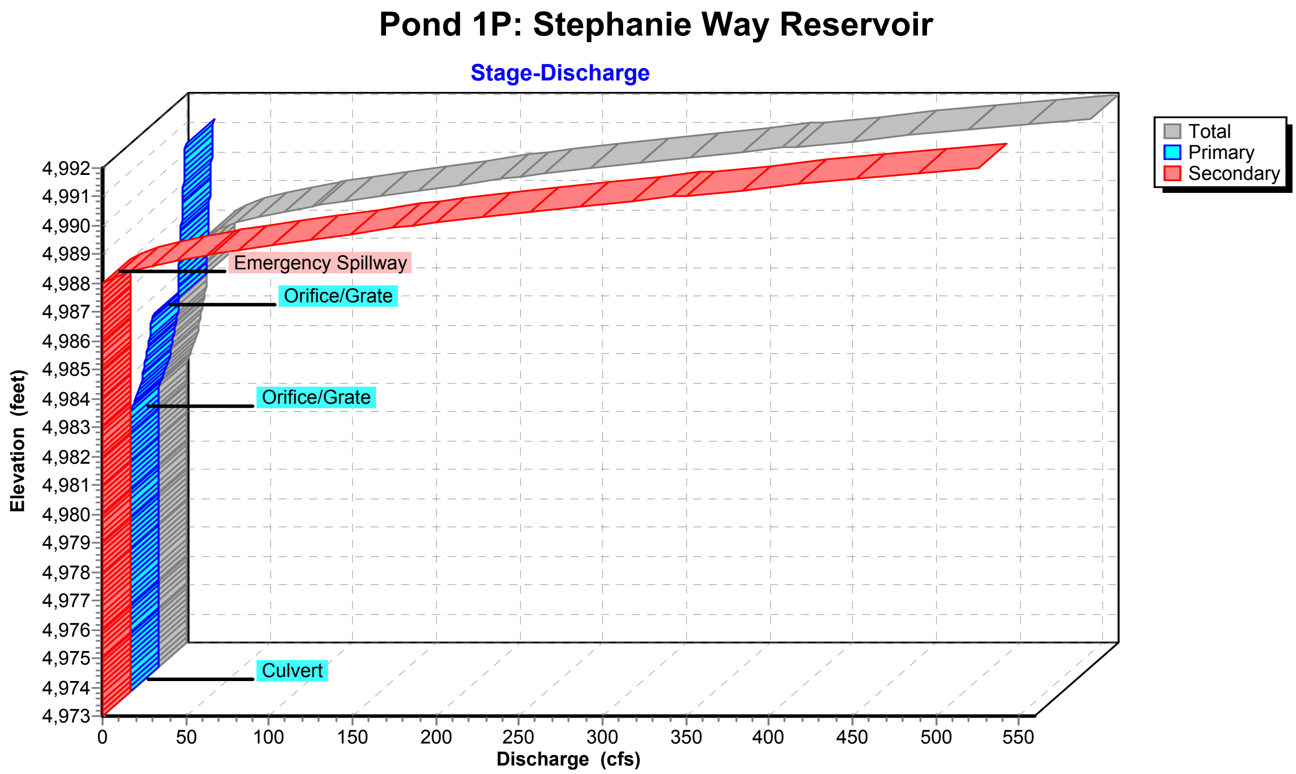
Device	Routing	Invert	Outlet Devices
#1	Primary	4,973.00'	**18.0" Round Culvert** L= 130.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 4,973.00' / 4,971.00' S= 0.0154 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Secondary	4,988.00'	**25.0' long x 20.0' breadth Emergency Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#3	Device 1	4,982.50'	**12.0" Vert. Orifice/Grate X 2.00** C= 0.600
#4	Device 1	4,986.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

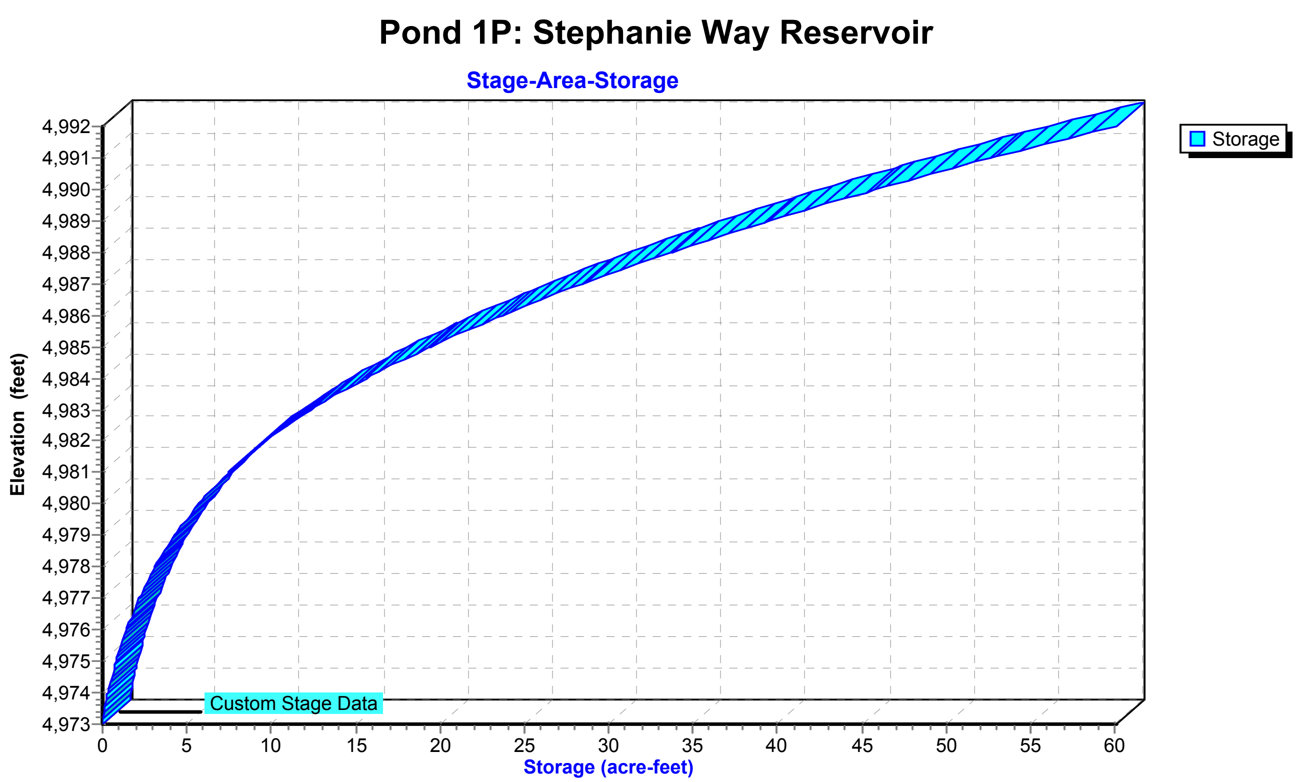
Primary OutFlow Max=29.78 cfs @ 14.74 hrs HW=4,988.27' (Free Discharge)
└─**1=Culvert** (Barrel Controls 29.78 cfs @ 16.85 fps)
  ├─**3=Orifice/Grate** (Passes < 17.37 cfs potential flow)
  └─**4=Orifice/Grate** (Passes < 51.31 cfs potential flow)

Secondary OutFlow Max=9.58 cfs @ 14.74 hrs HW=4,988.27' (Free Discharge)
└─**2=Emergency Spillway** (Weir Controls 9.58 cfs @ 1.40 fps)

Pond 1P: Stephanie Way Reservoir




Pond 1P: Stephanie Way Reservoir
Stage-Discharge
Total
Primary
Secondary
Emergency Spillway
Orifice/Grate
Orifice/Grate
Culvert
Elevation (feet)
4,992
4,991
4,990
4,989
4,988
4,987
4,986
4,985
4,984
4,983
4,982
4,981
4,980
4,979
4,978
4,977
4,976
4,975
4,974
4,973
0
50
100
150
200
250
300
350
400
450
500
550
Discharge (cfs)


Pond 1P: Stephanie Way Reservoir
Stage-Area-Storage
Storage
Custom Stage Data
Elevation (feet)
4,992
4,991
4,990
4,989
4,988
4,987
4,986
4,985
4,984
4,983
4,982
4,981
4,980
4,979
4,978
4,977
4,976
4,975
4,974
4,973
0
5
10
15
20
25
30
35
40
45
50
55
60
Storage (acre-feet)

Summary for Pond 1P: Stephanie Way Reservoir

Inflow	=	421.75 cfs @	12.72 hrs,	Volume=	47.296 af
Outflow	=	66.29 cfs @	14.42 hrs,	Volume=	26.152 af, Atten= 84%, Lag= 102.1 min
Primary	=	30.14 cfs @	14.42 hrs,	Volume=	21.649 af
Secondary	=	36.14 cfs @	14.42 hrs,	Volume=	4.504 af

Routing by Stor-Ind method, Time Span= 10.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 4,988.66' @ 14.42 hrs Surf.Area= 0.000 ac Storage= 37.531 af

Plug-Flow detention time= 265.1 min calculated for 26.152 af (55% of inflow)
Center-of-Mass det. time= 230.8 min (1,026.4 - 795.7)

Volume	Invert	Avail.Storage	Storage Description
#1	4,973.00'	60.100 af	**Custom Stage Data** Listed below

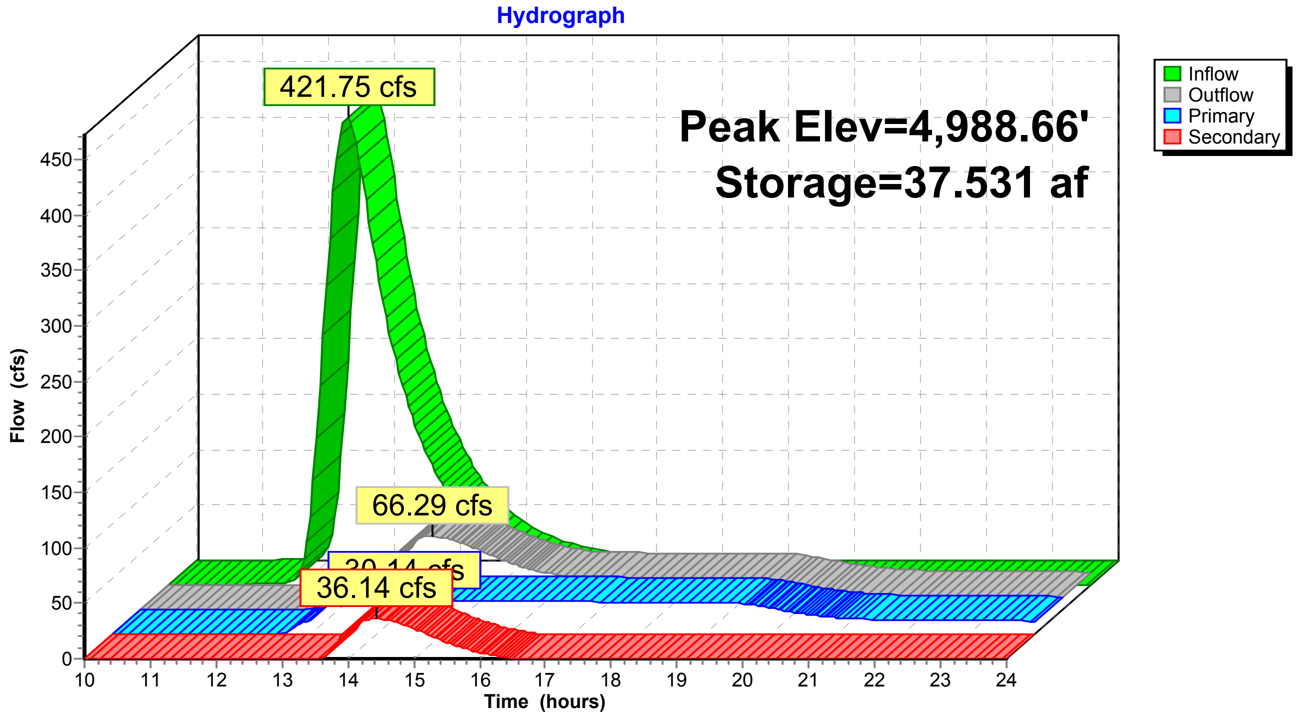
Elevation (feet)	Cum.Store (acre-feet)
4,973.00	0.000
4,974.00	0.300
4,975.00	0.800
4,976.00	1.400
4,977.00	2.200
4,978.00	3.100
4,979.00	4.300
4,980.00	5.700
4,981.00	7.500
4,982.00	9.600
4,983.00	12.300
4,984.00	15.600
4,985.00	19.400
4,986.00	23.700
4,987.00	28.400
4,988.00	33.700
4,989.00	39.500
4,990.00	45.800
4,991.00	52.600
4,992.00	60.100

Device	Routing	Invert	Outlet Devices
#1	Primary	4,973.00'	**18.0" Round Culvert** L= 130.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 4,973.00' / 4,971.00' S= 0.0154 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Secondary	4,988.00'	**25.0' long x 20.0' breadth Emergency Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#3	Device 1	4,982.50'	**12.0" Vert. Orifice/Grate X 2.00** C= 0.600
#4	Device 1	4,986.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

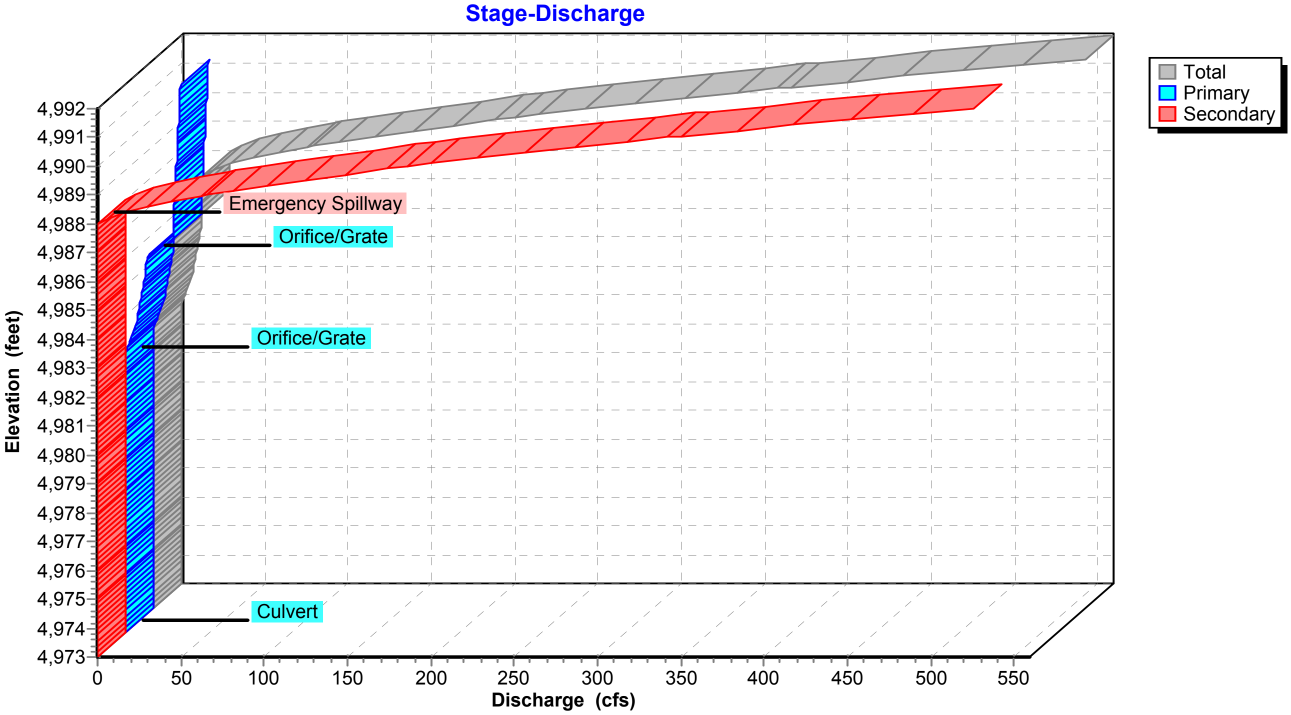
Primary OutFlow Max=30.14 cfs @ 14.42 hrs HW=4,988.66' (Free Discharge)
└**1=Culvert** (Barrel Controls 30.14 cfs @ 17.06 fps)
  ├**3=Orifice/Grate** (Passes < 17.99 cfs potential flow)
  └**4=Orifice/Grate** (Passes < 55.51 cfs potential flow)

Secondary OutFlow Max=35.97 cfs @ 14.42 hrs HW=4,988.66' (Free Discharge)
└**2=Emergency Spillway** (Weir Controls 35.97 cfs @ 2.18 fps)

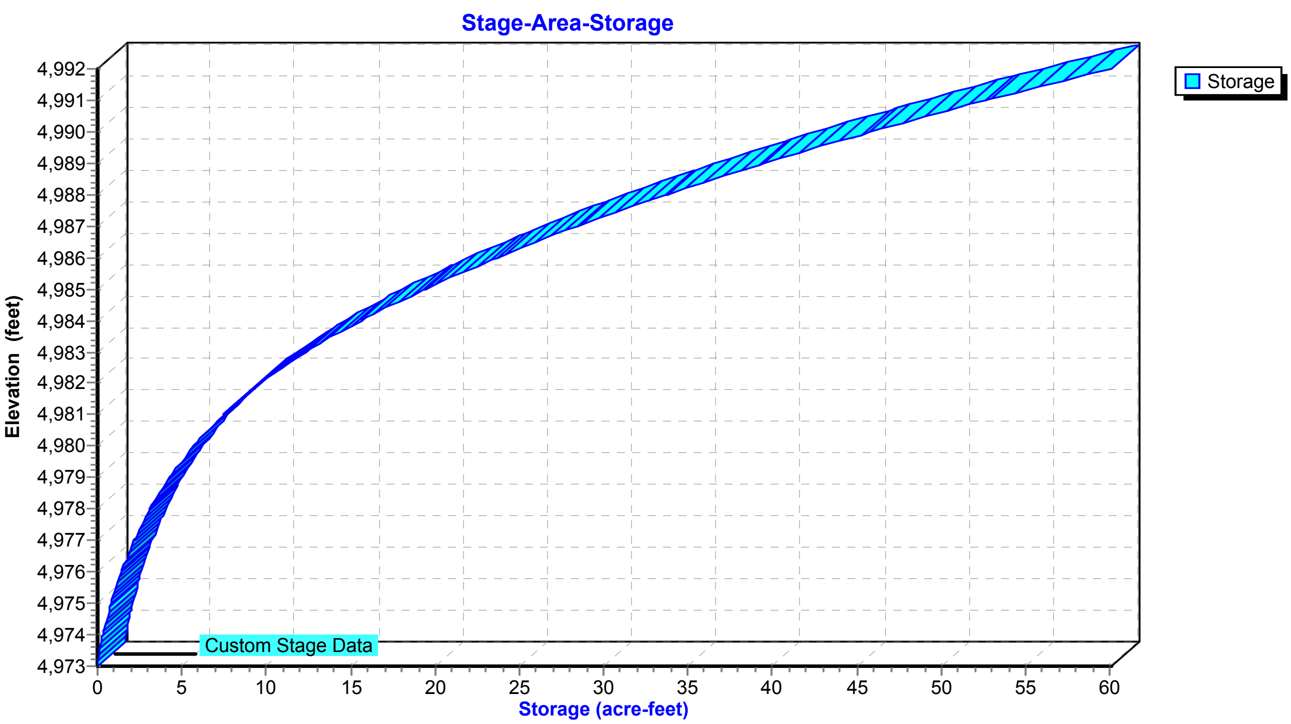
Pond 1P: Stephanie Way Reservoir



Pond 1P: Stephanie Way Reservoir



Pond 1P: Stephanie Way Reservoir



R|O Anderson

APPENDIX 3
HYDRAULIC ANALYSES RESULTS

Cross Section 1: Current Conditions 2-Year Storm Event

Results

Normal Depth		0.72	ft
Elevation Range	4959.87 to 4970.08 ft		
Flow Area		2.51	ft²
Wetted Perimeter		6.92	ft
Hydraulic Radius		0.36	ft
Top Width		6.76	ft
Normal Depth		0.72	ft
Critical Depth		0.63	ft
Critical Slope		0.05516	ft/ft
Velocity		2.39	ft/s
Velocity Head		0.09	ft
Specific Energy		0.81	ft
Froude Number		0.69	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.72	ft
Critical Depth	0.63	ft
Channel Slope	0.02500	ft/ft
Critical Slope	0.05516	ft/ft

Cross Section 1: Current Conditions 2-Year Storm Event

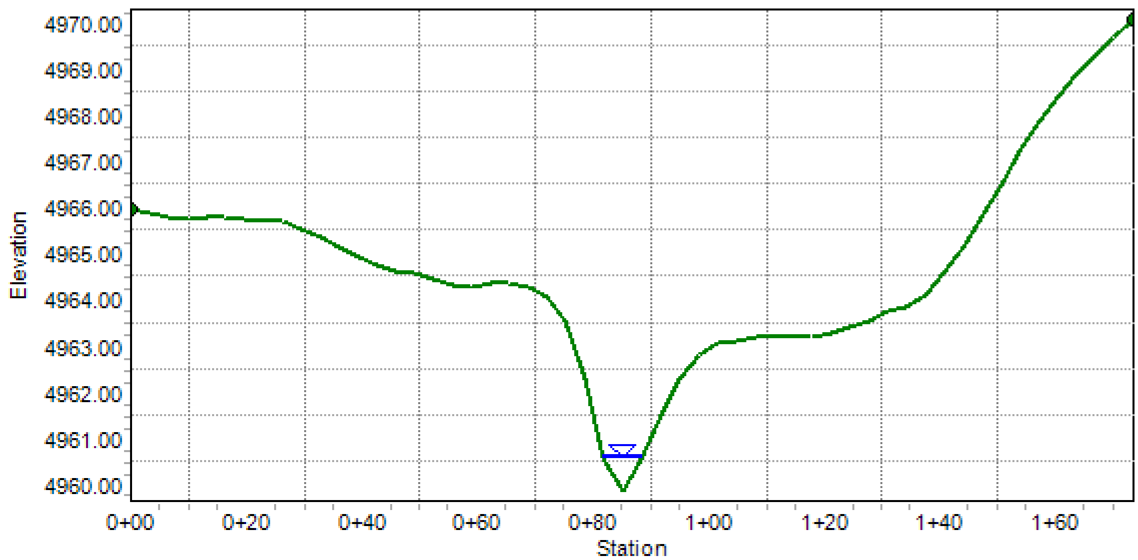
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02500	ft/ft
Normal Depth	0.72	ft
Discharge	6.00	ft^3/s

Cross Section Image



Cross Section 2: Current Conditions 2-Year Storm Event

Results

Elevation Range	4937.61 to 4948.30 ft	
Flow Area	2.54	ft²
Wetted Perimeter	9.56	ft
Hydraulic Radius	0.27	ft
Top Width	9.51	ft
Normal Depth	0.41	ft
Critical Depth	0.36	ft
Critical Slope	0.05878	ft/ft
Velocity	2.36	ft/s
Velocity Head	0.09	ft
Specific Energy	0.49	ft
Froude Number	0.81	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.41	ft
Critical Depth	0.36	ft
Channel Slope	0.03700	ft/ft
Critical Slope	0.05878	ft/ft

Cross Section 2: Current Conditions 2-Year Storm Event

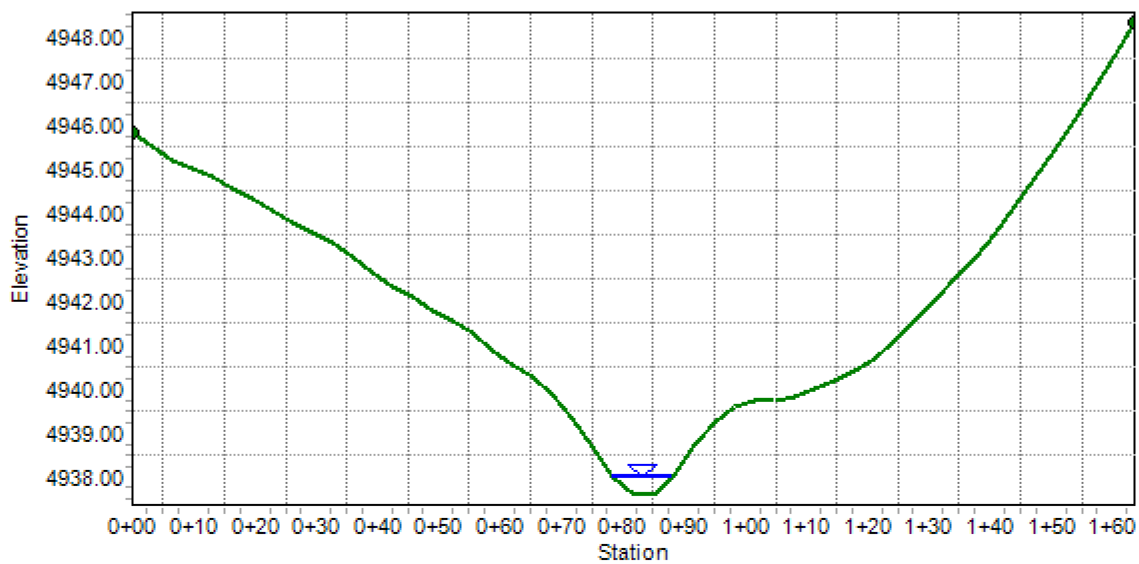
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03700	ft/ft
Normal Depth	0.41	ft
Discharge	6.00	ft³/s

Cross Section Image



Cross Section 3: Current Conditions 2-Year Storm Event

Input Data

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4928.47)	(2+00, 4932.21)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth		0.57	ft
Elevation Range	4924.63 to 4932.21 ft		
Flow Area		2.90	ft²
Wetted Perimeter		8.95	ft
Hydraulic Radius		0.32	ft
Top Width		8.87	ft
Normal Depth		0.57	ft
Critical Depth		0.47	ft
Critical Slope		0.05750	ft/ft
Velocity		2.07	ft/s
Velocity Head		0.07	ft
Specific Energy		0.64	ft
Froude Number		0.64	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s

Cross Section 3: Current Conditions 2-Year Storm Event

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.57	ft
Critical Depth	0.47	ft
Channel Slope	0.02170	ft/ft
Critical Slope	0.05750	ft/ft

Cross Section 3: Current Conditions 2-Year Storm Event

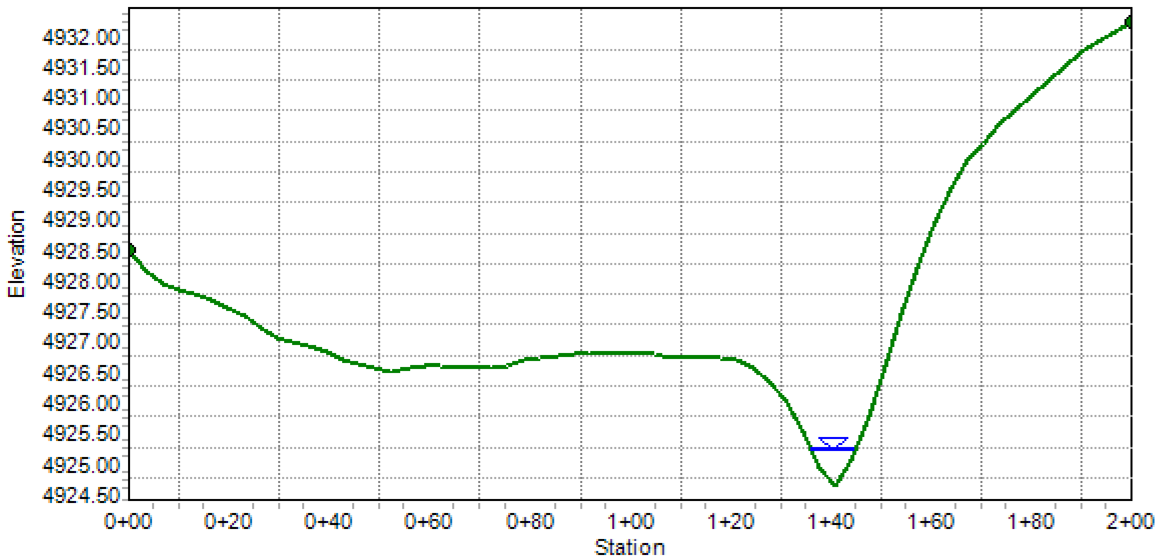
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02170	ft/ft
Normal Depth	0.57	ft
Discharge	6.00	ft³/s

Cross Section Image



Cross Section 1: Current Conditions 10-Year Storm Event

Results

Normal Depth		1.52	ft
Elevation Range	4959.87 to 4970.08 ft		
Flow Area		9.62	ft²
Wetted Perimeter		11.55	ft
Hydraulic Radius		0.83	ft
Top Width		11.09	ft
Normal Depth		1.52	ft
Critical Depth		1.37	ft
Critical Slope		0.04170	ft/ft
Velocity		4.16	ft/s
Velocity Head		0.27	ft
Specific Energy		1.79	ft
Froude Number		0.79	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.52	ft
Critical Depth	1.37	ft
Channel Slope	0.02500	ft/ft
Critical Slope	0.04170	ft/ft

Cross Section 1: Current Conditions 10-Year Storm Event

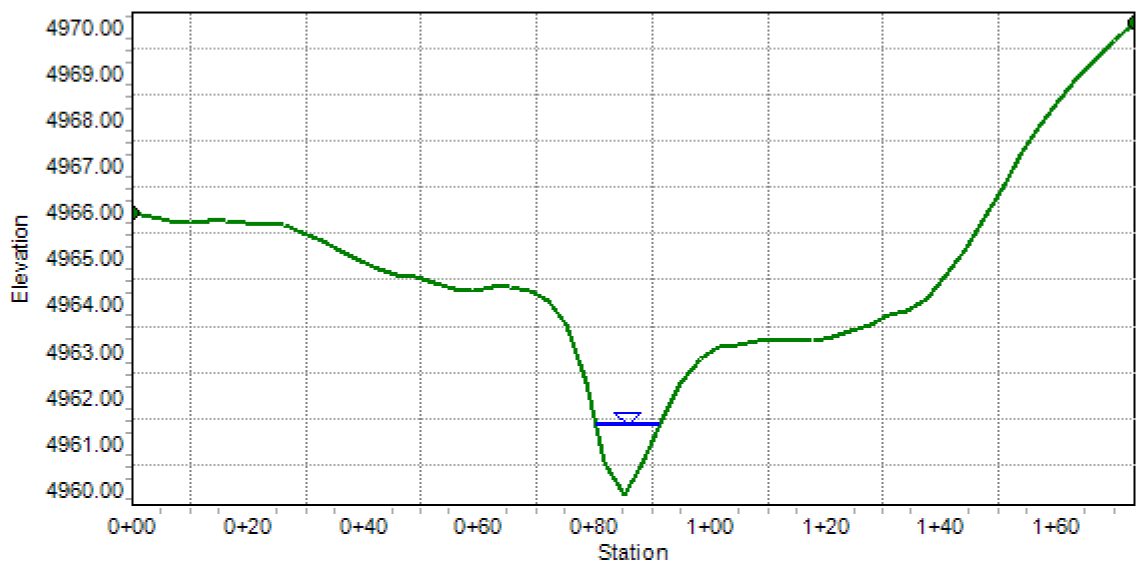
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02500	ft/ft
Normal Depth	1.52	ft
Discharge	40.00	ft^3/s

Cross Section Image



Cross Section 2: Current Conditions 10-Year Storm Event

Results

Elevation Range	4937.61 to 4948.30 ft	
Flow Area	9.56	ft²
Wetted Perimeter	15.28	ft
Hydraulic Radius	0.63	ft
Top Width	15.11	ft
Normal Depth	0.98	ft
Critical Depth	0.94	ft
Critical Slope	0.04354	ft/ft
Velocity	4.18	ft/s
Velocity Head	0.27	ft
Specific Energy	1.25	ft
Froude Number	0.93	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.98	ft
Critical Depth	0.94	ft
Channel Slope	0.03700	ft/ft
Critical Slope	0.04354	ft/ft

Cross Section 2: Current Conditions 10-Year Storm Event

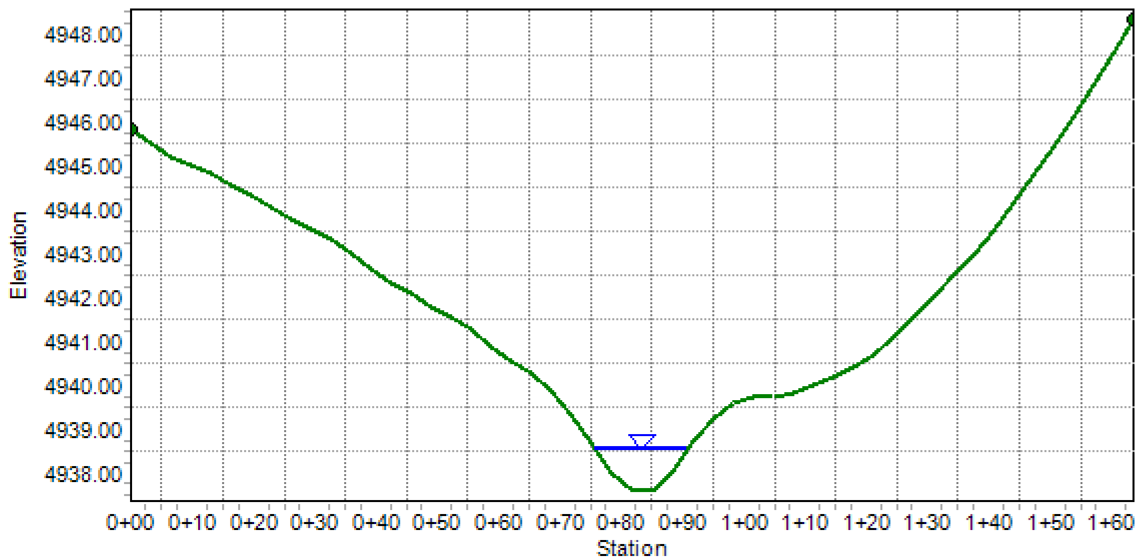
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03700	ft/ft
Normal Depth	0.98	ft
Discharge	40.00	ft³/s

Cross Section Image



Cross Section 3: Current Conditions 10-Year Storm Event

Input Data

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4928.47)	(2+00, 4932.21)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1.26	ft
Elevation Range	4924.63 to 4932.21 ft	
Flow Area	11.49	ft²
Wetted Perimeter	16.21	ft
Hydraulic Radius	0.71	ft
Top Width	15.99	ft
Normal Depth	1.26	ft
Critical Depth	1.09	ft
Critical Slope	0.04332	ft/ft
Velocity	3.48	ft/s
Velocity Head	0.19	ft
Specific Energy	1.45	ft
Froude Number	0.72	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s

Cross Section 3: Current Conditions 10-Year Storm Event

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	1.26	ft
Critical Depth	1.09	ft
Channel Slope	0.02170	ft/ft
Critical Slope	0.04332	ft/ft

Cross Section 3: Current Conditions 10-Year Storm Event

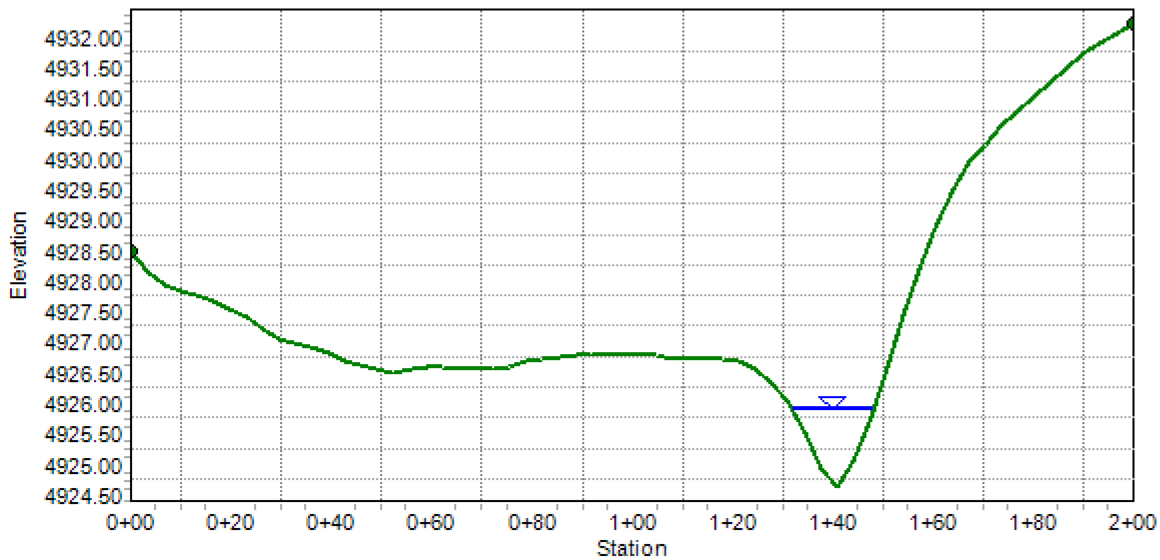
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02170	ft/ft
Normal Depth	1.26	ft
Discharge	40.00	ft^3/s

Cross Section Image



Cross Section 1: Current Conditions 25-Year Storm Event

Results

Normal Depth		1.93	ft
Elevation Range	4959.87 to 4970.08 ft		
Flow Area		14.71	ft²
Wetted Perimeter		14.12	ft
Hydraulic Radius		1.04	ft
Top Width		13.51	ft
Normal Depth		1.93	ft
Critical Depth		1.77	ft
Critical Slope		0.03860	ft/ft
Velocity		4.83	ft/s
Velocity Head		0.36	ft
Specific Energy		2.30	ft
Froude Number		0.82	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.93	ft
Critical Depth	1.77	ft
Channel Slope	0.02500	ft/ft
Critical Slope	0.03860	ft/ft

Cross Section 1: Current Conditions 25-Year Storm Event

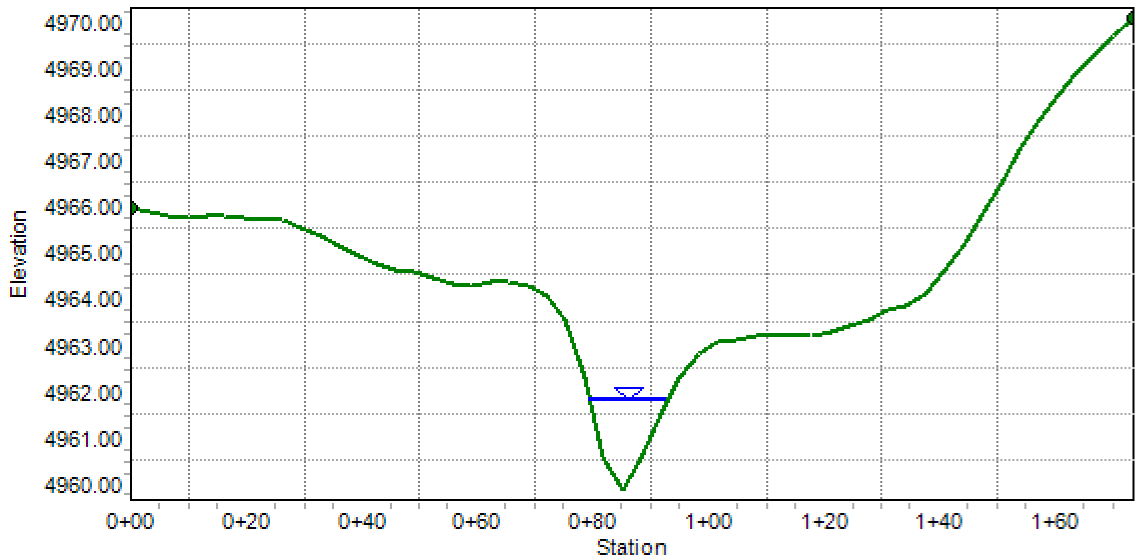
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02500	ft/ft
Normal Depth	1.93	ft
Discharge	71.00	ft^3/s

Cross Section Image



Cross Section 2: Current Conditions 25-Year Storm Event

Results

Elevation Range	4937.61 to 4948.30 ft	
Flow Area	14.56	ft²
Wetted Perimeter	18.48	ft
Hydraulic Radius	0.79	ft
Top Width	18.26	ft
Normal Depth	1.28	ft
Critical Depth	1.25	ft
Critical Slope	0.04013	ft/ft
Velocity	4.87	ft/s
Velocity Head	0.37	ft
Specific Energy	1.65	ft
Froude Number	0.96	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.28	ft
Critical Depth	1.25	ft
Channel Slope	0.03700	ft/ft
Critical Slope	0.04013	ft/ft

Cross Section 2: Current Conditions 25-Year Storm Event

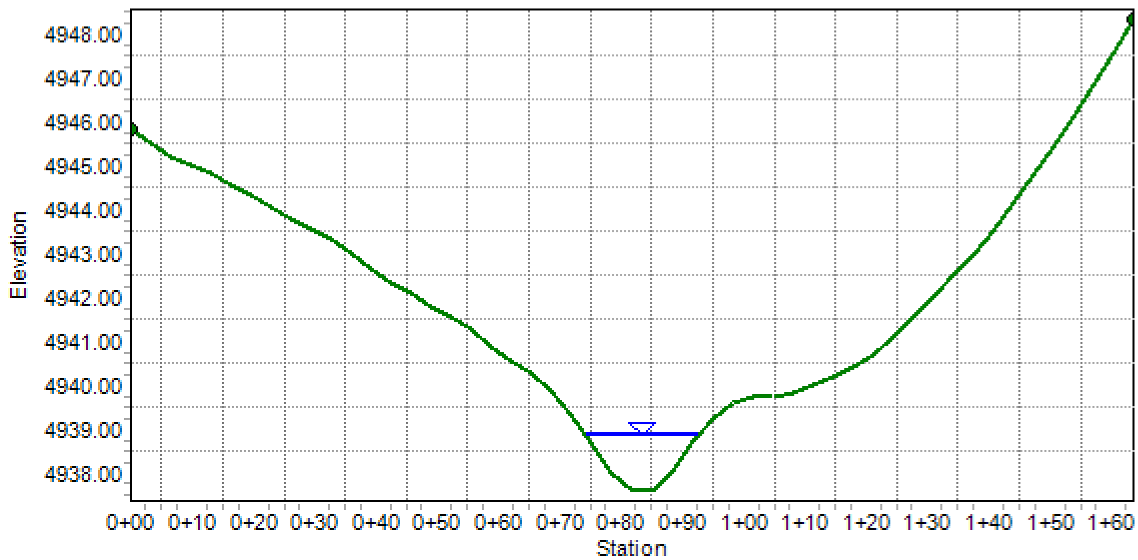
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03700	ft/ft
Normal Depth	1.28	ft
Discharge	71.00	ft³/s

Cross Section Image



Cross Section 3: Current Conditions 25-Year Storm Event

Input Data

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4928.47)	(2+00, 4932.21)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth		1.62	ft
Elevation Range	4924.63 to 4932.21 ft		
Flow Area		17.94	ft²
Wetted Perimeter		20.86	ft
Hydraulic Radius		0.86	ft
Top Width		20.58	ft
Normal Depth		1.62	ft
Critical Depth		1.41	ft
Critical Slope		0.04002	ft/ft
Velocity		3.96	ft/s
Velocity Head		0.24	ft
Specific Energy		1.86	ft
Froude Number		0.75	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s

Cross Section 3: Current Conditions 25-Year Storm Event

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	1.62	ft
Critical Depth	1.41	ft
Channel Slope	0.02170	ft/ft
Critical Slope	0.04002	ft/ft

Cross Section 3: Current Conditions 25-Year Storm Event

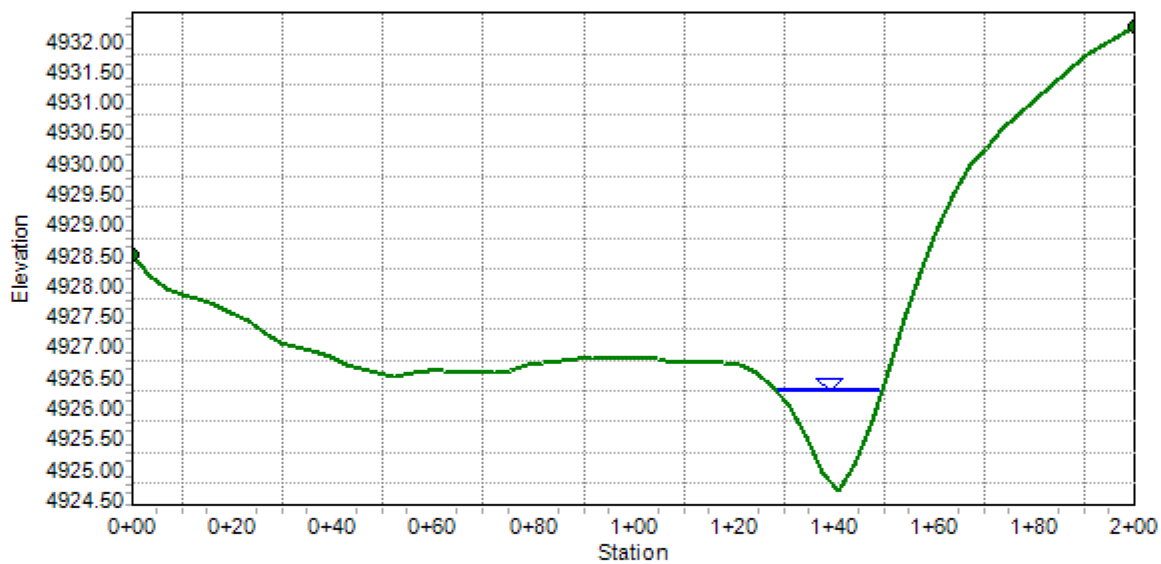
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02170	ft/ft
Normal Depth	1.62	ft
Discharge	71.00	ft³/s

Cross Section Image



Cross Section 1: Current Conditions 100-Year Storm Event

Results

Normal Depth		2.55	ft
Elevation Range	4959.87 to 4970.08 ft		
Flow Area		24.13	ft²
Wetted Perimeter		18.36	ft
Hydraulic Radius		1.31	ft
Top Width		17.53	ft
Normal Depth		2.55	ft
Critical Depth		2.36	ft
Critical Slope		0.03548	ft/ft
Velocity		5.64	ft/s
Velocity Head		0.49	ft
Specific Energy		3.04	ft
Froude Number		0.85	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.55	ft
Critical Depth	2.36	ft
Channel Slope	0.02500	ft/ft
Critical Slope	0.03548	ft/ft

Cross Section 1: Current Conditions 100-Year Storm Event

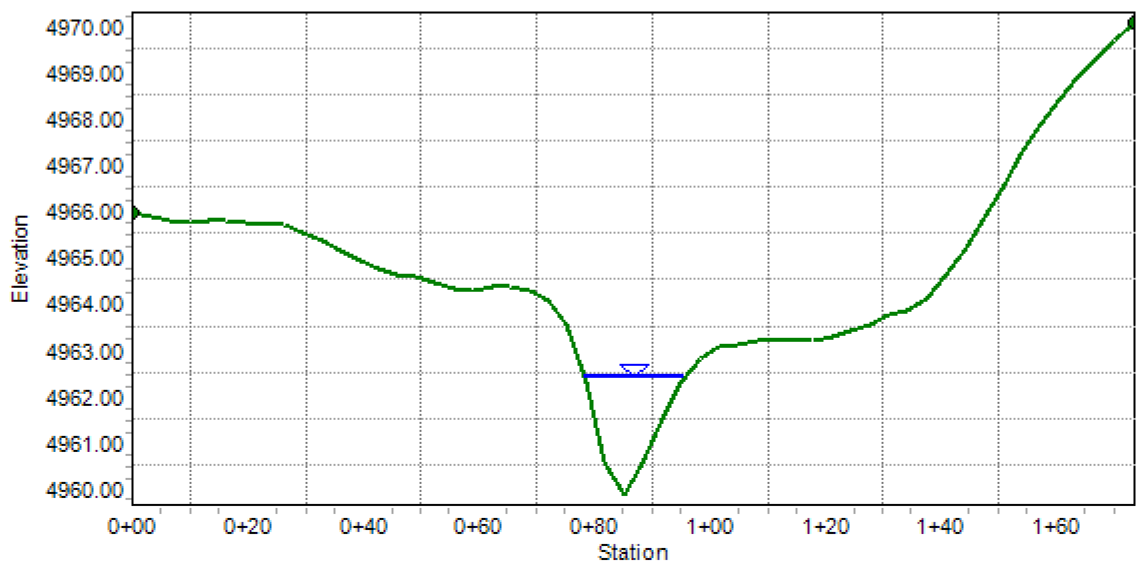
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02500	ft/ft
Normal Depth	2.55	ft
Discharge	136.00	ft³/s

Cross Section Image



Cross Section 2: Current Conditions 100-Year Storm Event

Results

Elevation Range	4937.61 to 4948.30 ft	
Flow Area	23.77	ft²
Wetted Perimeter	23.73	ft
Hydraulic Radius	1.00	ft
Top Width	23.43	ft
Normal Depth	1.72	ft
Critical Depth	1.72	ft
Critical Slope	0.03687	ft/ft
Velocity	5.72	ft/s
Velocity Head	0.51	ft
Specific Energy	2.23	ft
Froude Number	1.00	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.72	ft
Critical Depth	1.72	ft
Channel Slope	0.03700	ft/ft
Critical Slope	0.03687	ft/ft

Cross Section 2: Current Conditions 100-Year Storm Event

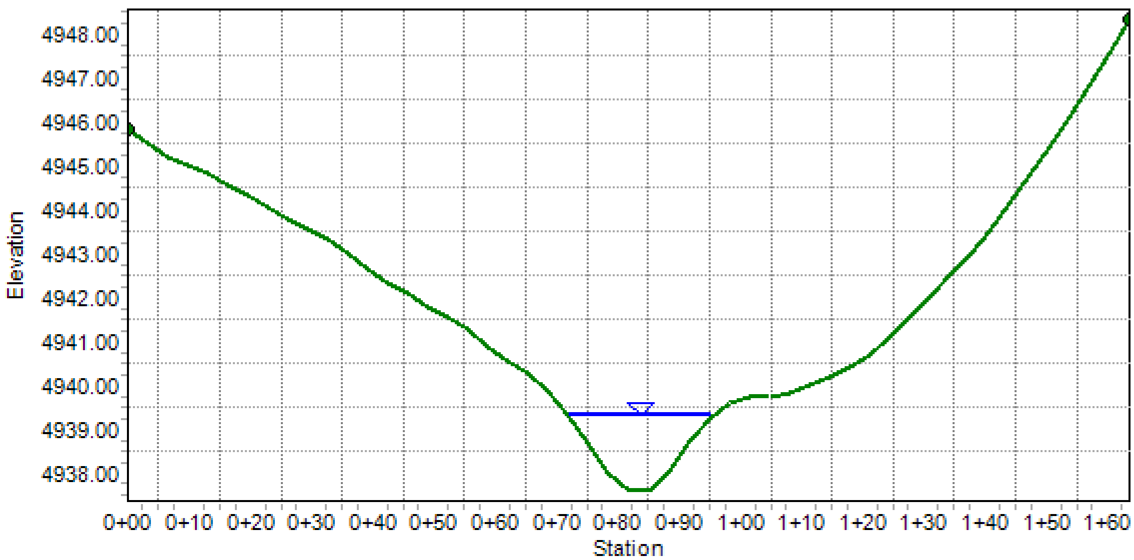
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03700	ft/ft
Normal Depth	1.72	ft
Discharge	136.00	ft³/s

Cross Section Image



Cross Section 3: Current Conditions 100-Year Storm Event

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	2.25	ft
Critical Depth	2.11	ft
Channel Slope	0.02170	ft/ft
Critical Slope	0.04876	ft/ft

Cross Section 3: Current Conditions 100-Year Storm Event

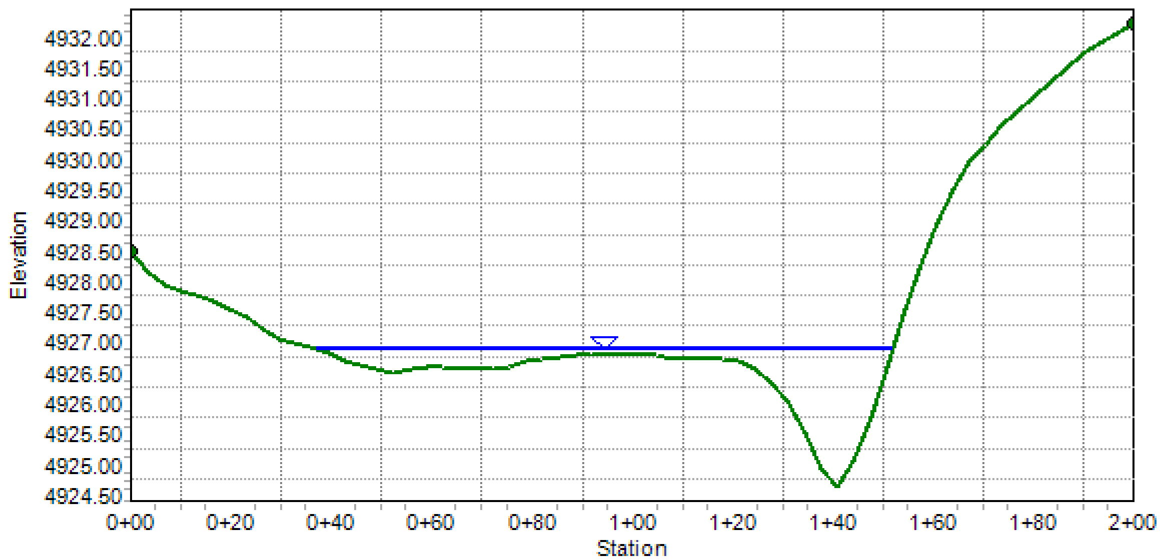
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02170	ft/ft
Normal Depth	2.25	ft
Discharge	136.00	ft³/s

Cross Section Image



Cross Section 1: Current Conditions 500-Year Storm Event

Results

Normal Depth		3.57	ft
Elevation Range	4959.87 to 4970.08 ft		
Flow Area		52.36	ft²
Wetted Perimeter		50.49	ft
Hydraulic Radius		1.04	ft
Top Width		49.42	ft
Normal Depth		3.57	ft
Critical Depth		3.40	ft
Critical Slope		0.03751	ft/ft
Velocity		4.81	ft/s
Velocity Head		0.36	ft
Specific Energy		3.93	ft
Froude Number		0.82	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	3.57	ft
Critical Depth	3.40	ft
Channel Slope	0.02500	ft/ft
Critical Slope	0.03751	ft/ft

Cross Section 1: Current Conditions 500-Year Storm Event

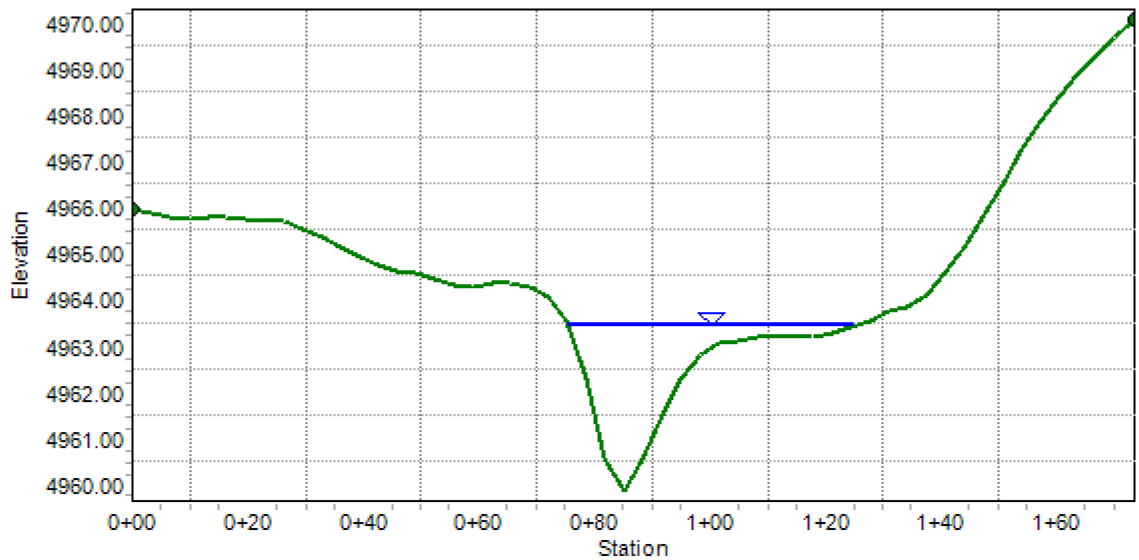
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02500	ft/ft
Normal Depth	3.57	ft
Discharge	252.00	ft³/s

Cross Section Image



Cross Section 2: Current Conditions 500-Year Storm Event

Results

Elevation Range	4937.61 to 4948.30 ft	
Flow Area	43.30	ft^2
Wetted Perimeter	42.18	ft
Hydraulic Radius	1.03	ft
Top Width	41.81	ft
Normal Depth	2.35	ft
Critical Depth	2.36	ft
Critical Slope	0.03640	ft/ft
Velocity	5.82	ft/s
Velocity Head	0.53	ft
Specific Energy	2.88	ft
Froude Number	1.01	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.35	ft
Critical Depth	2.36	ft
Channel Slope	0.03700	ft/ft
Critical Slope	0.03640	ft/ft

Cross Section 2: Current Conditions 500-Year Storm Event

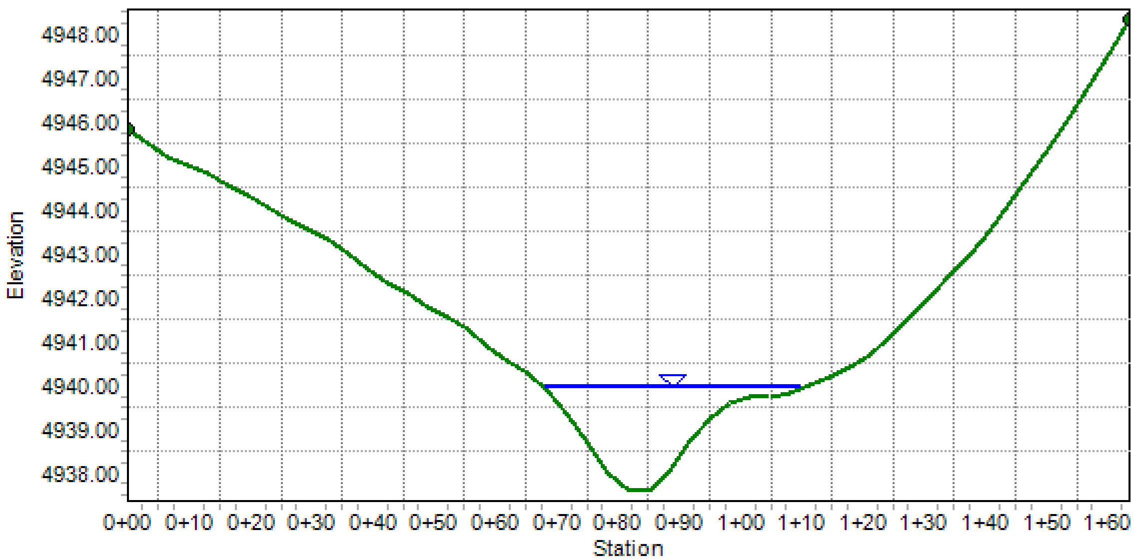
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03700	ft/ft
Normal Depth	2.35	ft
Discharge	252.00	ft³/s

Cross Section Image



Cross Section 3: Current Conditions 500-Year Storm Event

Input Data

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4928.47)	(2+00, 4932.21)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	2.47	ft
Elevation Range	4924.63 to 4932.21 ft	
Flow Area	78.41	ft²
Wetted Perimeter	124.66	ft
Hydraulic Radius	0.63	ft
Top Width	124.24	ft
Normal Depth	2.47	ft
Critical Depth	2.33	ft
Critical Slope	0.04549	ft/ft
Velocity	3.21	ft/s
Velocity Head	0.16	ft
Specific Energy	2.63	ft
Froude Number	0.71	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s

Cross Section 3: Current Conditions 500-Year Storm Event

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	2.47	ft
Critical Depth	2.33	ft
Channel Slope	0.02170	ft/ft
Critical Slope	0.04549	ft/ft

Cross Section 3: Current Conditions 500-Year Storm Event

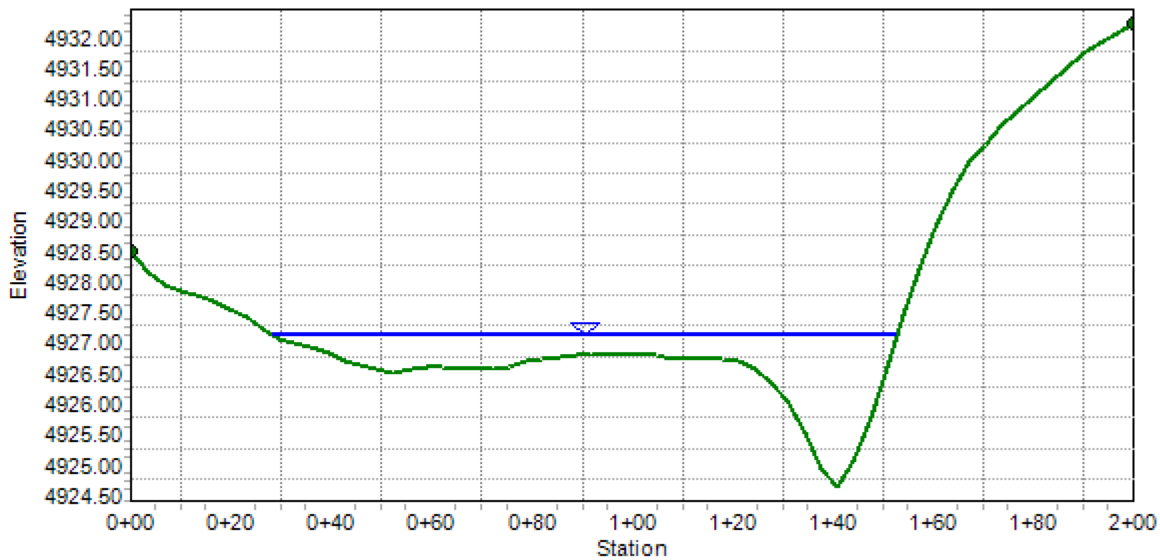
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02170	ft/ft
Normal Depth	2.47	ft
Discharge	252.00	ft^3/s

Cross Section Image



Cross Section 1: Proposed Conditions 100-Year Storm Event

Results

Normal Depth		0.90	ft
Elevation Range	4959.87 to 4970.08 ft		
Flow Area		3.82	ft²
Wetted Perimeter		7.97	ft
Hydraulic Radius		0.48	ft
Top Width		7.74	ft
Normal Depth		0.90	ft
Critical Depth		0.79	ft
Critical Slope		0.05035	ft/ft
Velocity		2.88	ft/s
Velocity Head		0.13	ft
Specific Energy		1.03	ft
Froude Number		0.72	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.90	ft
Critical Depth	0.79	ft
Channel Slope	0.02500	ft/ft
Critical Slope	0.05035	ft/ft

Cross Section 1: Proposed Conditions 100-Year Storm Event

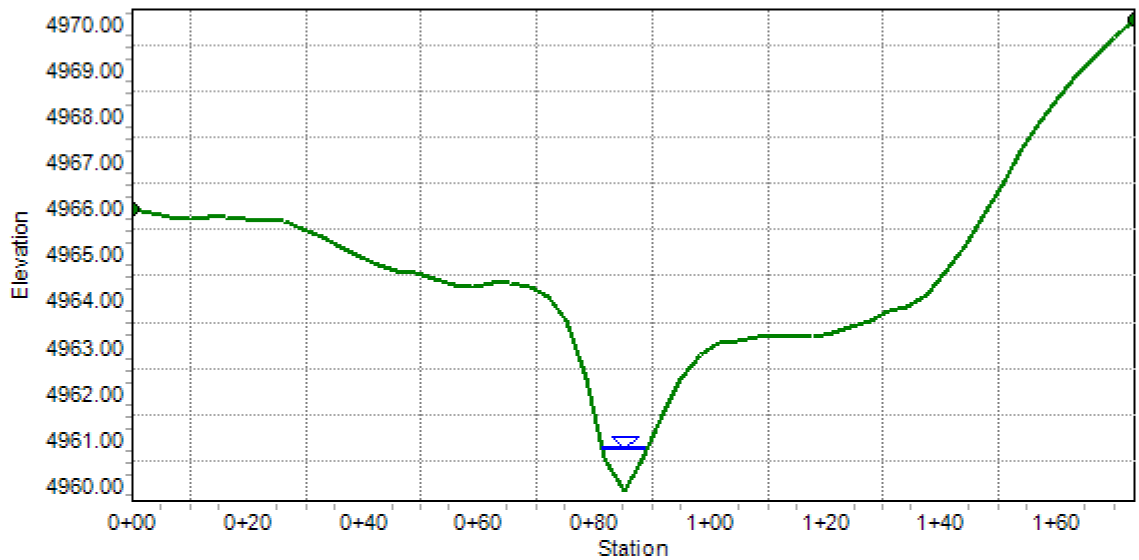
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02500	ft/ft
Normal Depth	0.90	ft
Discharge	11.00	ft³/s

Cross Section Image



Cross Section 2: Proposed Conditions 100-Year Storm Event

Results

Elevation Range	4937.61 to 4948.30 ft	
Flow Area	3.86	ft^2
Wetted Perimeter	10.94	ft
Hydraulic Radius	0.35	ft
Top Width	10.86	ft
Normal Depth	0.54	ft
Critical Depth	0.49	ft
Critical Slope	0.05341	ft/ft
Velocity	2.85	ft/s
Velocity Head	0.13	ft
Specific Energy	0.66	ft
Froude Number	0.84	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.54	ft
Critical Depth	0.49	ft
Channel Slope	0.03700	ft/ft
Critical Slope	0.05341	ft/ft

Cross Section 2: Proposed Conditions 100-Year Storm Event

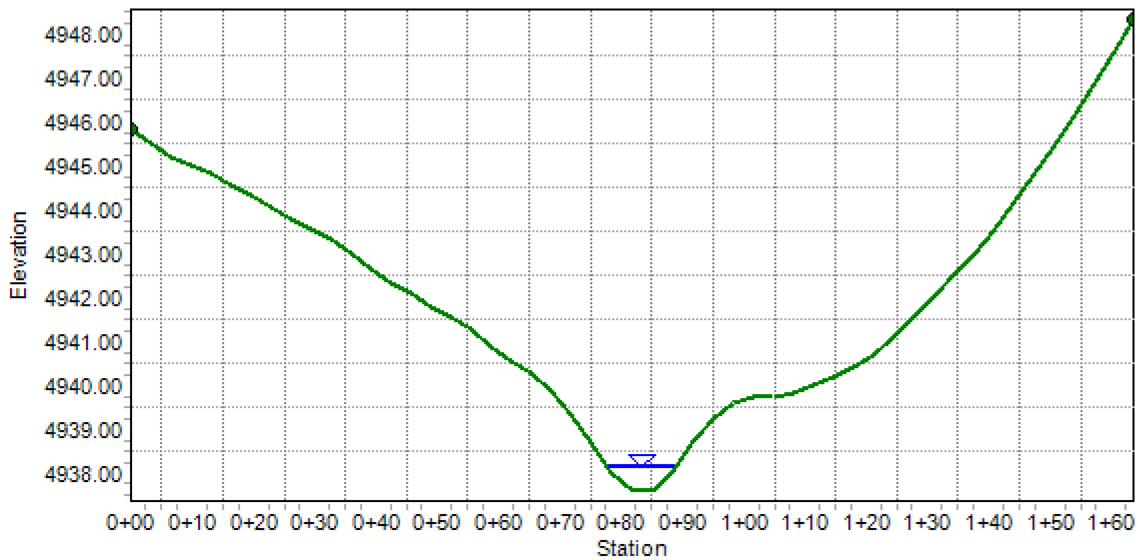
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03700	ft/ft
Normal Depth	0.54	ft
Discharge	11.00	ft³/s

Cross Section Image



Cross Section 3: Proposed Conditions 100-Year Storm Event

Input Data

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4928.47)	(2+00, 4932.21)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	0.73	ft
Elevation Range	4924.63 to 4932.21 ft	
Flow Area	4.47	ft²
Wetted Perimeter	10.58	ft
Hydraulic Radius	0.42	ft
Top Width	10.47	ft
Normal Depth	0.73	ft
Critical Depth	0.61	ft
Critical Slope	0.05223	ft/ft
Velocity	2.46	ft/s
Velocity Head	0.09	ft
Specific Energy	0.82	ft
Froude Number	0.66	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s

Cross Section 3: Proposed Conditions 100-Year Storm Event

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.73	ft
Critical Depth	0.61	ft
Channel Slope	0.02170	ft/ft
Critical Slope	0.05223	ft/ft

Cross Section 3: Proposed Conditions 100-Year Storm Event

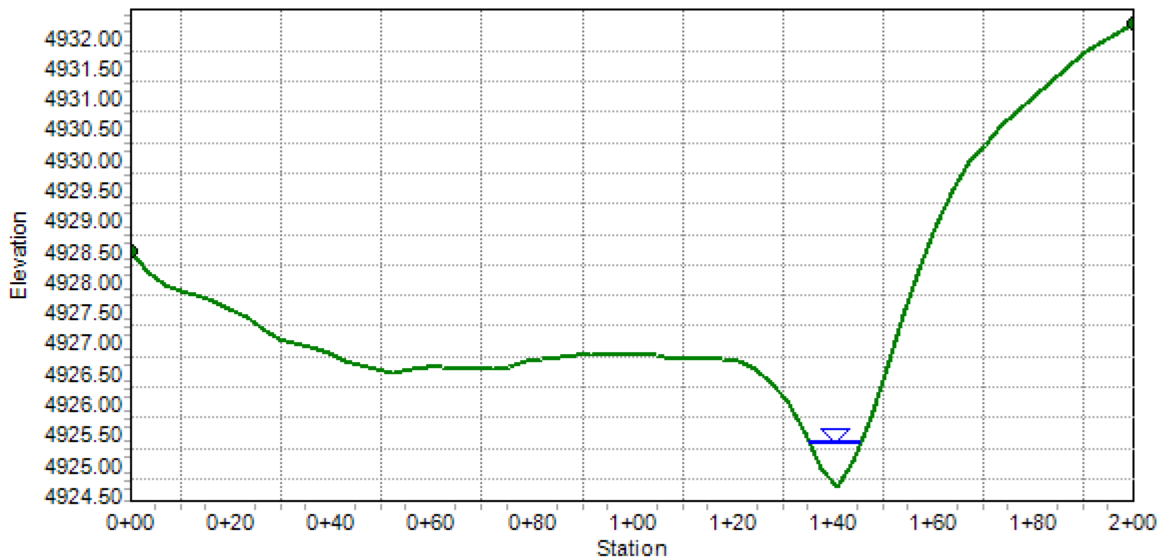
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02170	ft/ft
Normal Depth	0.73	ft
Discharge	11.00	ft³/s

Cross Section Image



Cross Section 1: Proposed Conditions 500-Year Storm Event

Results

Normal Depth		1.54	ft
Elevation Range	4959.87 to 4970.08 ft		
Flow Area		9.81	ft²
Wetted Perimeter		11.65	ft
Hydraulic Radius		0.84	ft
Top Width		11.18	ft
Normal Depth		1.54	ft
Critical Depth		1.38	ft
Critical Slope		0.04154	ft/ft
Velocity		4.19	ft/s
Velocity Head		0.27	ft
Specific Energy		1.81	ft
Froude Number		0.79	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.54	ft
Critical Depth	1.38	ft
Channel Slope	0.02500	ft/ft
Critical Slope	0.04154	ft/ft

Cross Section 1: Proposed Conditions 500-Year Storm Event

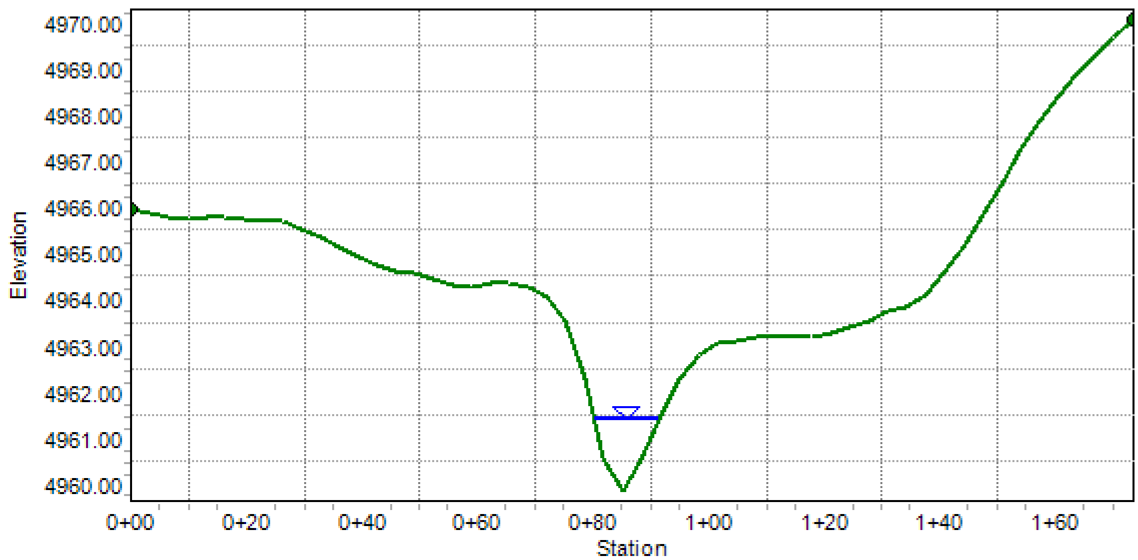
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02500	ft/ft
Normal Depth	1.54	ft
Discharge	41.10	ft³/s

Cross Section Image



Cross Section 2: Proposed Conditions 500-Year Storm Event

Results

Elevation Range	4937.61 to 4948.30 ft	
Flow Area	9.75	ft^2
Wetted Perimeter	15.40	ft
Hydraulic Radius	0.63	ft
Top Width	15.23	ft
Normal Depth	0.99	ft
Critical Depth	0.95	ft
Critical Slope	0.04337	ft/ft
Velocity	4.21	ft/s
Velocity Head	0.28	ft
Specific Energy	1.26	ft
Froude Number	0.93	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.99	ft
Critical Depth	0.95	ft
Channel Slope	0.03700	ft/ft
Critical Slope	0.04337	ft/ft

Cross Section 2: Proposed Conditions 500-Year Storm Event

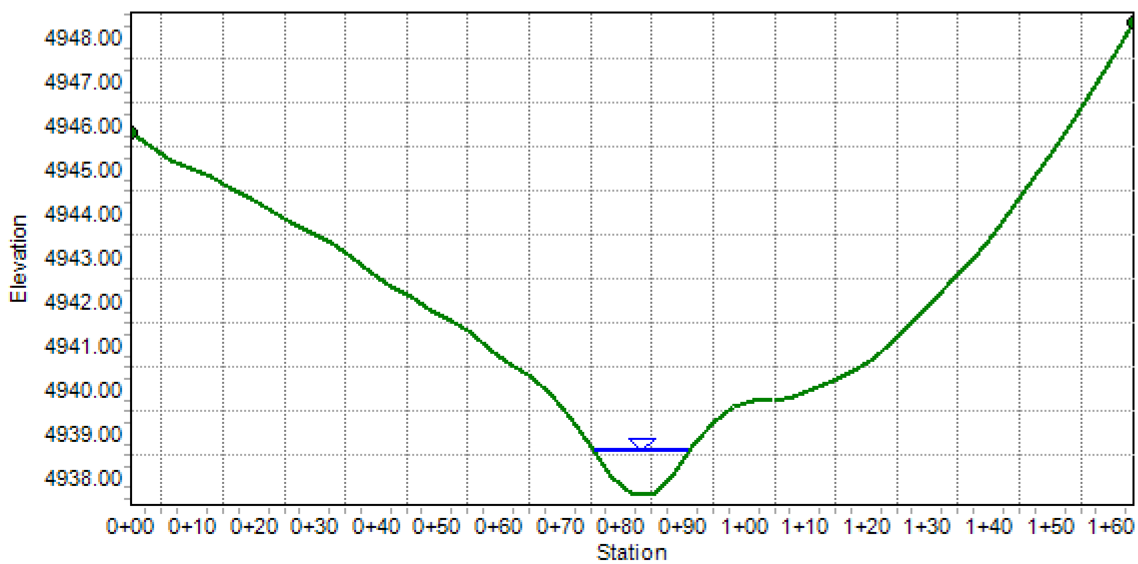
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03700	ft/ft
Normal Depth	0.99	ft
Discharge	41.10	ft^3/s

Cross Section Image



Cross Section 3: Proposed Conditions 500-Year Storm Event

Input Data

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4928.47)	(2+00, 4932.21)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1.28	ft
Elevation Range	4924.63 to 4932.21 ft	
Flow Area	11.72	ft²
Wetted Perimeter	16.36	ft
Hydraulic Radius	0.72	ft
Top Width	16.14	ft
Normal Depth	1.28	ft
Critical Depth	1.11	ft
Critical Slope	0.04316	ft/ft
Velocity	3.51	ft/s
Velocity Head	0.19	ft
Specific Energy	1.47	ft
Froude Number	0.73	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s

Cross Section 3: Proposed Conditions 500-Year Storm Event

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	1.28	ft
Critical Depth	1.11	ft
Channel Slope	0.02170	ft/ft
Critical Slope	0.04316	ft/ft

Cross Section 3: Proposed Conditions 500-Year Storm Event

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02170	ft/ft
Normal Depth	1.28	ft
Discharge	41.10	ft³/s

Cross Section Image