Appendix D

East Locust Creek Reservoir Water Budget Model

EAST LOCUST CREEK RESERVOIR
WATER BUDGET MODEL

PREPARED FOR

NORTH CENTRAL MISSOURI REGIONAL WATER COMMISSION

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Appendix

A - Water Budget Model – Varying Water Supply Demand Rates
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C – East Locust Creek Reservoir Hydrology Report

Acronyms and Initialisms

Term	Definition
CFS	Cubic Feet per Second
EIS	Environmental Impact Statement
ET	Evapotranspiration
ELCR	East Locust Creek Reservoir
MDC	Missouri Department of Conservation
MDNR	Missouri Department of Natural Resources
MGD	Million Gallons per Day
NCMRWC	North Central Missouri Regional Water Commission
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
URM	Unit Runoff Method
WBM	Water Budget Model

1.0 INTRODUCTION AND BACKGROUND

The proposed East Locust Creek Reservoir (ELCR) project in Sullivan County Missouri will create a 2,328-acre multi-purpose reservoir designed to provide drinking water, recreation, and flood damage reduction for a 10-county region in north central Missouri. The reservoir is sized to provide 7 million gallons a day (MGD) to the 10-county region in north central Missouri. The reservoir is designed to provide a reliable drinking water source during the drought of record. The current proposed reservoir location, reservoir size, and the drinking water capacity of 7 MGD was established in 2007 in the "East Locust Creek Watershed Revised Plan and Environmental Impact Statement" (EIS) (NRCS 2007).

Because the 10-county area is one of the poorest regions in the state and because existing drinking water rates are among the highest in the state, the residents of the region currently pay an inordinately high percentage of their household income for drinking water. For this reason, it is also critical that the reservoir provide high quality recreational opportunities to help subsidize drinking water production. An important constraint in designing recreational opportunities that will attract consumers to the reservoir is an understanding of how the lake levels will fluctuate through typically varying weather patterns.

1.1 Water Budget Model

The ELCR water budget model is intended to estimate the amount of drinking water the proposed ELCR can provide during a drought equivalent to the drought of record while considering watershed runoff, rainfall, downstream flows, seepage, and evaporation and without excluding the other two project purposes (recreation and flood damage reduction). In the absence of reliable predictions of future rainfall, runoff, evaporation rates, etc, a water budget model relies on historical climate records to simulate reservoir operations during past conditions with an assumption that future conditions will be similar to past conditions. As such, the model cannot predict conditions on any particular day, but can provide information about the range of conditions that may be encountered.

The NRCS model, Reservoir Operation Study Program TR-19 (RESOP) was developed by the Soil Conservation Service in the 1960s to determine the storage requirements necessary to meet supply-demand relationship (NRCS 1967). In other words, it was developed to produce a water budget model.

NRCS used RESOP to develop the original ELCR water budget model for the 2007 EIS (NRCS 2007). This RESOP model used climatological data from 1951-1992 as the proxy for future conditions. The RESOP model was a monthly water budget model that considered seepage, evaporation, rainfall, runoff, 0.5 CFS in-stream flow, and 7 MGD in water use.

This current water budget model, referred to hereafter as the Water Budget Model (WBM) is intended to update the previous NRCS RESOP model to include the additional 25 years of records now available and to gain a better understanding of daily reservoir fluctuation. An improved understanding of daily reservoir fluctuations is needed to make sure that the reservoir design facilitates reservoir operation during normal, wet and dry periods.

The project team determined that a daily model with additional years of data, would be helpful to:

- Extend the results to consider 25 additional years of information
- Gain a better understanding of the reservoir level fluctuations for optimal design
- Provide additional information for the Supplemental Environmental Impact Statement
- Consider in-stream flow capabilities on a daily basis. (The term “in-stream flow” is intended to be synonymous with the terms “ecological flows” or “environmental flows” which have all been used by various parties to describe the water that is released through the dam to maintain the downstream reach of East Locust Creek.)

Because the RESOP model is based on a monthly time step, it could not provide daily results. Accordingly, a spreadsheet (the WBM) was developed to replicate the RESOP model calculations, but on a daily time step basis. This WBM uses historical data (1950-2017) along with projected watershed, reservoir and water supply conditions to simulate the reservoir water level fluctuation caused by natural, water supply and in-stream flow withdrawals from ELCR.

The WBM is not intended to modify the original 7 MGD estimate of the reservoir firm yield or the size of the reservoir. It does not provide any improved accuracy in estimating firm yield during the design drought (the drought of the 1950s). Rather, it provides additional years of record and details of daily fluctuation to help plan reservoir design and operations.

1.2 Proposed East Locust Creek Reservoir

The proposed East Locust Creek Reservoir will consist of a multipurpose reservoir with a zoned earth dam and a drainage area of approximately 21,000 acres, creating a reservoir surface area of approximately 2,328 acres at normal pool elevation of 922.3 feet, MSL. Watershed land use is described in Table 3. The proposed reservoir spillway is a two-stage labyrinth weir with the first stage at normal pool and the second stage at the 25-year flood level. There is no auxiliary or emergency spillway as the principal spillway is designed to handle all events up to the probable maximum precipitation (PMP) storm. Approximately 10 feet below normal pool will be a passive in-stream flow orifice system that will pass 0.5 cubic feet per second (CFS) to the existing stream below the reservoir (on average). The orifices will pass flow at diminishing rates as the reservoir drops until they cease when the lake falls below the orifice elevations..

2.0 METHODS

The WBM uses historical data (1950-2017) with projected reservoir, in-stream flow and water supply conditions to simulate the reservoir water level fluctuation caused by natural, water supply and in-stream flow withdrawals from ELCR. This approach provides a range of conditions typical for this region to help us to understand reservoir levels but does not predict any particular temporal sequence of events. In other words, it can't be used to predict conditions at any particular time and it can't be used to model a progression of water demand unless we assume an exact repeat of past history. Further, it does not account for climate change.

The inflow to the reservoir consists of direct rainfall on the reservoir and runoff from the East Locust Creek watershed. Outflow from the reservoir consists of seepage, evaporation, outflow through the two-state labyrinth weir, water supply demand, and through the passive in-stream flow system. Based on the inflows and outflows, the WBM calculates the modeled reservoir level for each day.

The model inputs and data sources for the WBM consist of water exchange within the reservoir watershed system, based on factors that add or deduct from the reservoir's water supply. These inputs are listed in Error! Reference source not found. and further described below.

Table 1. Water Budget Model Inputs.

Model Input	Data Source
Rainfall	Historic rain gage data near ELCR. (Green City, Milan (preferentially in that order))
Watershed Runoff	Historic stream gage data. (Locust Creek at Linneus, Medicine Creek Near Galt, South Fork Chariton (preferentially in that order))
Seepage Rate	NRCS (NRCS 2007) and DNR (MDNR 2013)
Evaporation	Historic regional pan evaporation gages as described in Section 2.4.
Water Demand	Varies as described for each scenario.
Reservoir Stage Storage	2008 and 2017 LIDAR Data (Surdex 2009) (Woolpert 2017)
Reservoir Stage Outflow	Hydraulics Report (Appendix B).
In-Stream Flow Releases	Varies as described for each scenario.

2.1 Rainfall

Rain gage data was used to model the quantity of direct rainfall on the reservoir surface. The rainfall that falls on the rest of the watershed is included in the measured watershed runoff, so the rain gage data is not used to calculate watershed runoff.

Rain gages within 11 miles of the reservoir were investigated to determine which gages could provide continuous data for the full period of modeling from gages closest to the reservoir watershed. Of the five evaluated, it was determined that the combination of the two closest gages, Green City 5 N gage (6.6 miles from watershed center) and Mo Milan gage (8.08 miles from watershed center), provided a continuous period of record over the modeled time frame. Data from the Green City 5 N gage was used when available and data from the Mo Milan gage was used to fill in any gaps. The Green City 5 N gage was installed in June of 2006, so the Mo Milan Gage provided the bulk of the data.

2.2 Watershed Runoff – Unit Runoff Method

The Unit Runoff Method (URM) estimates watershed flow into the reservoir based on measured runoff per unit area at nearby stream gages. The URM is based on the measured runoff per unit area at an existing stream gage for each day during the period of record. This runoff per unit area is then applied directly to the watershed area for the proposed reservoir (less the area of the reservoir surface for each day) to determine a volume of watershed runoff into the reservoir. Because the gage data reflects both base flow and storm runoff, the application of this method includes both the base flow and storm runoff merged as watershed runoff. So, no independent estimate of base flow is necessary for the model.

For example, on January 1, 1950, USGS Gage #06901500 reported an average daily flow of 1510 CFS. Dividing this flow into the watershed area (550 mi^2) and making appropriate simple conversions indicates that the flow represented 0.102 inches of runoff averaged over the watershed. This 0.102 watershed inches was applied to the watershed area for the proposed reservoir (29.2 mi^2) to estimate the watershed runoff volume that would have flowed into the reservoir on that day (158.9 ac-ft (or 80.1 CFS)).

To determine the best gage(s) to use for this purpose, stream gages within 50 miles of the watershed centroid with drainage area between 10 and 600 square miles were evaluated to find the most suitable watershed for determining the unit runoff. Consideration was given to similarity of drainage area size, degree of existing impoundment, distance of the gage watershed centroid from the ELCR watershed centroid, and period of record available during the modeled time frame. Of the 21 gages found, 10 were pre-screened out due to insufficient period of record or because they were just downstream of existing major impoundments. Eleven gages were short listed as shown in Figure 1. From this set, two gages stood out due to their proximity to the reservoir and were selected as potential sources of runoff data. However, the combination of these two gages did not provide a complete record of runoff for the period modeled so a third gage was necessary. While the Medicine Creek near Laredo Gage would have been the most appropriate third gage due to location and shape, its data gaps coincided with the gaps in the two primary gage records. Details of the 11 short listed gages are presented in Table 2.

Figure 1. Surrogate Gage Selection for Watershed Runoff

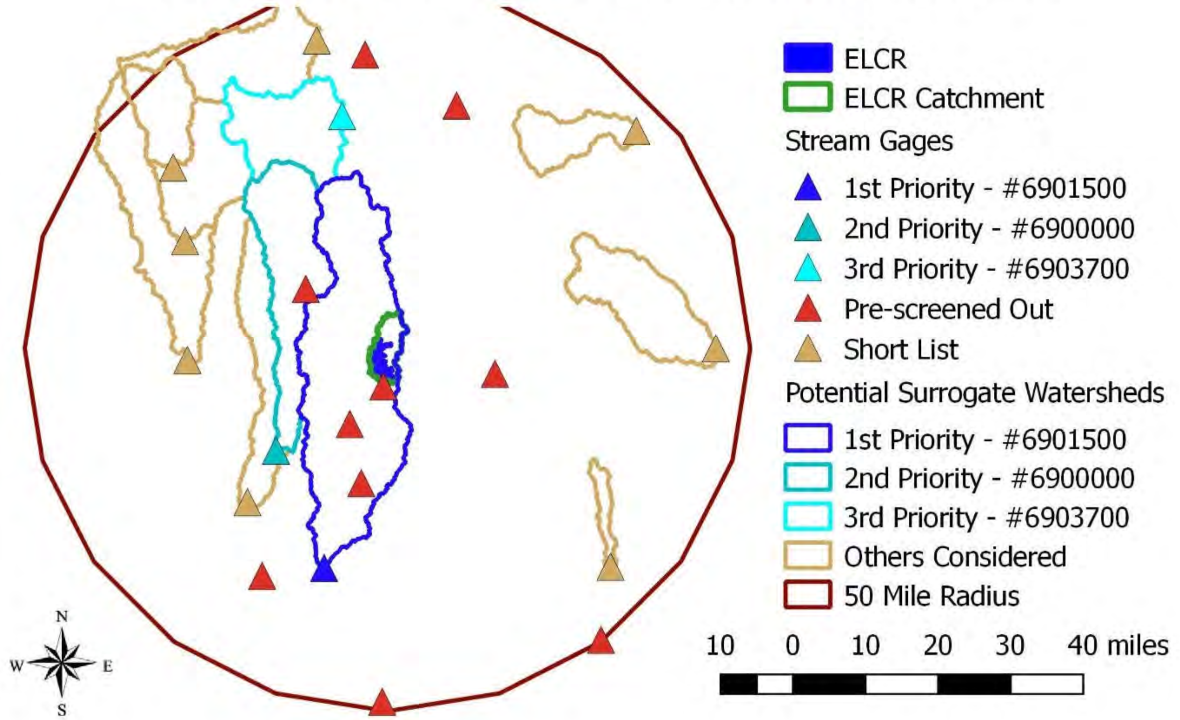


Table 2. Short Listed Stream Gages				
Gage	Drainage Area (mi^2)	Distance (Watershed Centroid to ELCR Catchment Centroid) (mi)	Relevant Period of Record (years)	Priority
6901500 Locust Creek near Linneus, MO	550	5.8	68	1
6900000 Medicine Creek near Galt, MO	225	16.9	40	2
6903700 South Fork Chariton River near Promise City, IA	168	33.3	52	3
6900050 Medicine Creek near Laredo, MO	355	16.0	17	NA

Table 2. Short Listed Stream Gages				
Gage	Drainage Area (mi^2)	Distance (Watershed Centroid to ELCR Catchment Centroid) (mi)	Relevant Period of Record (years)	Priority
5497500 Middle Fabius River near Baring, MO	185	35.6	11	NA
6906150 Long Branch Creek near Atlanta, MO	23	37.2	22	NA
6899000 Weldon River at Mill Grove, MO	494	37.4	23	NA
5494300 Fox River at Bloomfield, IA	88	38.3	65	NA
6898500 Weldon River near Mercer, MO	246	39.1	28	NA
6903400 Chariton River near Chariton, IA	182	44.8	68	NA
6898400 Weldon River near Leon, IA	104	45.2	33	NA

The two primary gages are Locust Creek near Linneus (#06901500) and Medicine Creek near Galt, MO (#06900000) and the third gage on the South Fork of Chariton River near Promise City, IA (#06903700). Because the Locust Creek gage watershed includes the ELCR catchment, it is selected as the unit runoff source whenever it has valid data. Likewise, the Medicine Creek near Galt data is used when Locust Creek gage doesn't have data and the S. Fork Chariton data fills in the remaining data gaps. Figure 2. graphically represents the availability of data from these gages. Table 3 compares the land use in the respective watersheds to the land use in the ELCR watershed.

Figure 2. Runoff Records from Three Gages Used in Unit Runoff Analysis

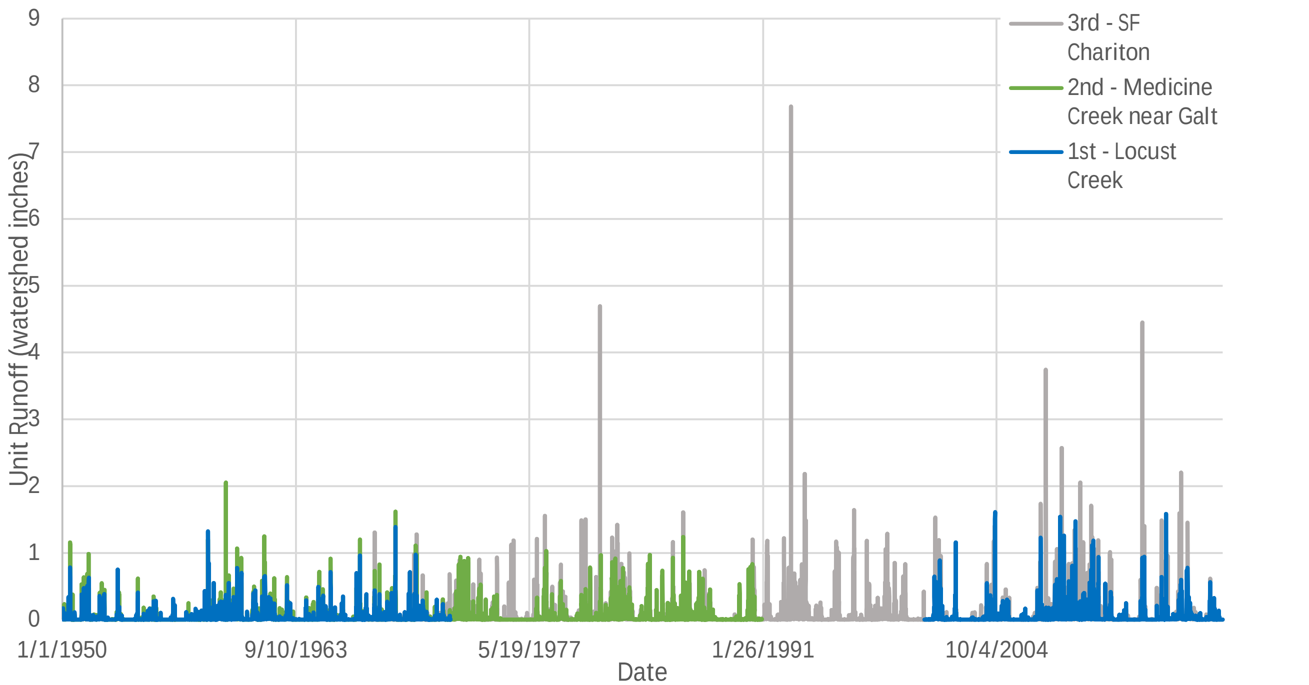


Table 3. Watershed Land Use (2016 NLCD).

NLCD Class	Land Use	Linneus #6901500	Medicine Creek Near Galt #6900000	S. Fork Chariton near Promise City #6903700	ELCR Catchment (above pool)
		Land Use Percentage (%)			
11	Open Water	0.7%	0.4%	0.7%	0.8%
21	Developed, Open Space	2.8%	2.6%	2.8%	3.5%
22	Developed, Low Intensity	0.7%	0.9%	1.4%	0.6%
23	Developed, Medium Intensity	0.1%	0.1%	0.1%	0.0%
24	Developed, High Intensity	0.0%	0.0%	0.0%	0.0%
31	Barren Land (Rock/Sand/Clay)	0.0%	0.0%	0.0%	0.0%
41	Deciduous Forest	19.7%	16.1%	11.0%	21.3%
42	Evergreen Forest	0.3%	0.3%	0.1%	0.5%
43	Mixed Forest	5.5%	3.4%	0.8%	7.1%
52	Shrub/Scrub	0.3%	0.2%	0.1%	0.2%
71	Grassland/Herbaceous	0.6%	0.5%	0.3%	0.5%
81	Pasture/Hay	51.6%	50.0%	39.6%	60.3%
82	Cultivated Crops	15.6%	23.2%	41.6%	5.0%
90	Woody Wetlands	1.9%	1.9%	1.2%	0.1%
95	Emergent Herbaceous Wetlands	0.2%	0.2%	0.4%	0.0%
	Total	100.0%	100.0%	100.0%	100.0%

2.3 Seepage Rate

The seepage from the reservoir varies with the surface area of the lake. Table 4 lists the lake surface area and the seepage rate for the lake. The seepage rate table for ELCR was provided by NRCS and is consistent with seepage values used for Elmwood Reservoir and Lake Thunderhead in the 2013 MDNR Water Supply Study (MDNR 2013).

Table 4. Seepage Values

Lake Surface Area (acres)	Seepage (inches/day)	Seepage (inches/month)
2100 and above	0.100	3.0
1100 to 2100	0.067	2.0
1100 and below	0.033	1.0

2.4 Evaporation

The evaporation from the reservoir was simulated using the pan evaporation data from gages throughout the region and the daily calculated reservoir surface area. The evaporation gages near ELCR are listed below in Table 5:

Table 5. Evaporation Gage Stations.

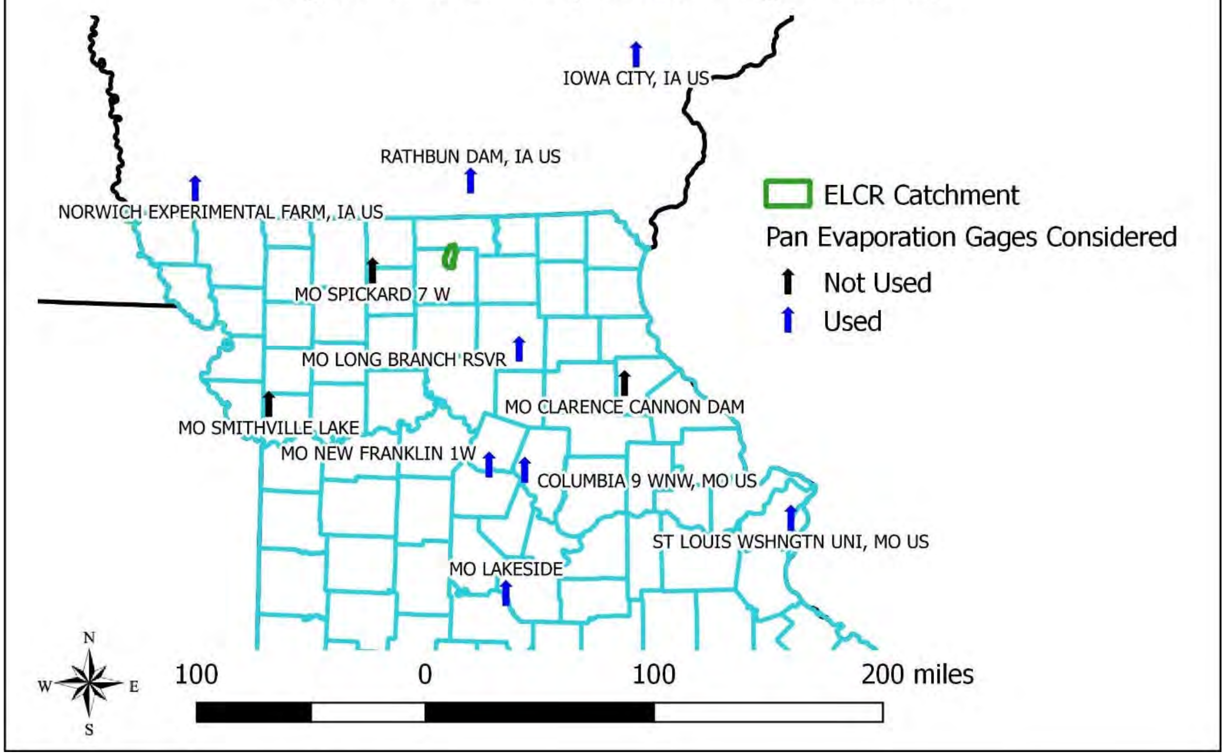
Station	Distance from ELCR Catchment Centroid	Period of Record	Comments
Rathbun Dam, IA US (USC00136910)	34.5	1970-Present	
Mo Spickard 7 W (USC00237963)	34.5	1957-1993	Not used – data quality concerns.
Mo Long Branch Rsvr. (USC00235050)	50.2	2011-Present	
Mo New Franklin 1W (USC00236012)	92.4	1956-2016	
Mo Clarence Cannon Dam (USC00231600)	94.1	1996-1997	Not Used– insufficient data
Columbia 9 WNW, MO US	98.6	1944-1953	
Norwich Experimental Farm, IA US	115	1937-1970	
Smithville Lake (USC00237862)	102	1985-Present	Not used – data quality concerns.
Ames 3 SW, IA US	118	1893-1964	
Iowa City, IA US	120	1950-Present	
Mo Lakeside (Lake of the Ozarks) (USC00234694)	149	1931-1990	
St. Louis Washington University, MO US	187	1938-1957	

Because the records were often missing within the reported period of record, it was necessary to find multiple stations to ensure some data was available. The model used the average of all gages with data for each individual day.

There were some dates for which none of these stations had data. In that case, the average pan evaporation value for that day of the year was used. The average pan evaporation for the day of the year was calculated by averaging all values at all stations in every year there was data for that day.

The pan evaporation coefficient of 0.76 was used to convert the pan evaporation values from the National Oceanic and Atmospheric Administration (NOAA) gages to the free water surface of the reservoir (NOAA 1982). The pan coefficient of 0.76 was selected based on DNR's WBM of Elmwood Reservoir (4 miles from ELCR) (DNR 2013). Figure 3 shows the locations of the pan evaporation gages considered and used.

Figure 3. Pan Evaporation Gage Selection

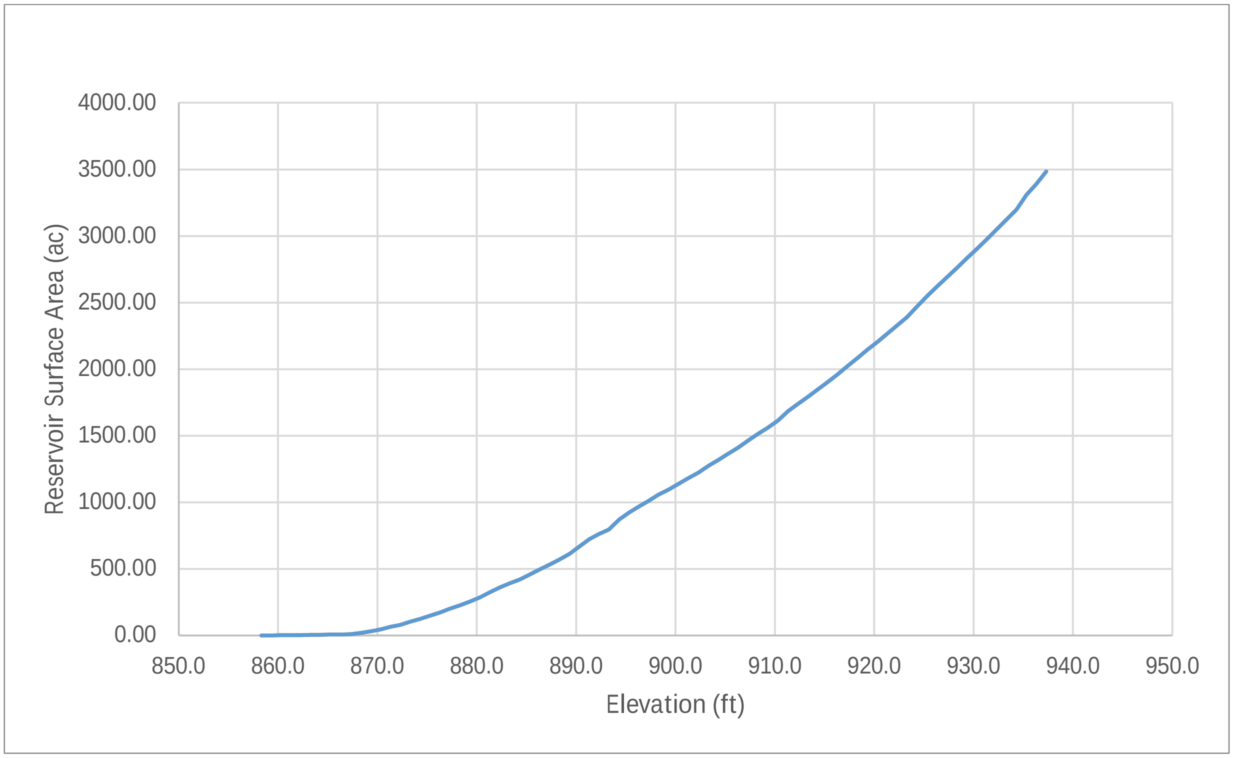


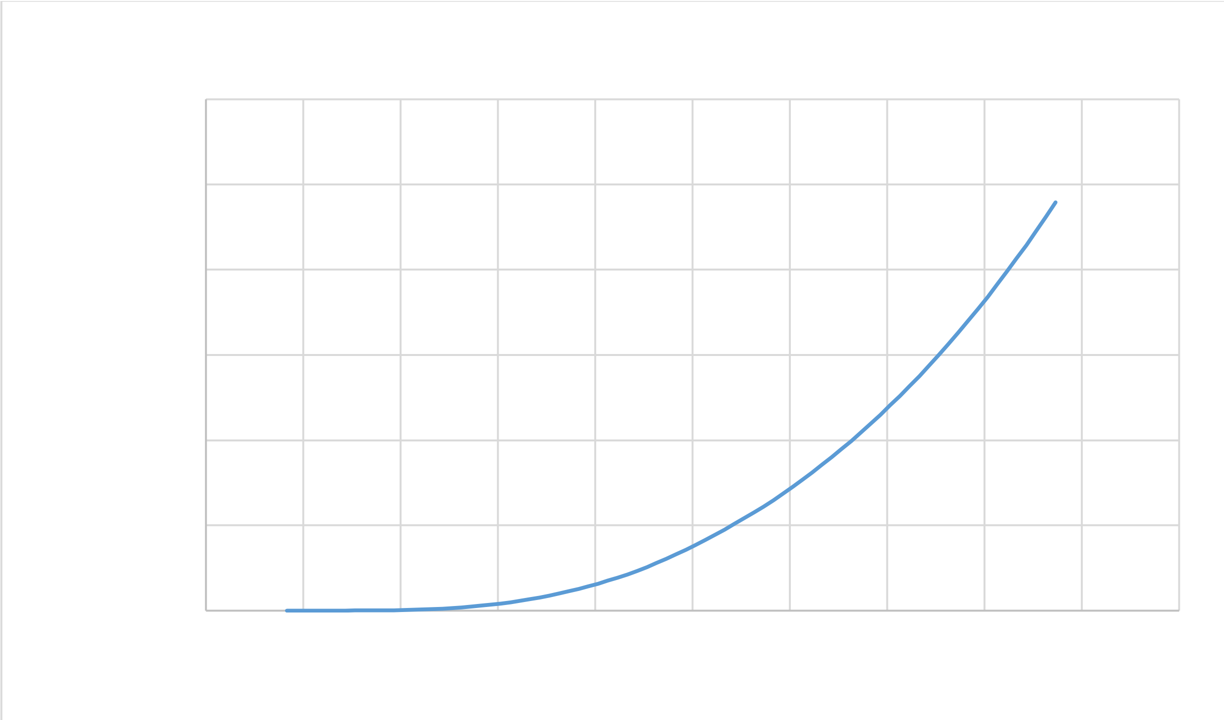
2.5 Water Demand

The reservoir is designed to be able to supply 7 MGD during the drought of record (NRCS 2003). The model was setup to be able to run various demand scenarios up to 7MGD. For basic sizing of the reservoir a constant demand of 7MGD is used. Appendix A contains a summary of results for other demand rates for use in understanding reservoir fluctuation and ecological flows during periods prior to when the full 7 MGD is being sold.

2.6 Reservoir Stage Storage

The reservoir stage storage data was created from LIDAR taken in 2008 and 2017. The 2017 LIDAR only includes the floodplains but is used to the extent possible with the 2008 data being used for higher elevations not covered by the 2017 data. Figure 4 displays the relationship of reservoir surface area to elevation and Figure 5 displays the relationship of total reservoir volume to elevation.





2.7 Reservoir Stage Outflow

The proposed reservoir spillway is a two-stage labyrinth weir with the first stage at normal pool and the second stage at the 25-year flood level. There is no auxiliary or emergency spillway as the principal spillway is designed to handle all events up to the probable maximum precipitation (PMP) storm. The reservoir stage outflow for the two stage spillway is described fully in appendix B. The stage discharge data from that report is compiled in Table 6.

Table 6. Stage Outflow Data

Elevation (ft)	Flow (CFS)	Description
922.3	0	1st Stage Weir
922.55	19	
922.8	59	
923.05	112	
923.3	179	
923.55	256	
923.8	342	
924.05	436	
924.3	536	
924.55	642	
924.8	753	2nd Stage Weir
924.9	762	

Table 6. Stage Outflow Data		
Elevation (ft)	Flow (CFS)	Description
925	780	
926.4	1501	
927.8	2730	
929.2	4152	
930.5	5491	
931.9	6887	
933.3	8235	
934.6	9469	
936	10817	2' Below Top of Dam

2.8 In-Stream Flow Releases

To maintain the stream below the dam, the reservoir will be configured to provide in-stream flow when the principal spillway is not active. This in-stream flow will be in addition to what passes through the spillway due to storm events and what seeps through the dam. The original Environmental Impact Study (NRCS 2006) assumed an average value of 0.5 CFS for in-stream flows. As discussed in Section 3.2, a passive system has been devised to produce at least an average of 0.5 CFS over the period of record given projected lake elevation and the results below reflect the presence of that system unless stated otherwise.

2.8.1 Existing Stream Flow

To provide a description of existing flows in East Locust Creek for comparison to post project flows, a USGS stream gage, #06901205, was constructed on East Locust Creek and began operation on September 30, 2013. The gage is about a mile downstream of the proposed dam and has a drainage area of 33.8 mi^2. The proposed reservoir has a drainage area of 32.5 mi^2. Figure 6 shows the mean daily discharge over the period of record.

The following flow duration curves were created from the gage data to illustrate the flow characteristics of East Locust Creek. Figure 7 shows the flow duration curve for the entire period of record and Figure 8 shows the flow duration curve separated by calendar year. Data over the period from 7/1/2014-6/30/2019 indicates that the gage records zero flow approximately 6% of days and less than 0.1 CFS approximately 15% of days. The median daily average discharge during this time period was 2.86 CFS.

Breaking down the flow data by year (Figure 8) reveals that in the 2018 the creek did not have any flow for 25% of the year and that in 2015 the creek had year round flow. Three out of five years had some periods with zero flow.

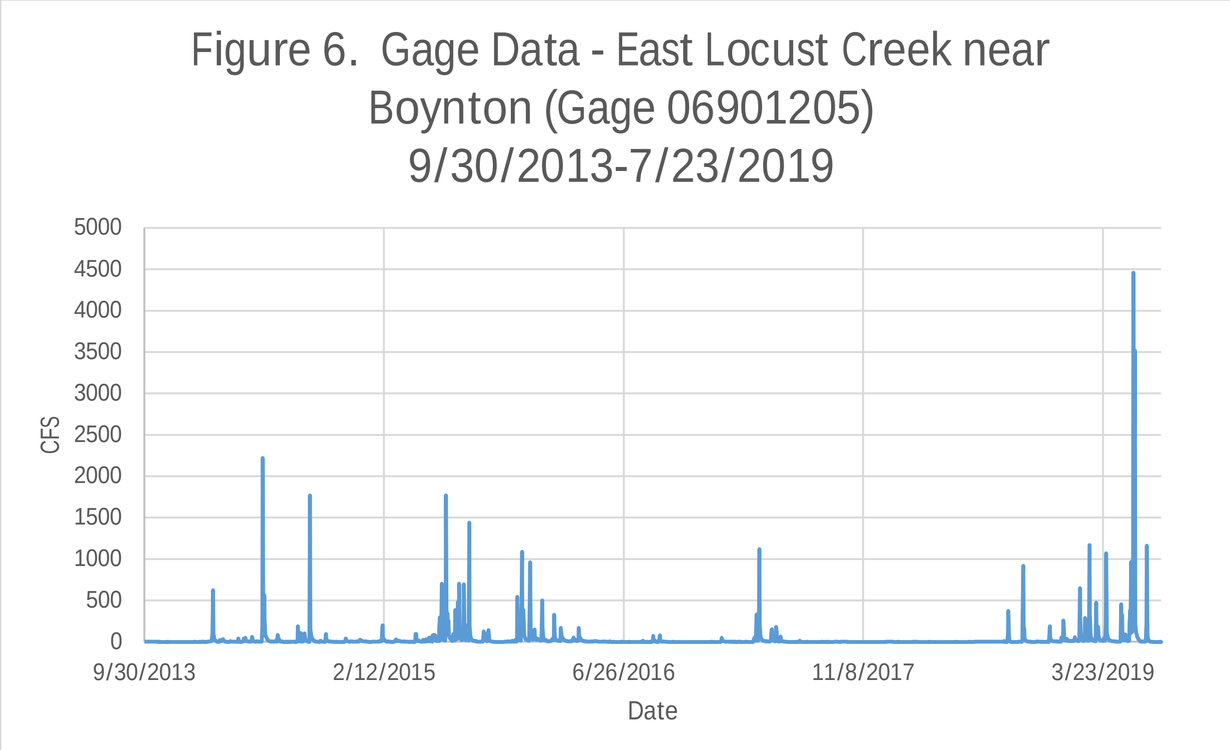
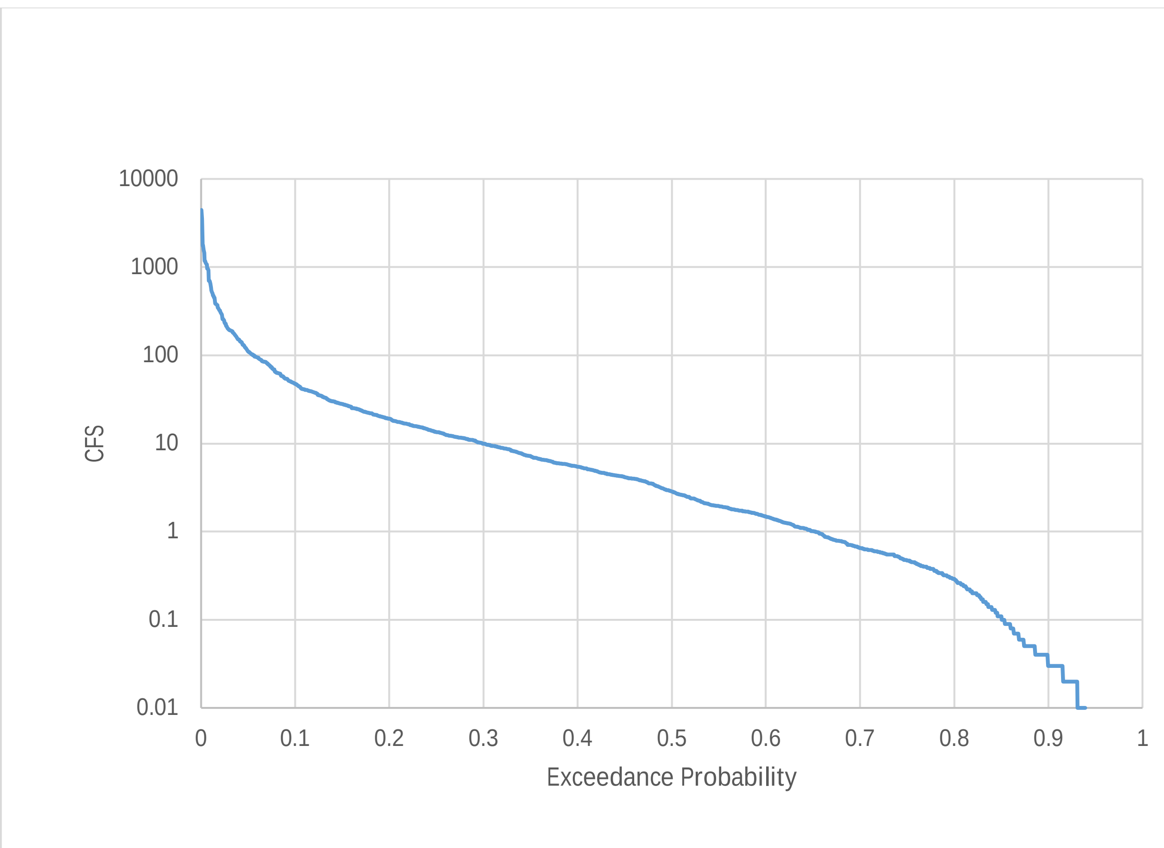
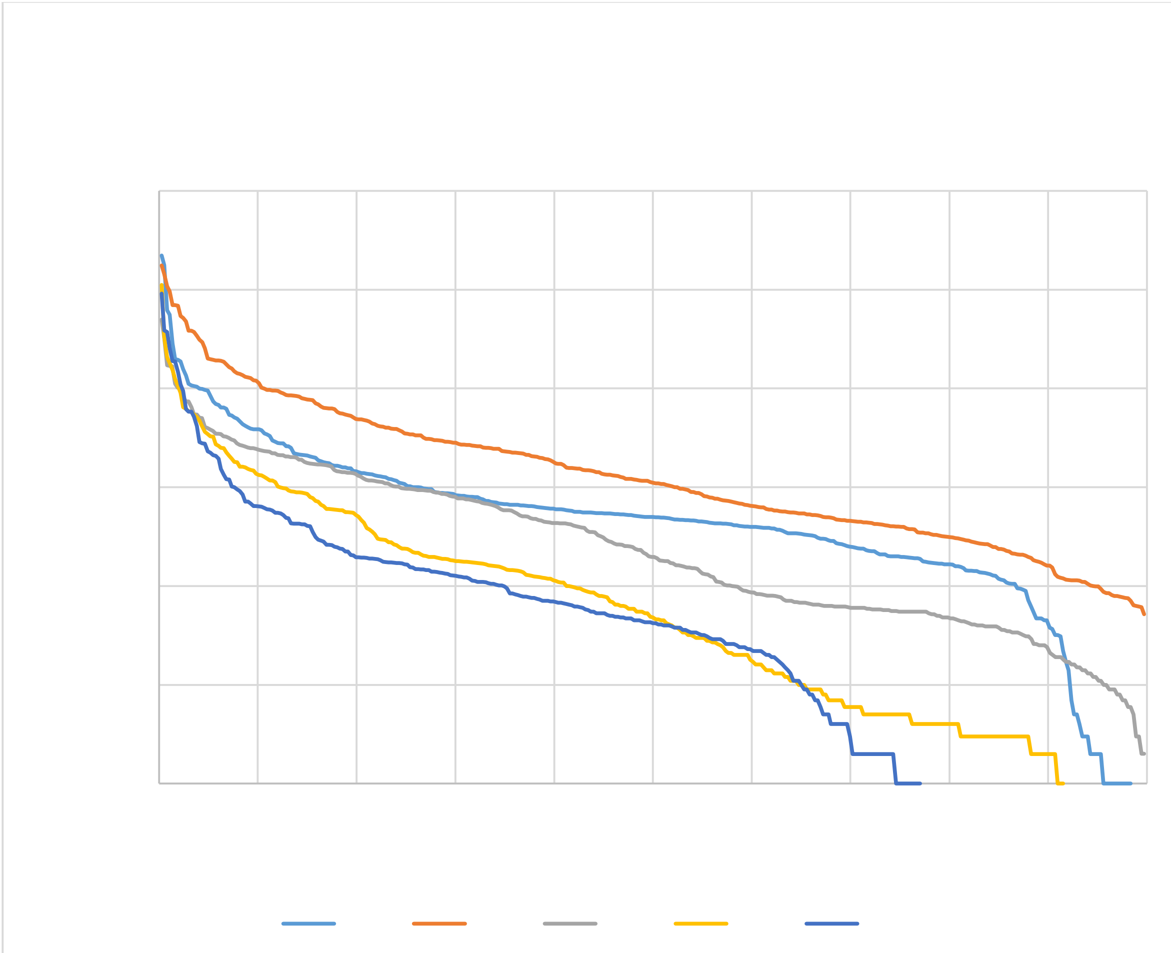


Figure 6. Gage Data - East Locust Creek near Boynton (Gage 06901205) 9/30/2013-7/23/2019
CFS
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
9/30/2013
2/12/2015
6/26/2016
11/8/2017
3/23/2019
Date


CFS
10000
1000
100
10
1
0.1
0.01
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Exceedance Probability



3.0 RESULTS

The WBM was applied with varying assumptions and constraints to develop a better understanding of how the reservoir might operate, based on historical climate and conditions. Particularly the model was used to:

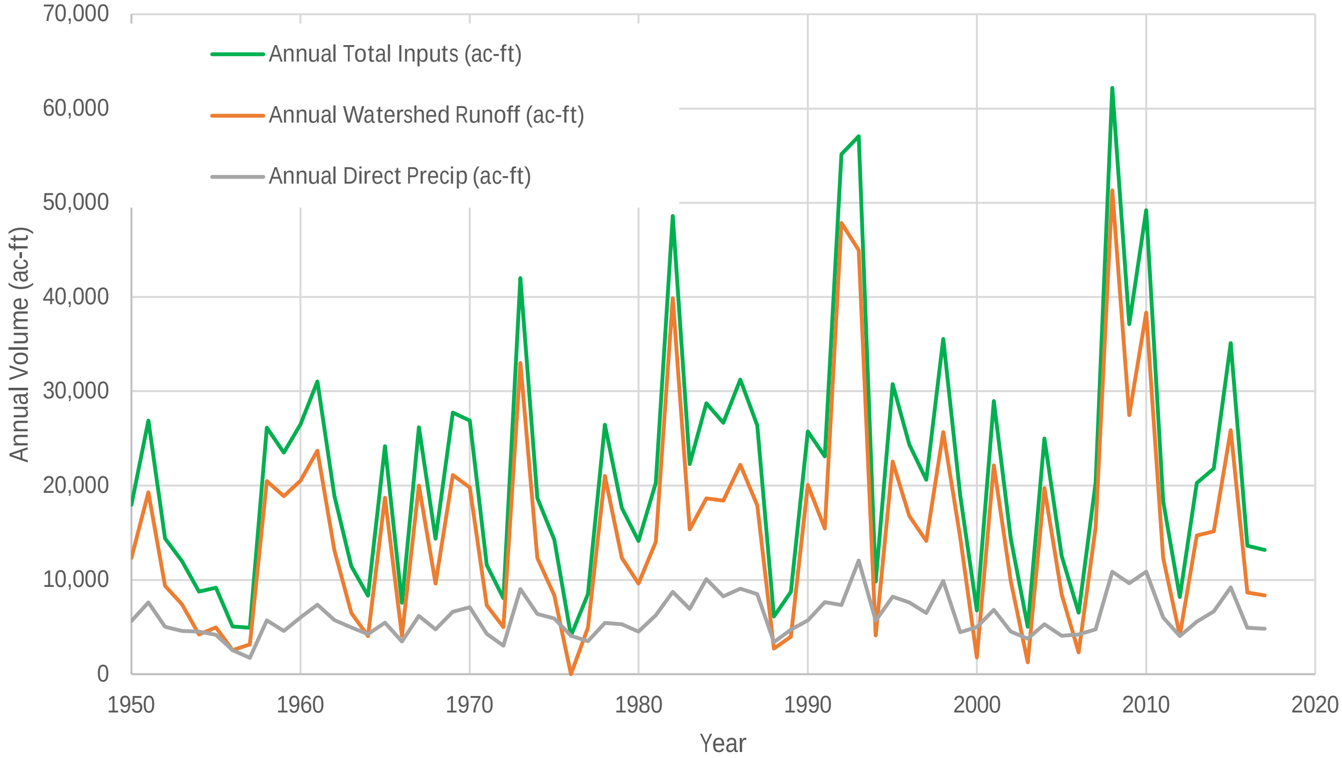
- Simulate flow conditions in East Locust Creek below the dam
- Estimate the time required to fill the reservoir once dam construction is complete
- Model how lake levels will tend to fluctuate after the reservoir is initially filled

3.1 General results

Figures 9-11 display the annual average values of the model inputs and outputs. As expected, the watershed runoff is the primary source of water into the reservoir, but direct rainfall on the reservoir is also a significant source. The 7 MGD water supply demand is the largest steady outflow of water from the reservoir, although there are years when evaporation exceeds demand. Seepage is also a significant source of loss, especially in years when the reservoir is full. Spillway release is highly variable, reflecting the fact that the reservoir will often be below normal pool and many storms will add significant volumes of water to the reservoir without raising it to the level that allows spillway flow.

The average annual reservoir volume has been added to Figure 11 to provide a sense of scale for the total inputs and outputs from the reservoir. As can be expected, the inputs tend to be more variable while the outputs are dampened by reservoir storage.

Figure 9. Annual Total Volumes of Reservoir Inputs



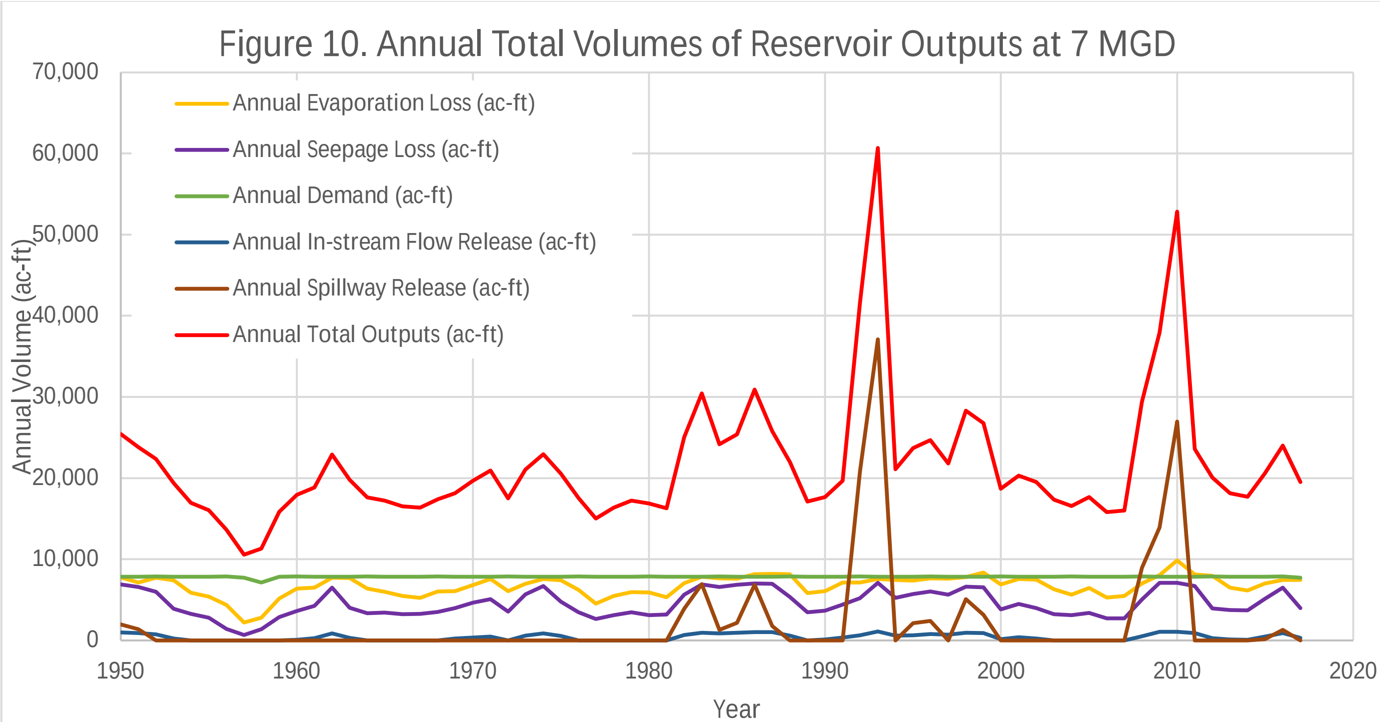


Figure 10. Annual Total Volumes of Reservoir Outputs at 7 MGD
Annual Evaporation Loss (ac-ft)
Annual Seepage Loss (ac-ft)
Annual Demand (ac-ft)
Annual In-stream Flow Release (ac-ft)
Annual Spillway Release (ac-ft)
Annual Total Outputs (ac-ft)
Annual Volume (ac-ft)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
1950
1960
1970
1980
1990
2000
2010
2020
Year

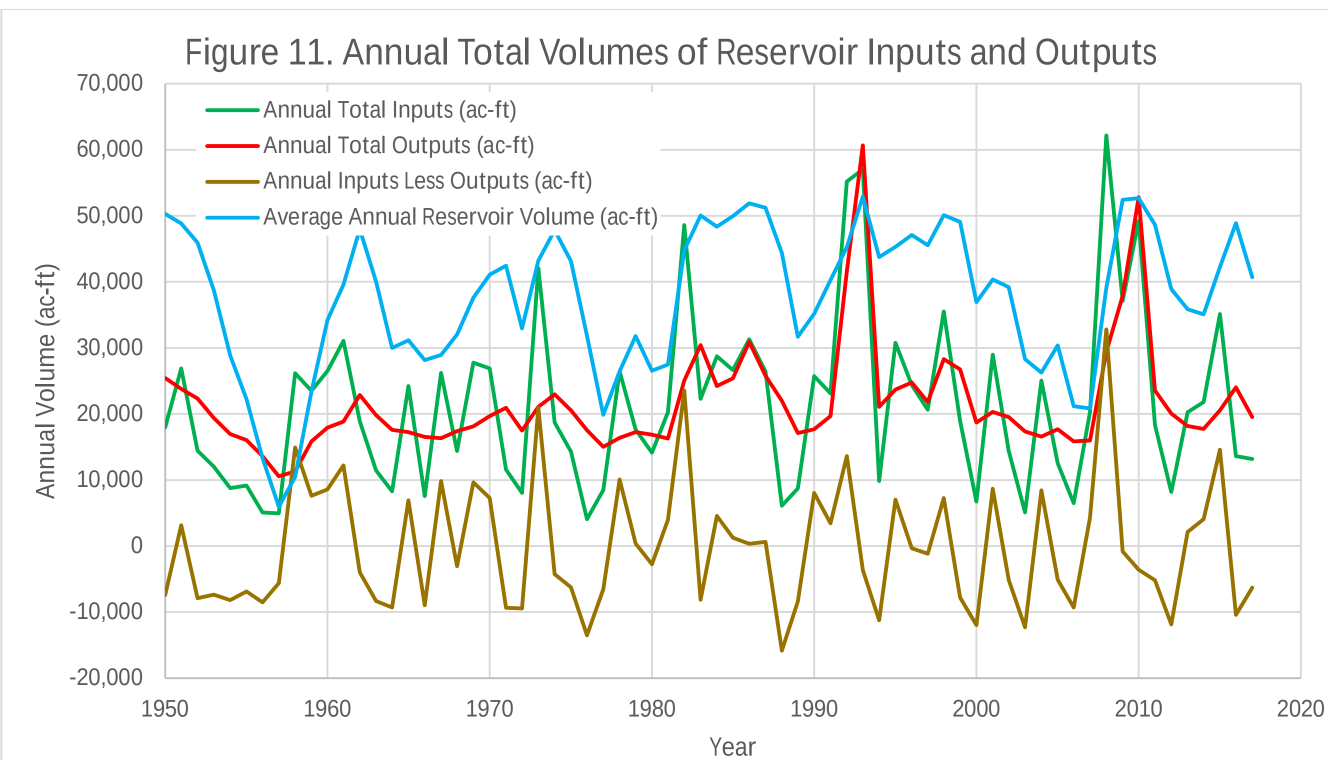
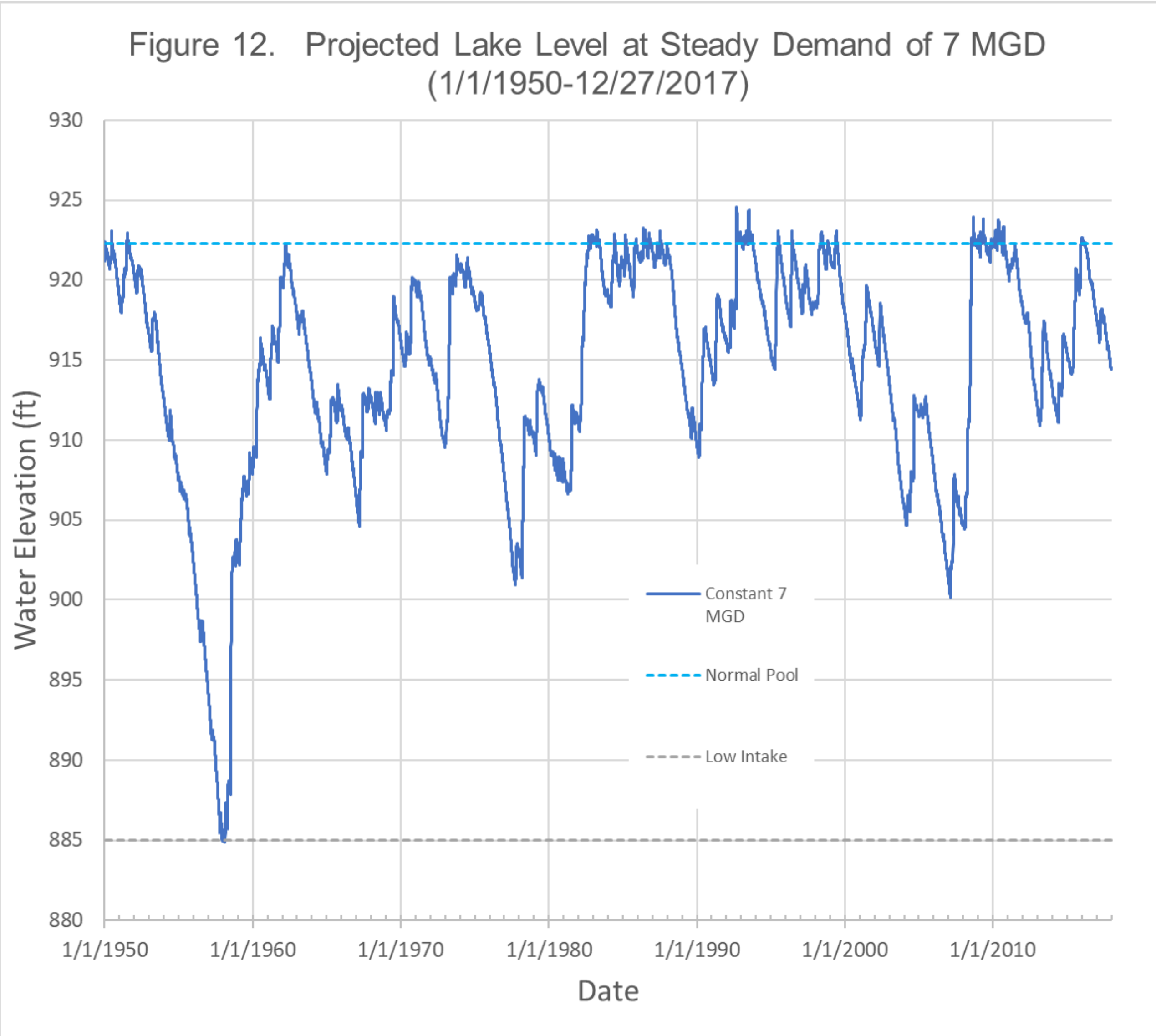


Figure 11. Annual Total Volumes of Reservoir Inputs and Outputs
Annual Total Inputs (ac-ft)
Annual Total Outputs (ac-ft)
Annual Inputs Less Outputs (ac-ft)
Average Annual Reservoir Volume (ac-ft)
Annual Volume (ac-ft)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
-10,000
-20,000
1950
1960
1970
1980
1990
2000
2010
2020
Year

Figure 12 shows the estimated reservoir level over the period of record for the full 7 MGD demand, starting with the reservoir full, and utilizing the proposed in-stream flow configuration. It shows that during the drought of record the lake level drops to the low intake level, confirming that the firm yield of the reservoir is 7 MGD given this configuration. Appendix A and Figure 16 below provide similar curves for other demand rates less than 7 MGD for use in estimating reservoir operations prior to when the full demand is developed.



3.2 In-stream flow Implementation

This study evaluated various options for passive orifices placed to provide an average in-stream value of 0.5 CFS over the period of record.

The reservoir principal/auxiliary/emergency spillway is controlled by a labyrinth weir placed in the spillway (Appendix B). The weir wall is notched to 10' tall for the principal spillway and is 12.5' tall for the auxiliary/emergency spillway. Normal pool level is at elevation 922.3 and from normal pool to the 25-year level the principle spillway notch handles all storm flow. At the 25-year flood elevation (924.8') the auxiliary/emergency portion of the weir wall becomes active.

The proposed plan is to place orifices in the weir wall to provide in-stream flows. The orifices will be fitted with flanges so that orifice sizes can be modified in the future using an adaptive management approach. A configuration that is estimated by the model to provide 0.55 CFS of in-stream flow on average over the period of record at the full 7MGD demand is described in Table 7. In addition, there will be some seepage through the dam and abutments that will also contribute to in-stream flow. When the reservoir level drops below the elevation of 913' (9.3' below normal pool) the orifices will not allow any flow to pass, but seepage through the dam and abutments will continue. The low orifice elevation is set 0.7' above the base of the weir wall to allow space for the flange.

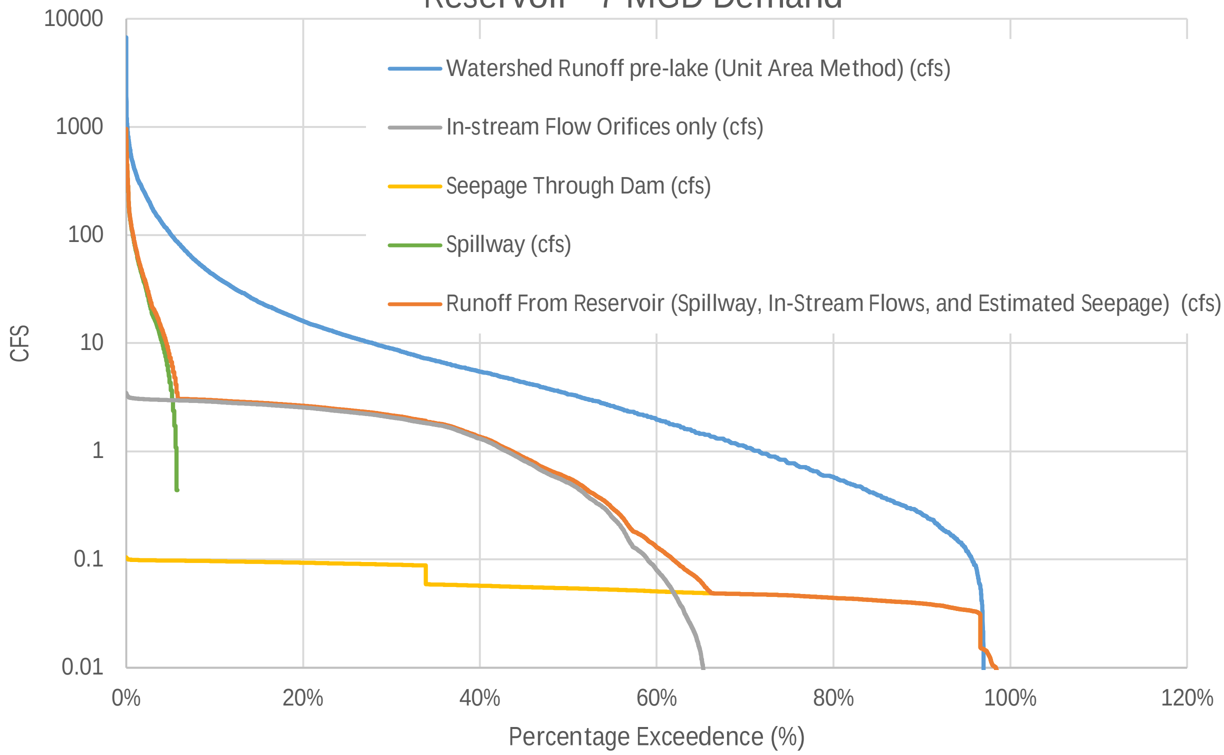
To provide some basis for comparison to existing conditions, the unit runoff method was applied to the entire East Locust Creek Reservoir watershed using the unit area flow from the stream gage system described above to develop the pre-lake watershed runoff curve shown in Figure 13. Based on the daily reservoir elevation calculated by the model over the 1950-2017 time frame with the full 7 MGD demand, the orifice equation was used to calculate daily flow through the in-stream flow orifice system and the spillway rating curve was used to calculate daily spillway flow. The seepage through the dam was estimated at 1% of the total seepage from the reservoir described in Section 2.3. Figure 13 also shows the resulting flow duration curve for East Locust Creek and the contributions from the proposed reservoir for the 1950-2017 time frame. Appendix A provides similar curves for other demand rates less than 7 MGD for use in estimating reservoir operations and stream flows prior to when the full demand is used.

The flow duration curves show that at most times there will be less water in the stream after the reservoir is built. This is not a surprising result because the reservoir is being built for water supply purposes and a significant amount of water will go to water supply. However, during the drier times, it will not be uncommon for the reservoir to pass some water downstream when there wouldn't have been water without the dam. The curves also show that the reservoir will reduce the volume of flooding that is passed downstream during large storm events.

Table 7. In-Stream Flow Orifice Details

Reservoir Elevation (ft)	Feature	Flow through Orifice System at Reservoir Elevation (CFS)
924.8’	Auxiliary/Emergency Spillway	1.77
922.3’	Principal Spillway	1.49
917’	2 – 2.5” diameter orifices	0.52
915’	2 – 2” diameter orifices	0.15
913’	1 – 1.5” diameter orifices	0
912.3’	Bottom of weir wall, spillway floor	0

Figure 13. Flow Duration Curves - Estimated Pre and Post Reservoir - 7 MGD Demand



The impacts on the stream below the dam will be most noticeable near the dam and will recede further downstream as the reservoir watershed becomes a smaller fraction of the overall watershed. At 5.39 miles downstream from the dam the Elmwood Branch brings the drainage area up to 49.3 square miles, but it also adds the relatively reliable 1.5 CFS flow of treated process water from the Smithfield pork processing plant. According to the 2019 MDNR Stream Classification data set, East Locust Creek becomes perennial just downstream of this point.

At 20.9 miles downstream is the confluence with Little East Locust Creek and the total drainage area is 121 square miles and the reservoir drainage area is 27% of the watershed. At 22.8 miles downstream from the dam East Locust Creek pours into Locust Creek and the total drainage area below the confluence is 380 square miles and the reservoir drainage area is 8.5% of the total watershed. By the time Locust Creek reaches Pershing State Park, the first downstream recreational access to a stream, the reservoir is only 6% of the total watershed.

3.3 Reservoir Startup Scenarios

Several model scenarios were run to evaluate the range of possibilities for time to fill the reservoir based on the precipitation conditions throughout the period of record. These scenarios were run by setting the reservoir volume to zero at various scenario start dates to see how long it takes to recover to full pool under the conditions at that time. To capture the likely range of water demand during the filling period, startup scenarios were run at both 0 MGD and 3 MGD demand. Given the time it will take to expand the water supply system to reach the additional customers, it is unlikely that the water demand will exceed 3 MGD during the filling period. Table 8 describes the scenarios and lists the time required to fill the reservoir under the evaluated scenarios. The average monthly Palmer Drought Severity Index (PDSI) for north eastern Missouri is also included for the time of filling to provide a quantitative description of conditions at the time. To simulate in-stream flows during the reservoir startup scenarios it was assumed that steps would be taken to produce an average of 0.5 CFS of in-stream flow per day during filling, even when the reservoir level is below the passive in-stream flow orifices. The in-stream flow release during reservoir filling is anticipated as a step taken to avoid an initial possible multiyear dry period for the stream. Figure 14 shows the reservoir level over time for these various scenarios. Figure 15 compares the filling time to the average PDSI during the filling time.

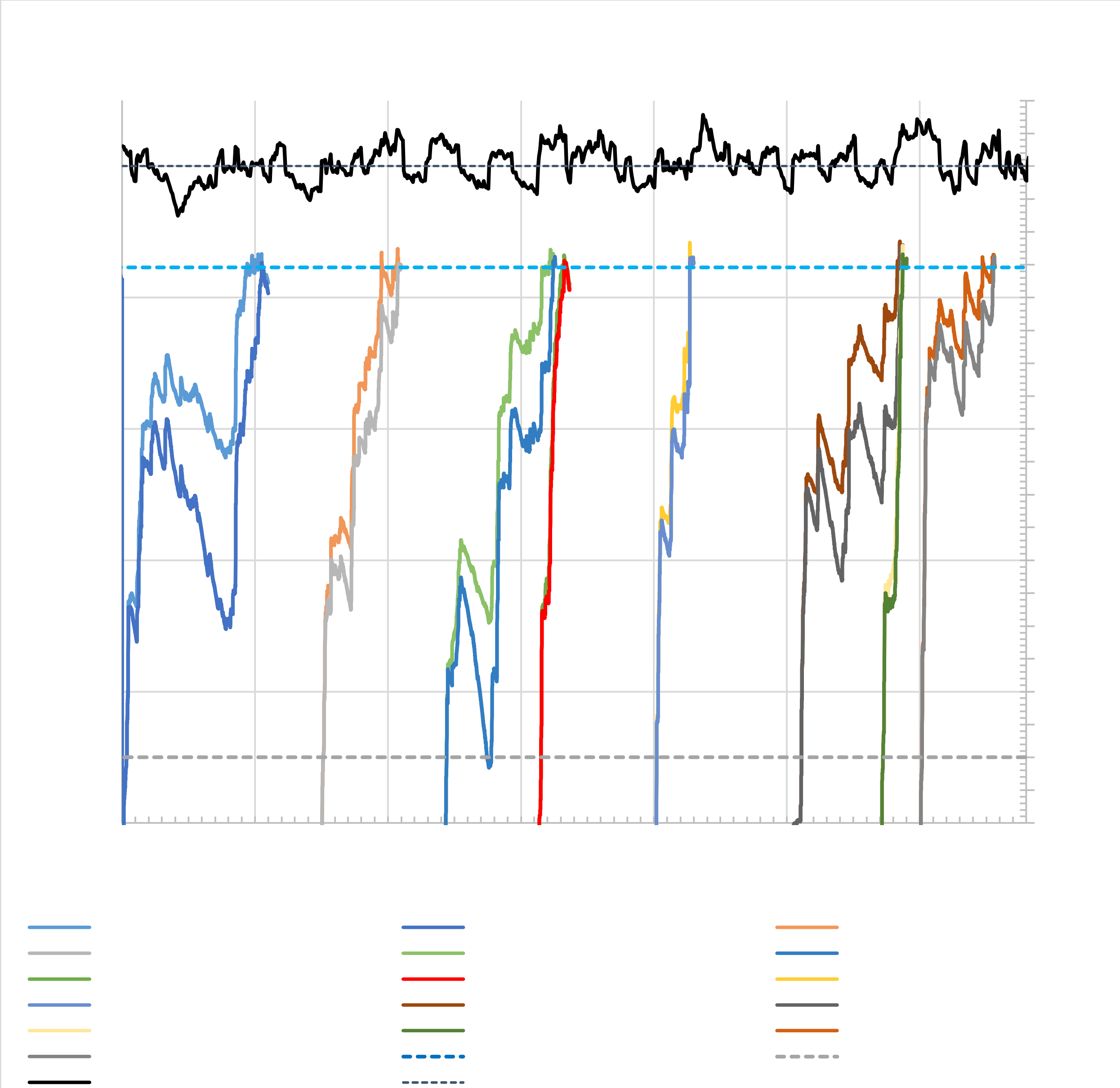
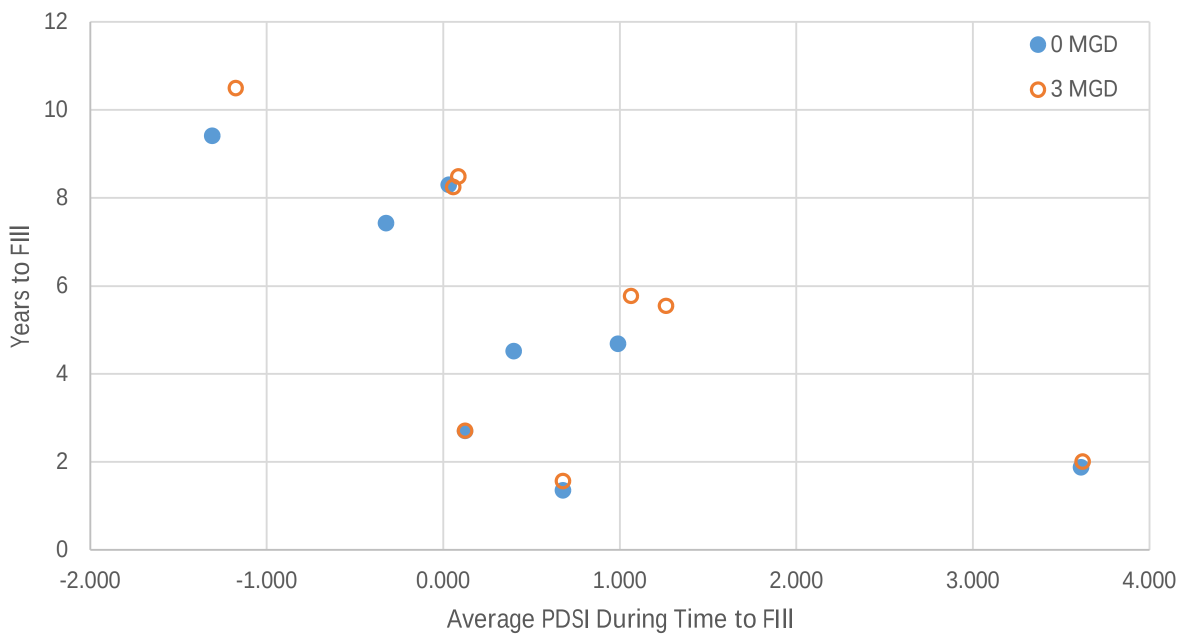


Table 8. Time Required to Fill Reservoir under various scenarios				
Start Date	Scenario Description	Average PDSI* during filling period @0 MGD (and @3MGD)	Years To Fill	
			@ 0 MGD Demand	@ 3 MGD Demand
1/1/1950	Drought of Record	-1.310 (-1.175)	9.41	10.50

Table 8. Time Required to Fill Reservoir under various scenarios				
Start Date		Average PDSI*	Years To Fill	
			@ 0 MGD Demand	@ 3 MGD Demand
1/1//1965		0.400 (1.063)	4.52	5.77
3/1/1974		-0.325 (0.057)	7.43	8.25
4/1/1981	Wettest Period in 1900s	3.613 (3.622)	1.88	2.01
1/1/1990		0.123 (0.123)	2.70	2.71
1/1/2000	Drought of early 2000s	0.031 (0.085)	8.30	8.49
2/19/2007	Recovery from dry years	0.678 (0.678)	1.36	1.57
1/1/2010		0.989 (1.261)	4.69	5.55

* Palmer Drought Severity Index

Figure 15. Estimated Time to Fill vs. Average Palmer Drought Severity Index (PDSI)



3.4 Reservoir Fluctuation

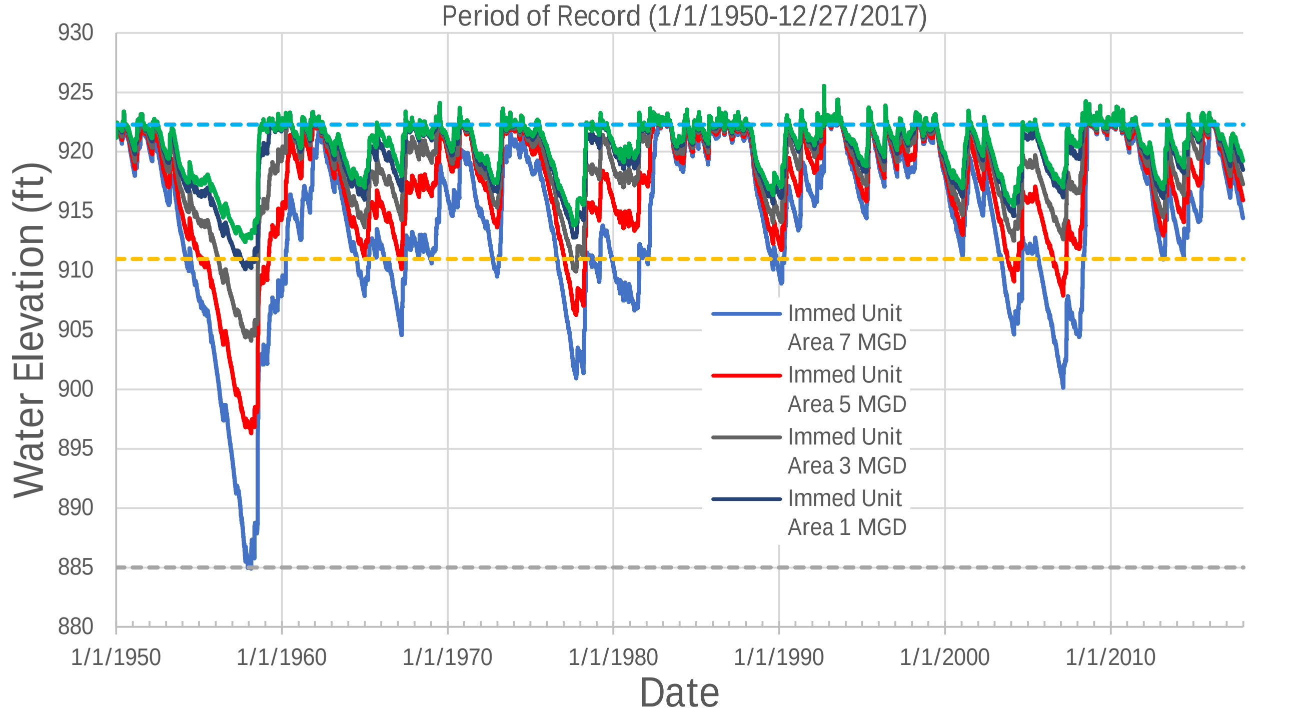
3.4.1 Operational Levels

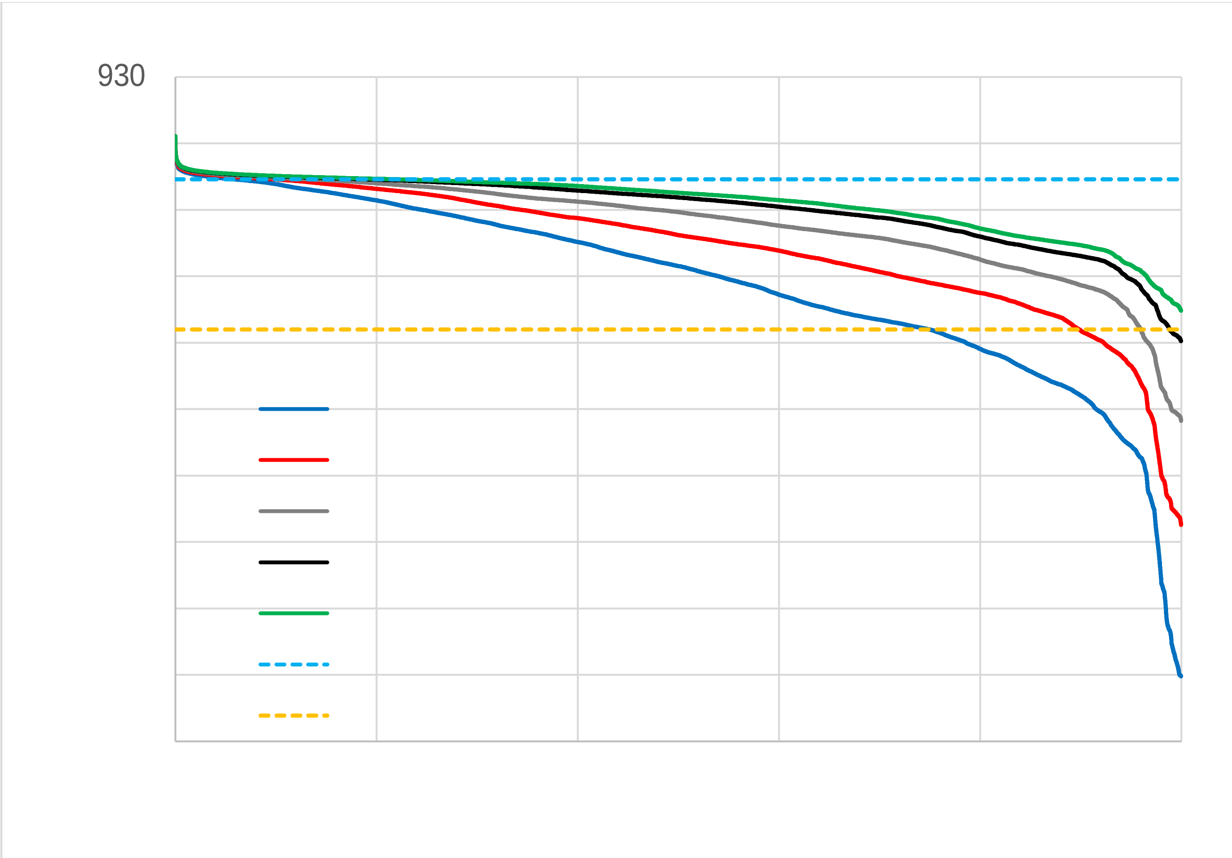
Operational level simulations were run by setting the reservoir level to normal pool at the beginning of 1950 and running the model for the period of record. In-stream flows were included in these simulations based on the orifice system described above and all other inputs and outputs were as described in Section 2. However, the water supply demand was varied from 0 to 7 MGD to provide insight into how the reservoir levels might fluctuate at any given demand because the full 7 MGD demand could take anywhere from 10-30 years to develop. Consideration was given to modeling a ramp up in demand, but such modeling would only be valid if climactic conditions repeated the exact temporal patterns in the period of record.

Figure 16 provides the model results over the period of record for the range of demands from 0 MGD to 7 MGD. Figure 17 shows the results in the form of a reservoir level frequency diagram.

A main goal of completing this analysis is to set an operational level to which the reservoir facilities will be designed. Given the model results, it seems appropriate to design the reservoir facilities to operate normally for 75% of the time when the reservoir is providing 7 MGD. This equates to a reservoir level of 911’. At this elevation, the reservoir surface area would be 1,710 acres. Below this level, recreational activities on the lake can still be allowed but with restrictions and cautions.

Figure 16. Proposed East Locust Creek Reservoir - Estimated Reservoir Levels over Period of Record (1/1/1950-12/27/2017)





Model predicted maximum rates of reservoir level drop were evaluated to determine whether there is cause for concern about slope failures for embankments or shoreline around the reservoir. Rates were calculated by subtracting each day from the previous day to find the maximum reservoir level drop over a single day. The rate of drop was also averaged over 7, 14, 30, 90, and 180 days and over 1, 3 and 5 years to find the highest extended period of reservoir level drop. The maximum drop rates are provided in Table 9. Not surprisingly, the shorter duration drops all start when the reservoir is above normal pool (922.3').

Table 9. Maximum WBM Reservoir Level Drop Rates

Time Period	1 Day	7 Days	14 Days	30 Days	90 Days	180 Days	1 Year	3 Years	5 Years
Starting Date	7/6/1993	7/25/1993	7/25/1993	9/16/1992	7/16/1957	4/25/1957	10/22/1956	10/23/1954	10/22/1952
Ending Date	7/7/1993	8/1/1993	8/8/1993	10/16/1992	10/14/1957	10/22/1957	10/22/1957	10/22/1957	10/22/1957
Starting Elev. (ft)	924.3	924.4	924.4	924.6	889.4	891.8	896.2	908.9	917.8
Ending Elev. (ft)	924.0	923.0	922.6	922.7	885.8	885.5	885.5	885.5	885.5
Total Drop (ft)	0.3	1.3	1.8	1.9	3.5	6.2	10.7	23.4	32.2
Rate (in/day)	3.9	2.3	1.5	0.7	0.5	0.4	0.4	0.3	0.2

A similar analysis was conducted to determine the maximum rates of rise. Results are presented in Table 10.

Table 10. Maximum WBM Reservoir Level Rise Rates									
Time Period	1 Day	7 Days	14 Days	30 Days	90 Days	180 Days	1 Year	3 Years	5 Years
Starting Date	9/14/1992	7/14/1958	7/14/1958	7/14/1958	7/2/1958	2/2/2008	5/3/1958	5/2/1958	10/22/1957
Ending Date	9/15/1992	7/21/1958	7/28/1958	8/13/1958	9/30/1958	7/31/2008	5/3/1959	5/1/1961	10/22/1962
Starting Elev. (ft)	918.3	888.7	888.7	888.7	887.9	904.4	885.8	885.8	885.5
Ending Elev. (ft)	923.4	897.5	897.7	901.7	902.6	922.6	907.0	916.7	919.1
Total Rise (ft)	5.1	8.9	9.0	13.1	14.8	18.2	21.2	30.9	33.6
Rate (in/day)	61.2	15.2	7.7	5.2	2.0	1.2	0.7	0.3	0.2

3.4.2 Flood Levels

The determination of flood levels for the reservoir are described in Appendix C, East Locust Creek Reservoir Hydrology Report. The watershed and proposed dam were modeled using the United States Department of Agriculture (USDA) SITES program, version 3.5. Per the guidance in the Natural Resources Conservation Service Technical Release 60, “Earth Dams and Reservoirs” issued July, 2005 (TR-60), and the National Engineering Handbook (NEH), Part 630 - Hydrology, USDA NRCS, version 2010. The report includes analysis of reservoir flood levels based on an assumption that the reservoir starts out full when flood events occur. Table 11 summarizes the flood elevations documented in Appendix C and applies the stage storage information described in Section 2.6 to estimate the area flooded. If a flood event occurs when the reservoir level is below normal pool, the flood elevation and area flooded will be lower than reported.

Table 11 Reservoir Flood Levels			
Annual Exceedance Probability (%)	Return Period (Yrs)	Elevation (ft)	Area (ac)
Normal Pool		922.30	2,328
100%	1	923.35	2,396
50%	2	923.59	2,416
20%	5	924.03	2,452
10%	10	924.44	2,484
4%	25	924.90	2,519
2%	50	925.53	2,565
1%	100	925.97	2,595
Top of Dam		938	3,580

4.0 Conclusions

Based on the assumptions and historical conditions used in this model, it appears that the proposed reservoir level will fluctuate significantly, but in addition to being able to provide 7 MGD will be able to sustain the other two major project purposes which are:

- Provide reservoir based recreational opportunities
- Provide flood damage reduction benefits

In addition, the reservoir will be able to provide in-stream flows with an average value of 0.5 CFS over a range of historical conditions like those from the years 1950 through 2017.

The reservoir should be designed to function normally when the reservoir is down to an elevation of 911 feet. However, this should be considered a general guiding principle not a hard and fast rule. Some situations may warrant designing for lower water. For example, a boat ramp that facilitates emergency operations might be extended to be useable when the lake is below normal operating level. Likewise, for some situations it might be acceptable to design to higher levels. There would be no point in improving an entrance to a shallow cove to provide boat access to elevation 911' if the bottom of the cove is at 912'.

5.0 REFERENCES

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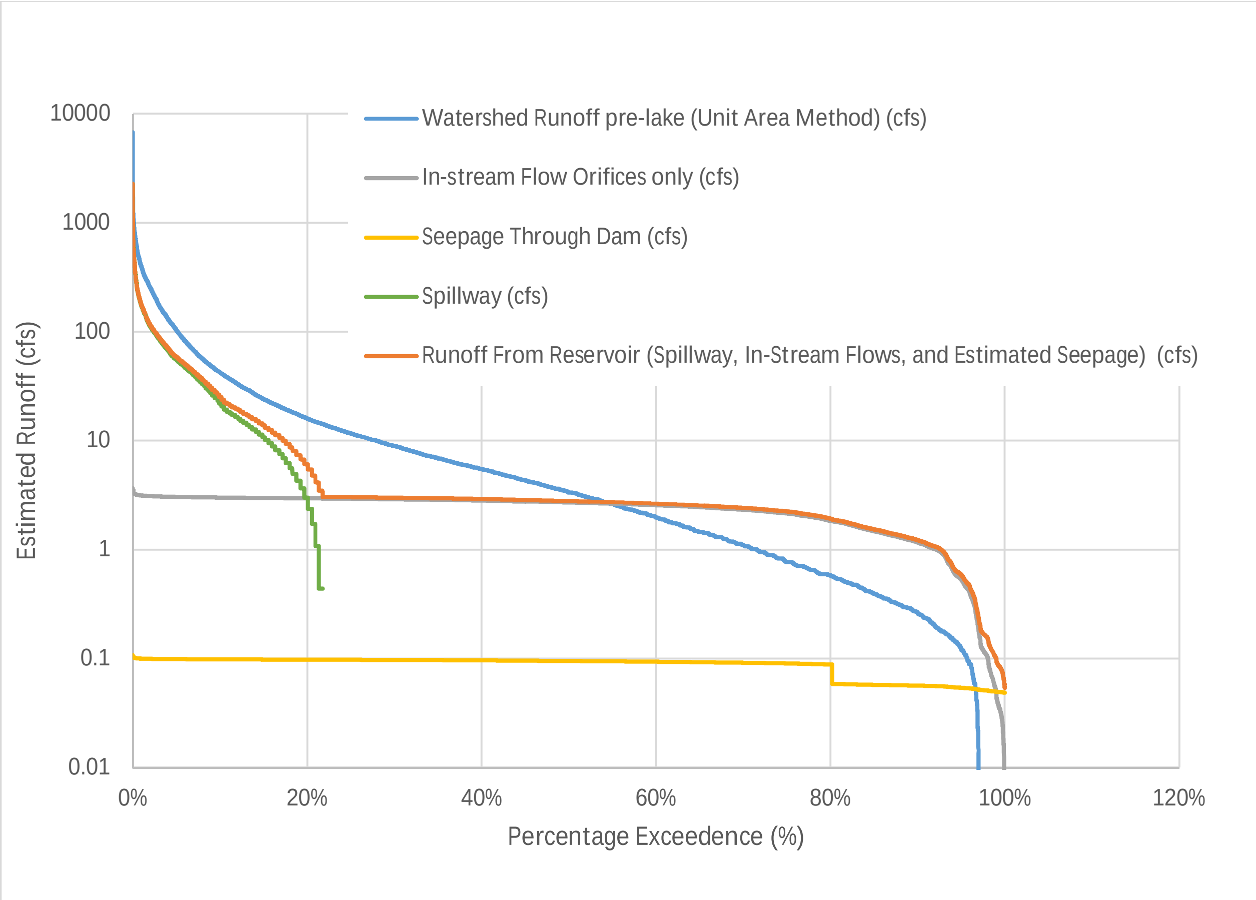
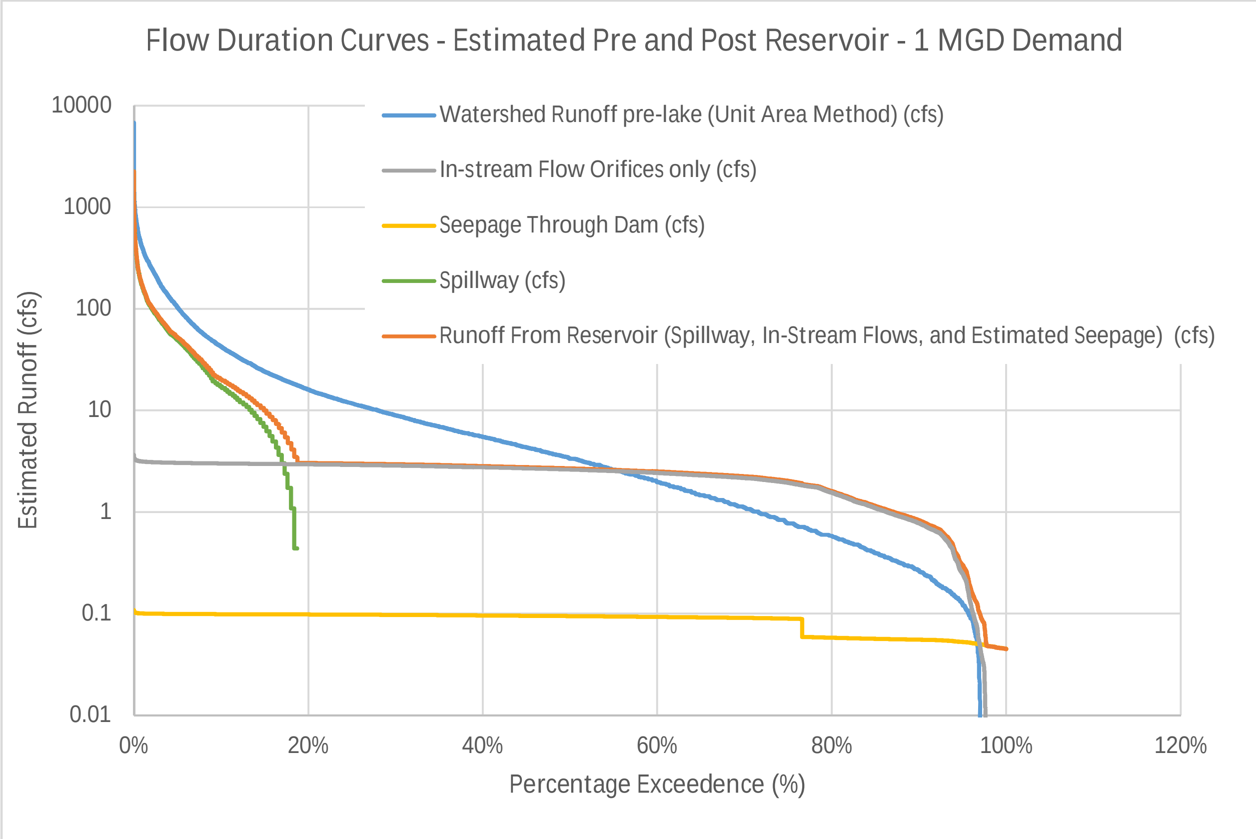
East Locust Creek Reservoir Water Budget Summary

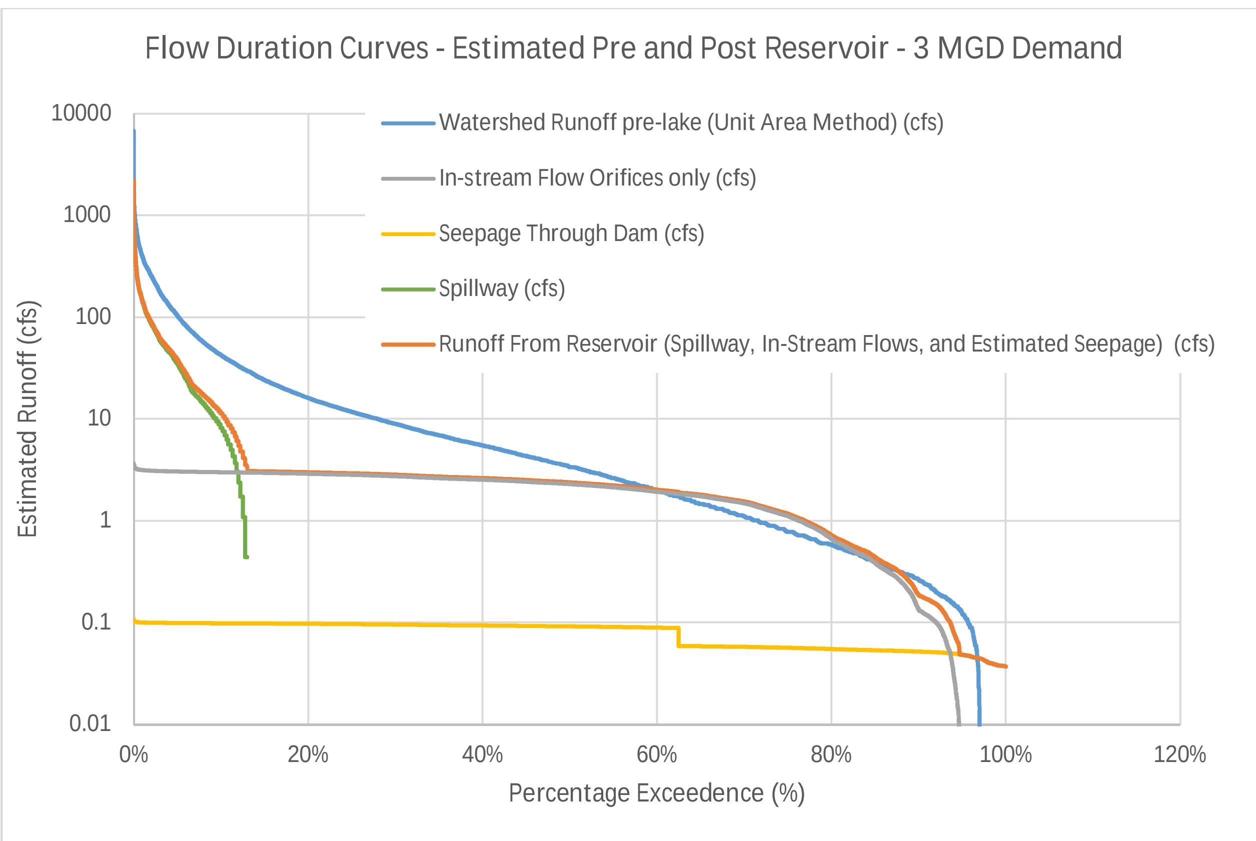
Appendix A.

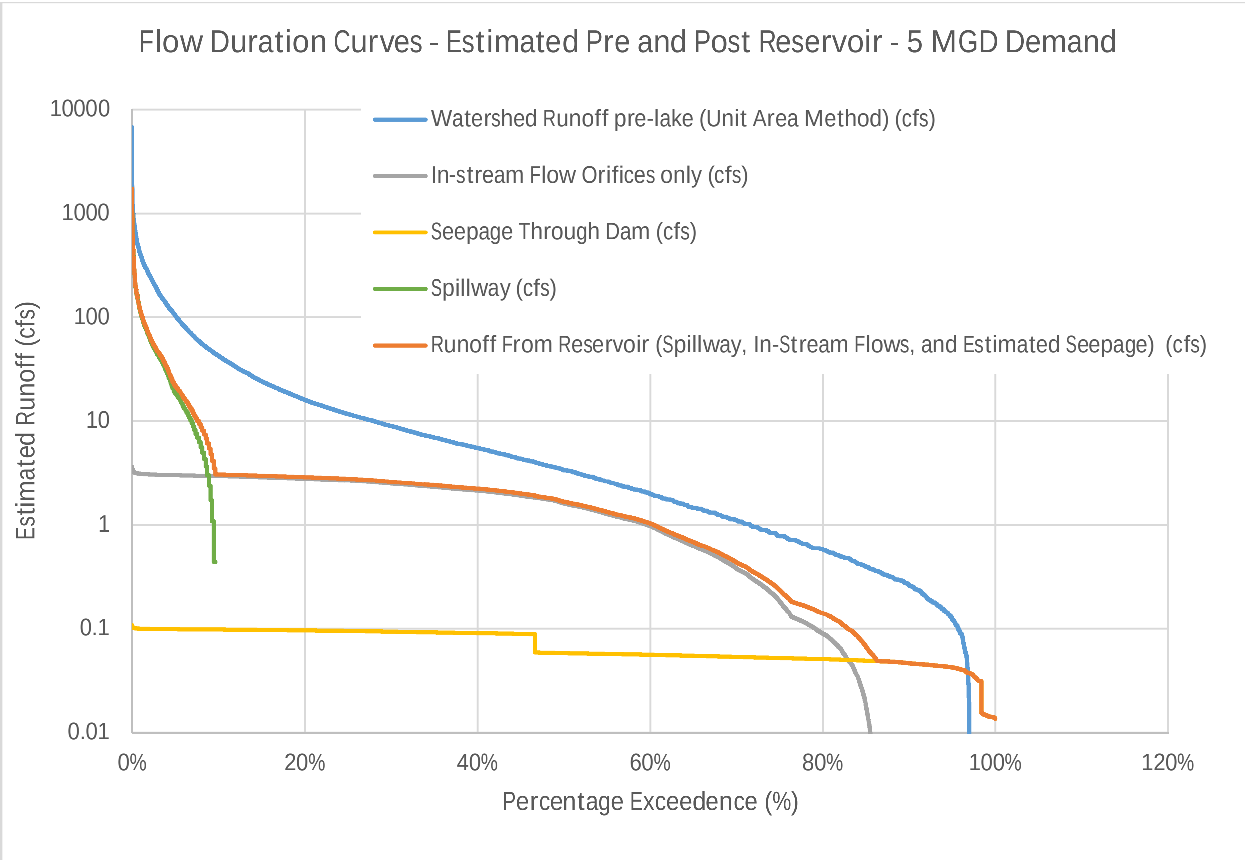
WBM Results at Varying Water Supply Demand Rates

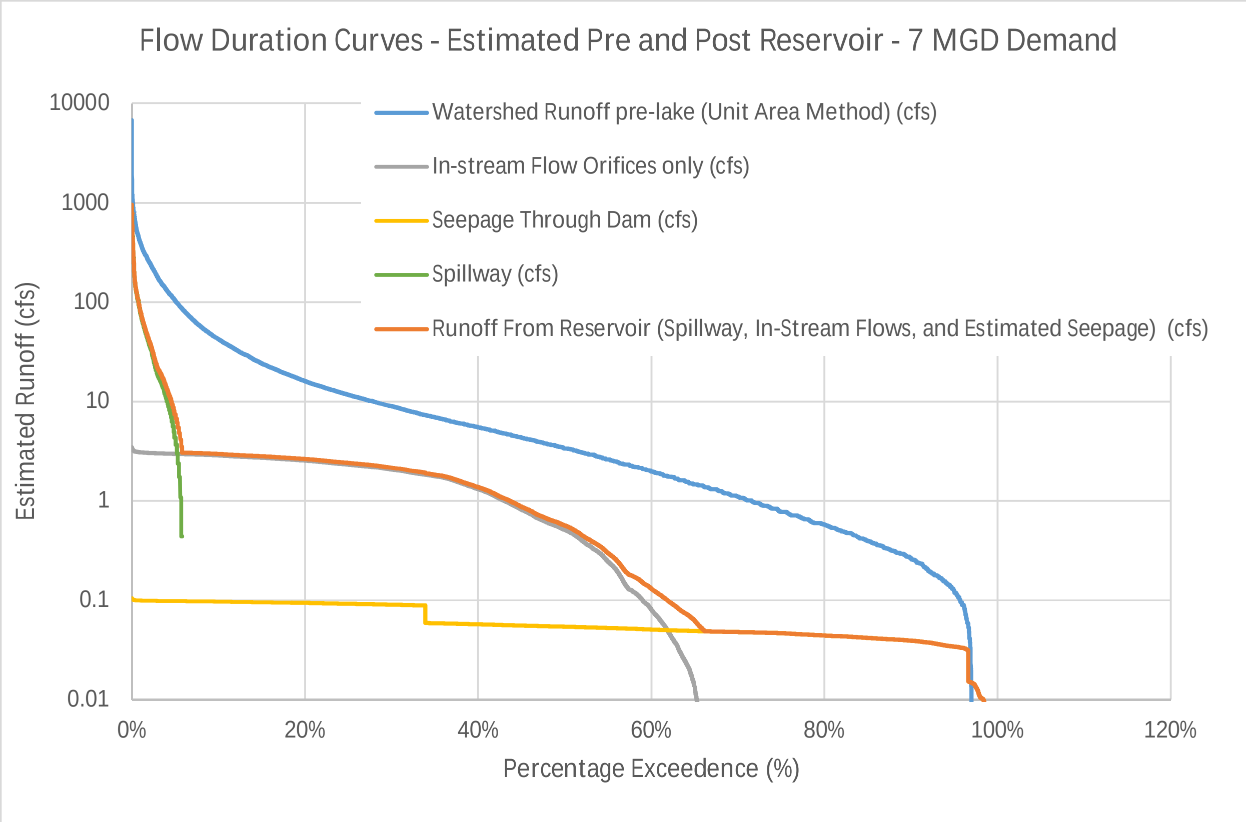
Flow Duration Curves



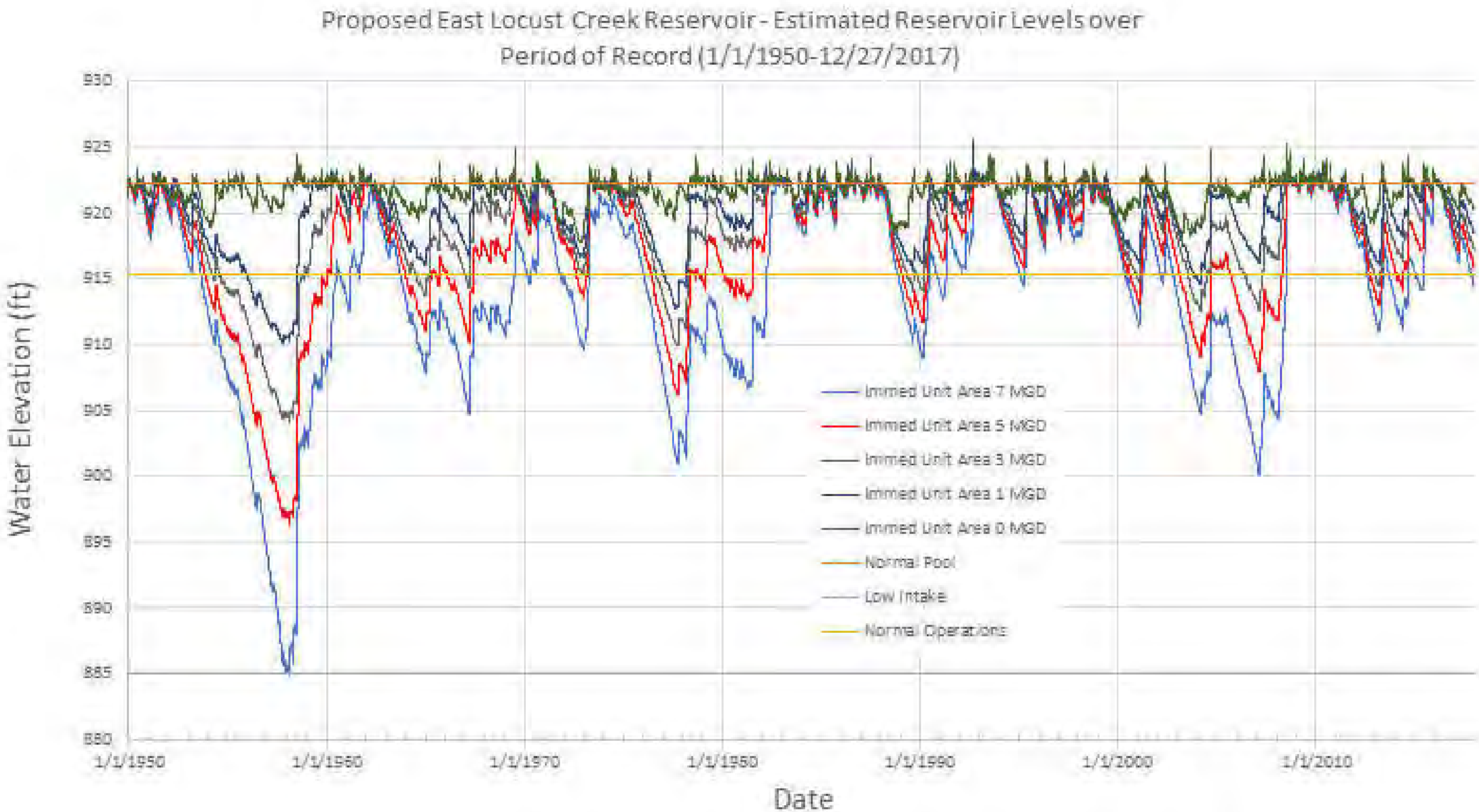

Flow Duration Curves - Estimated Pre and Post Reservoir - 1 MGD Demand
Watershed Runoff pre-lake (Unit Area Method) (cfs)
In-stream Flow Orifices only (cfs)
Seepage Through Dam (cfs)
Spillway (cfs)
Runoff From Reservoir (Spillway, In-Stream Flows, and Estimated Seepage) (cfs)
Estimated Runoff (cfs)
10000
1000
100
10
1
0.1
0.01
0%
20%
40%
60%
80%
100%
120%
Percentage Exceedence (%)


Flow Duration Curves - Estimated Pre and Post Reservoir - 3 MGD Demand
Watershed Runoff pre-lake (Unit Area Method) (cfs)
In-stream Flow Orifices only (cfs)
Seepage Through Dam (cfs)
Spillway (cfs)
Runoff From Reservoir (Spillway, In-Stream Flows, and Estimated Seepage) (cfs)
Estimated Runoff (cfs)
10000
1000
100
10
1
0.1
0.01
0%
20%
40%
60%
80%
100%
120%
Percentage Exceedence (%)


Flow Duration Curves - Estimated Pre and Post Reservoir - 5 MGD Demand
Watershed Runoff pre-lake (Unit Area Method) (cfs)
In-stream Flow Orifices only (cfs)
Seepage Through Dam (cfs)
Spillway (cfs)
Runoff From Reservoir (Spillway, In-Stream Flows, and Estimated Seepage) (cfs)
Estimated Runoff (cfs)
10000
1000
100
10
1
0.1
0.01
0%
20%
40%
60%
80%
100%
120%
Percentage Exceedence (%)


Flow Duration Curves - Estimated Pre and Post Reservoir - 7 MGD Demand
Estimated Runoff (cfs)
10000
1000
100
10
1
0.1
0.01
0%
20%
40%
60%
80%
100%
120%
Percentage Exceedence (%)
Watershed Runoff pre-lake (Unit Area Method) (cfs)
In-stream Flow Orifices only (cfs)
Seepage Through Dam (cfs)
Spillway (cfs)
Runoff From Reservoir (Spillway, In-Stream Flows, and Estimated Seepage) (cfs)

Reservoir Levels



East Locust Creek Reservoir Water Budget Summary

Appendix B.

East Locust Creek Reservoir Hydraulics Report

EAST LOCUST CREEK RESERVOIR
HYDRAULICS REPORT

PREPARED FOR

NORTH CENTRAL MISSOURI REGIONAL WATER COMMISSION

MILAN, MISSOURI

STATE OF MISSOURI
BRIAN H. LADD
NUMBER PE-2014012020
5-31-17
PROFESSIONAL ENGINEER

May 2017

Olsson Associates Project No. A11-1513

OLSSON® ASSOCIATES

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Appendix A Labyrnith Wier Design References and Calculations
Appendix B Spillway Chute and Stilling Basin Design References and Calculations

A reservoir on East Locust Creek near Milan, Missouri (ELCR) is proposed to provide flood control, recreation and water supply for the surrounding community. The proposed reservoir has been in the planning stages since the 1980s. The design of the proposed reservoir will follow the Natural Resource Conservation Service (NRCS) guidelines for dam construction. This report addresses the hydraulics for the dam outfall, spillway chute and stilling basin of the proposed reservoir. The hydrology for ELCR is addressed in "East Locust Creek Reservoir Hydrology Report" Olsson Associates, June 2015.

1.0 DAM OUTFLOW STRUCTURE

The outflow structure for ELCR was designed to provide an economical, low maintenance and safe structure that meets freeboard requirements and allows the dam to fulfill the primary purposes of flood control, recreation and water supply for the surrounding community. The freeboard requirements for the dam are based on passing the Probable Maximum Precipitation (PMP) 5-point storm event with adequate freeboard to the top of the dam. The required calculated freeboard for the dam during the PMP event is 1.13 feet from the peak PMP elevation to the top of the dam. ELCR is proposed as an earthen embankment and erosion must be prevented along the flow path from the reservoir outflow structure to the toe of the dam to ensure the stability of the dam.

1.1 Previously Completed Design

A preliminary sizing of an outflow structure and spillway chute was completed by URS in 2013. The design consisted of a combined emergency and primary spillway with a labyrinth weir as the outfall structure, a concrete spillway chute and an energy dissipation basin. The summary of the outflow structure designed in 2013 can be seen in Table 1. The spillway design met all freeboard requirements and provided a feasible outflow structure for ELCR.

Table 1. Previous Completed Outflow Structure Information

Outflow Structure Summary	
Spillway Width	70 feet
Apex Width	4 feet
Number of Cycles	2
Magnification	1.3
Side Wall Angle	46.05º
Flow Line Elevation	924.8

1.2 Design of Outflow Structure

The design completed by URS was reviewed and it was determined that potential cost savings could be achieved. Several options were investigated to reduce cost and meet the design requirements and reservoir objectives. It was determined that a combined primary and emergency outflow structure labyrinth weir was the most economical option, however a revised configuration was designed to reduce costs.

1.2.1 Design References

The design of labyrinth weirs was completed using the equations developed by Henry Falvey and published in *Hydraulic Design of Labyrinth Weirs* (Falvey, Henry, T., *Hydraulic Design of Labyrinth Weirs, 2003)* additional research and testing into the flow characteristics of labyrinth weirs was completed and published by Brian Crookston (Crookston, Brian, M. *Hydraulic Design and Analysis of Labyrinth Weirs. 1: Discharge Relationships,* J. Irrig Drain Eng. 2013). The research completed by Brian Crookston concluded that coefficient adjustments for the equations developed in the *Hydraulic Design of Labyrinth Weirs* should be used to better analyze the proposed weir structure. In the analysis of the outflow structure the updated coefficients were used to analyze the proposed outflow structure.

1.2.2 Proposed Outflow Structure

The proposed labyrinth outflow structure is a two-stage outflow structure with a lower notch of the outflow structure at elevation 922.3 and the higher portion of the outflow structure set at elevation 924.8. The lower and higher portions of the outflow structure will provide a stage outflow curve that will allow for flood storage and will also maintain adequate freeboard to the top of the dam.

1.2.2.1 Low Flow Notch

The lower notch portion of the outflow structure was set to a length of 27.5 feet. The lower notch of the outflow structure will be constructed in the middle of the proposed outflow structure. The outflow structure was modeled using the equations for the calculation of flow over labyrinth weirs with the amended coefficients. The proposed outflow structure design parameters for the notch portion of the outflow structure are summarized in Table 2.

Table 2. Proposed Outflow Structure Notch Information

Outflow structure Summary	
Outflow structure Width	27.5 feet
Apex Width	4 feet
Number of Cycles	0.5
Magnification	2
Side Wall Angle	27.07º
Flow Line Elevation	922.3

The stage outflow table for the notch portion of the weir can be seen in Table 3. The flow in the notch portion of the spillway ranges from 924.8 to 922.3. The upper stage of the outflow structure begins at 924.8 and the effective length of the outflow structure is increased.

Table 3. Proposed Outflow Structure Notch Stage Outflow

Flow (cfs)	Elevation
753	924.80
642	924.55
536	924.30
436	924.05
342	923.80
256	923.55
179	923.30
112	923.05
59	922.80
19	922.55
0	922.30

1.2.2.2 Full Outflow structure

Once flow from the proposed reservoir reaches 924.8 the outflow will flow over the full width of the weir. The full width of the proposed outflow structure is 55 feet at elevation 924.8. The proposed outflow structure design parameters are summarized in Table 4.

Table 4. Proposed Outflow Structure Information

Outflow Structure Summary	
Outflow structure Width	55 feet
Apex Width	4 feet
Number of Cycles	1
Magnification	2
Side Wall Angle	27.07º
Flow Line Elevation	924.8

The stage outflow for upper full width portion of the can be seen in Table 5. The peak elevation for the notch portion of the outflow structure was calculated to 936.

Table 5. Proposed Full Outflow Structure Stage Outflow

Flow (cfs)	Elevation
10064	936.00
8717	934.60
7482	933.30
6135	931.90
4739	930.50
3399	929.20
1977	927.80
749	926.40
27	925.00
9	924.90
0	924.80

1.2.2.3 Composite Runoff Curve

The composite runoff curve for the labyrinth weir was computed by combining the flow from the notch outflow structure and the flow from the full width weir. The flow from the notch portion of the weir was only added into the composite curve from 922.3 to 924.8. Above elevation 924.8 the flow goes over the full outflow structure. It is assumed that when flow is above elevation 924.8 that the notch outflow structure is flowing full at 753 cfs. Table 6 gives the composite stage outflow curve for the combined outflow structure.

Table 6. Composite Stage Outflow

Flow (cfs)	Elevation
10817	936.00
9469	934.60
8235	933.30
6887	931.90
5491	930.50
4152	929.20
2730	927.80
1501	926.40
780	925.00
762	924.90
753	924.80

The proposed outflow structure will function as the primary and emergency outflow structure for ELCR. The openness of the labyrinth weir outflow structure will prevent clogging by brush and other debris. The outflow structure can also be easily inspected to assure proper function. The 55-foot-wide outflow structure with the labyrinth weir will reduce costs compared to the 70-foot-wide design and will still provide the required outflow from the reservoir.

1.2.3 Storm Routing

The composite elevation discharge curve was input into the SITES software program to determine the water surface elevation for the various storms. The 25-year 24-hour storm and PMP 5-point hydrograph were routed through the proposed outflow structure. The maximum elevation of the 25-year storm is 924.9. The majority of the 25-year outflow from the dam will pass through the lower notch outflow structure and the upper portion of the outflow structure will experience flows in storm events larger than the 25-year storm.

The peak elevation for the PMP 24 hour 5-point storm is 936.02 the top of dam is 938 which results in a freeboard of 1.98 feet.

The calculated freeboard required for the dam is 1.13 feet. The provided freeboard of 1.98 feet is above the required freeboard of 1.13 feet and provides extra protection from overtopping in the extreme storm events.

2.0 SPILLWAY CHUTE AND STILLING BASIN

Flow from the proposed labyrinth weir structure flows down the dam and into East Locust Creek. To protect the integrity of the dam and slow flows before they reach East Locust Creek a concrete spillway and stilling basin were designed for the dam.

2.1 Previously Completed Design

A preliminary sizing of an outflow structure and spillway chute was completed by URS in 2013. The design consisted of a combined emergency and primary spillway with a labyrinth weir as the outfall structure, a concrete spillway chute and an energy dissipation basin. The width for the spillway chute was set at 70 feet to match the width of the previously designed labyrinth weir.

2.2 Proposed Spillway Chute

The proposed spillway chute was modeled using the HEC-RAS software program. Since the spillway functions as the primary and emergency spillway the spillway was sized using the peak PMP outflow, 10,817 cfs, calculated in the SITES modeling. The freeboard requirements for the spillway walls were calculated using the Design of Small Dams, United States Bureau of Reclamation, 1987. The spillway width was set at 55 feet to match the width of the labyrinth spillway. The labyrinth weir was modeled as an inline structure in HEC-RAS. The weir coefficient of the inline structure was adjusted to obtain the peak elevation that was calculated in the routing of the PMP storm in SITES. The proposed concrete spillway chute was modeled in HEC-RAS with an interpolated cross section approximately every 10 feet. The freeboard was calculated using the formula from the *Design of Small Dams*. The flow in the spillway is supercritical and will be dissipated in a stilling basin before the flow enters the receiving stream. The summary of the spillway chute can be seen in Table 7.

Table 7. Proposed Spillway Chute Information

Spillway Chute Summary	
Chute Width	55 feet
Minimum Wall Height	10 feet
Maximum Velocity	52 feet/sec
Maximum Froude Number	4.66
Channel Slope	12.5%

The wall heights along the spillway chute were adjusted to match site conditions, however the minimum 10-foot wall height was maintained along the spillway chute.

2.3 Proposed Stilling Basin

The stilling basin was designed to dissipate energy from the spillway chute before the flow enters East Locust Creek. The stilling basin was designed using the guidance in the USBR Design of Small Dams. The Froude number is greater than 4.5 and the velocity in the channel upstream of the stilling basin is less than 60 feet/sec therefore a Type III basin was selected to dissipate the energy from the spillway chute. Table 8 provides a summary of the stilling basin.

Table 8. Proposed Stilling Basin Information

Stilling Basin Summary	
Basin Width	55 feet
Height of Hydraulic Jump	24.5 feet
Required Freeboard	7.7 feet
Wall Height	32.2 feet
Chute Block Height	4.1 feet
Baffle Block Height	5.7 feet
Length of Basin	56.3 feet
End Sill Height	5.1 feet

2.4 Proposed Channel Downstream of Stilling Basin

The channel downstream of the stilling basin will convey the water from the stilling basin to East Locust Creek. A trapezoidal channel is the most efficient design shape, and the channel will be lined with riprap to prevent erosion. The channel downstream will also provide the required tailwater for the stilling basin. The downstream channel was modeled in HEC-RAS in conjunction with the spillway chute and stilling basin. The channel at the outlet of the stilling basin will be 50-foot-wide at the base, and the channel will transition to a 20-foot-wide base to increase the tailwater in the stilling basin and limit the extents of excavation. The velocity in the channel at the outlet to East Locust Creek in the 100-year event is approximately 13 feet/sec. Riprap will be utilized for energy dissipation and slope protection at the transition from the proposed trapezoidal channel to the natural stream channel. The flow parameters in the channel for the PMP storm and the 100-year event are summarized in Table 9 and Table 10.

Table 9. Downstream Channel, PMP Storm Summary

Downstream Channel, PMP Storm, Summary	
Channel Width	20 feet
Peak Velocity	24 feet/sec
Peak Flow Depth	13.0 feet

Table 10. Downstream Channel, 100-year Storm Summary

Downstream Channel, 100-year Storm, Summary	
Channel Width	20 feet
Peak Velocity	13 feet/sec
Peak Flow Depth	3.1 feet

The rip-rap for the trapezoidal channel was sized for the 100-year event using the HEC No. 11. The calculated required D_{50} riprap size for the channel during the 100-year event is 1.44 ft. MoDOT Type 4 ditch lining with a D_{50} of 1.58 feet was selected as the ditch lining gradation for the channel.

APPENDIX A
LABYRNITH WIER DESIGN REFERENCES AND CALCULATIONS

OLSSON ASSOCIATES

If the function of the roughness is to increase the boundary layer thickness, then the roughness does not need to be placed on the weir crest. In fact, placing the roughness further upstream on the curved crest may be more effective, and a smaller roughness could be used.

As opposed to nappe vibration, which creates noise, the most important consideration with surging deals with the fluctuating pressures on the walls. Frequencies of oscillation are usually measured in a model study to ensure that the surging frequency does not coincide with the natural frequency of the wall.

Attachment A

Chapter 8 Design

Significant Parameters

Studies on labyrinth weirs have shown that the most significant parameters are the length to width ratio, L/W; the total head to crest height ratio, H_o/P; and the sidewall angle, α. The aspect ratio, W/P, which others found to be important, has been replaced by a disturbance to sidewall length ratio, L_d/B. Rounding the crest has only a minor effect on improving the discharge coefficient (< 3%). Finally, the number of weir cycles, n, is not a significant parameter on the discharge characteristics of labyrinth weirs. The approach flow conditions to the labyrinth weir are significant in determining the discharge coefficient for the spillway.

General Guidelines for Parameter Selection

Headwater Ratio

The headwater ratio is the total head on the weir divided by the weir height, H_o/P. Because the discharge coefficient decreases with increasing head, labyrinth weirs have the greatest application where the head is small. Lux (1989) recommends that the maximum headwater ratio be in the range of 0.45 to 0.50. Nevertheless, some labyrinth spillways have been designed with headwater ratios as large as 1. The maximum headwater ratio is more a question of the range over which the model discharge coefficients were determined rather than some absolute value. For example, the maximum headwater ratio for the Tullis et al (1996) tests is an H_o/P of 0.9. Because the equations to be used in the analysis are only valid up to an H_o/P of 0.9, this is the upper headwater limit. If higher values are necessary, then a physical model study of the structure is required.

Vertical Aspect Ratio/Sidewall Angle

The vertical aspect ratio is the width of a weir cycle divided by the weir height, W/P. Taylor (1968) recommends that to minimize the effect of nappe interference, the vertical aspect ratio should be larger than 2. For design purposes, a value between 2.0 and 2.5 is recommended by Lux (1989) for initial computations. As shown in Chapter 3, Nappe Interference, this ratio does not have a significant effect on nappe interference, as has been thought up until now. This criterion has been superseded by the disturbance length concept described below

Magnification Ratio

The magnification ratio is the length of the labyrinth crest divided by the cycle width, L/W. The limit for the curves of Tullis (1994) is an angle of 6°, which corresponds to a magnification ratio of about 9.5. As shown below, the effectiveness of a labyrinth weir decreases rapidly as the magnification ratio exceeds 10. With a magnification ratio of less than 2, consideration should be given to widening the intake or using an ogee crest that is curved in plan rather than using a labyrinth weir.

Sidewall Angle/Magnification

With a triangular labyrinth, the sidewall angle and the magnification are interrelated. The angle is given by

$$\alpha = \sin^{-1}\left(\frac{W}{2 \cdot B}\right) = \sin^{-1}\left(\frac{1}{m}\right) \tag{1}$$

in which m = the magnification ratio.

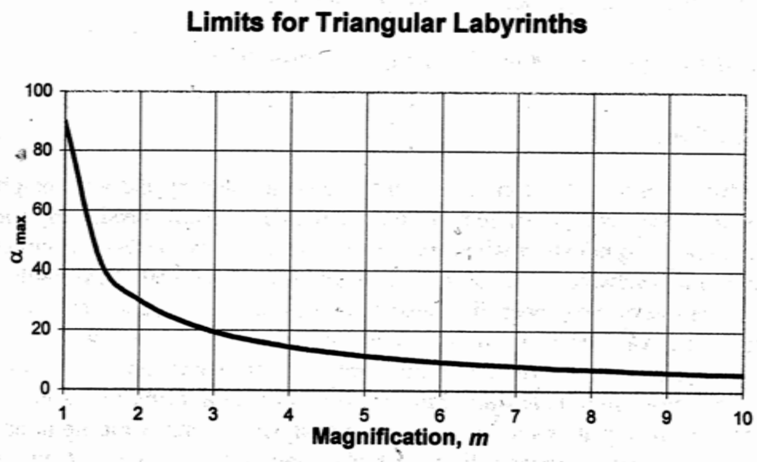


Figure 1. Maximum Angle for Triangular Labyrinth Weirs

Figure 1 gives the maximum angle for a triangular labyrinth weir. With a trapezoidal labyrinth, the angle of the sidewall will be less than that shown in Figure 1 for a given magnification. That is, the relationship between the magnification and the sidewall angle will lie below the curve with a trapezoidal labyrinth.

Efficacy

Actually, the magnification that is chosen applies only to small values of head. As the head increases, the discharge coefficient decreases. Thus, if a labyrinth is to pass the maximum discharge for a given reservoir elevation, then the product of the discharge coefficient and the magnification should be a maximum. Dividing this product by the discharge coefficient for a straight weir is called the efficacy. Efficacy is given by

$$\varepsilon = \frac{C_d(\alpha) \cdot M}{C_d(90^\circ)} \tag{2}$$

in which C_d (α) indicates that the discharge coefficient is a function of the sidewall angle.

Efficacy is essentially the same as the Q_L/Q_N parameter used by Taylor (1968). However, efficacy incorporates the magnification and the effect of the sidewall angle into one parameter. Thus, with this parameter, the benefits of changes in the labyrinth geometry can be estimated quickly during the design process.

The effects of head on the weir and the sidewall angle are clearly shown in Figure 2. The discharge coefficient for different angles is obtained from Figure 8 in Chapter 5, Design Curves.

The magnification parameter for a triangular labyrinth as a function of the sidewall angle is obtained from Figure 1 or Equation 1 above. For example, with an H_o/P of 0.7 and a sidewall angle of 18°, the discharge coefficient is 0.485, the magnification is

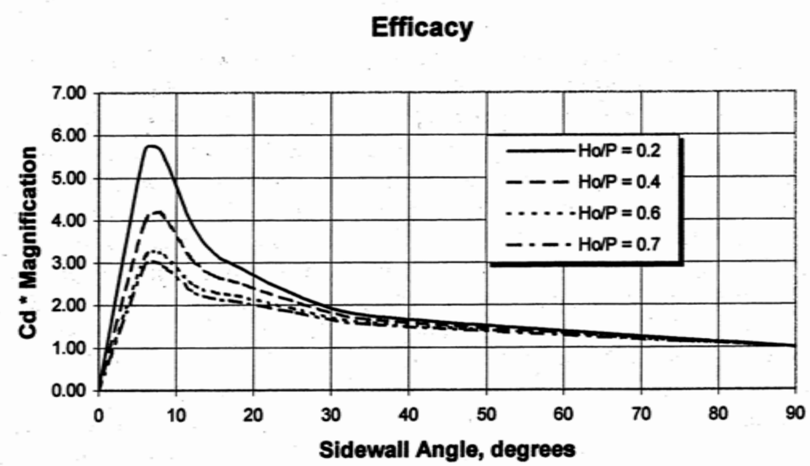


Figure 2. Efficacy for triangular weirs

3.24, and the discharge coefficient for the straight weir is 0.76. This gives an efficacy equal to 2.1. This means that the labyrinth can pass a little more than twice the flow for a given head than can a straight weir. However, if the sidewall angle is decreased to 8°, the efficacy increases to 3 because C_d (α) is 0.315, *m* is 7.18, and C_d (90°) is 0.76. Thus, the weir can pass three times the flow for a given head than can a straight weir.

The efficacy reaches a maximum value for all head ratios at a sidewall angle of about 8°. This angle corresponds to a magnification of 7.2. The efficacy decreases rapidly as the angle becomes smaller than 8°. In addition, Figure 2 shows that the efficacy decreases as the head over the weir increases.

The effects for a trapezoidal weir are similar to those for a triangular weir except that the efficacy does not approach zero as the sidewall angle approaches zero. With a trapezoidal or rectangular weir, the apex distance separates the two walls. For example, with a rectangular weir, a zero sidewall angle means that the two walls are parallel.

Taylor (1968) studied the decrease in the discharge for trapezoidal and rectangular weirs and presented his data in the form of Q_L/Q_N, as shown in Figure 3. This figure shows that the sidewall angle of 9.5° has a higher discharge than does the 7° angle. Unfortunately, the data are too incomplete to show the effect at larger angles. Note that Figure 2 is for a quarter-round crest, whereas the curves in Figure 3 are for a sharp crest. In addition, Figure 2 contains both the magnification and the angle effects in one curve. This is an area in which more research is needed.

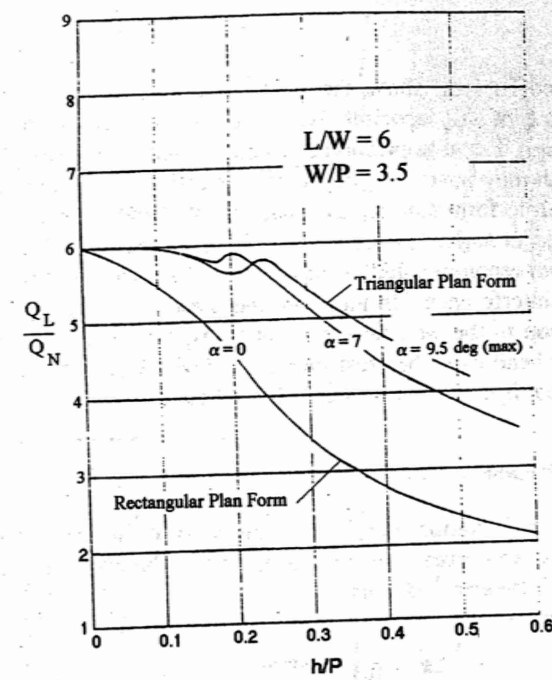


Figure 3. Effect of sidewall angle on discharge from Taylor, G., (1968). "The performance of labyrinth weirs." PhD thesis, University of Nottingham, Nottingham, England.

Apex Ratio

The apex ratio is the width of the apex divided by the cycle width, 2a/W. The most efficient labyrinth weir is the triangular plan form. Interference increases with an increase in the apex ratio. However, construction considerations often dictate the use of a finite apex width. Values of the apex ratio that are less than 0.08 will not have a significant effect on the performance of a labyrinth weir. This is because of two effects. One is interference at the upstream apex. With interference, the upstream section of the sidewall does not convey a significant amount of water. Therefore, replacing the sharp corner of the triangular labyrinth with a blunt apex has little effect on the overall performance of the labyrinth. Similarly, the downstream end of a triangular labyrinth is essentially a stagnation zone. This is made evident by the rise in the water surface profile at the downstream end of the channel between the sidewalls, as shown in Figure 1 in the Chapter 5, Design Curves. Because of the stagnant zone, the downstream end of the labyrinth can also be replaced with a blunt apex with little effect on the overall performance of the weir.

Crest Shape

As the discharge coefficients show, the crest shape does not have a significant effect on the performance of the labyrinth weir. The quarter-round and the half-round shapes are commonly found in prototype structures. An ogee shape that is not thicker than the wall width may have a slightly higher coefficient at small heads. This shape is not more difficult to form than are the quarter-round and the half-round shapes, and it may stay aerated at higher heads. The full ogee shape used by Megalháes and Lorena (1989) is not recommended. It has a lower discharge coefficient at high heads because of nappe interference. In addition, the mass on the top of the wall requires much more attention to the wall design. This configuration will be more susceptible to vibration as the head over the crest increases. The effect of the crest shape on the discharge coefficient is given in Chapter 4, Crest Shapes.

Interference Length Ratio

As shown in Chapter 3, Nappe Interference, the ratio of the disturbance length to the sidewall length is an important consideration to limit the effects of interference. The disturbance length is determined from

$$\frac{L_{de}}{h} = 6.1 \cdot e^{-0.052 \cdot \alpha} \tag{3}$$

in which α = the sidewall angle in degrees. Here, the equation of Indlekofer and Rouvé (1975) is used instead of the suggested equations based on model studies of labyrinth weirs. When research has been completed on the interference with labyrinth weirs, this equation will be replaced with a more accurate relationship.

The ratio of the disturbance length to the sidewall length, L_{de}/B, should be less than or equal to 0.3. This can be written as

$$\frac{L_{de}}{B} = \frac{h}{B} \cdot 6.1 \cdot e^{-0.052 \cdot \alpha} \leq 0.3 \tag{4}$$

Approach Flow Conditions

Houston (1983) made a very important study of the effect of placement of the labyrinth weir relative to the reservoir. As shown in Figure 4, the labyrinth can be placed within the chute in either the normal or the inverted position, at the entrance to the chute, or extending into the reservoir. With a magnification of 5 and the orientation of the labyrinth in the normal position, the discharge was 9% greater than it was in the inverted position. In the normal position, the friction on the chute walls is a minimum.

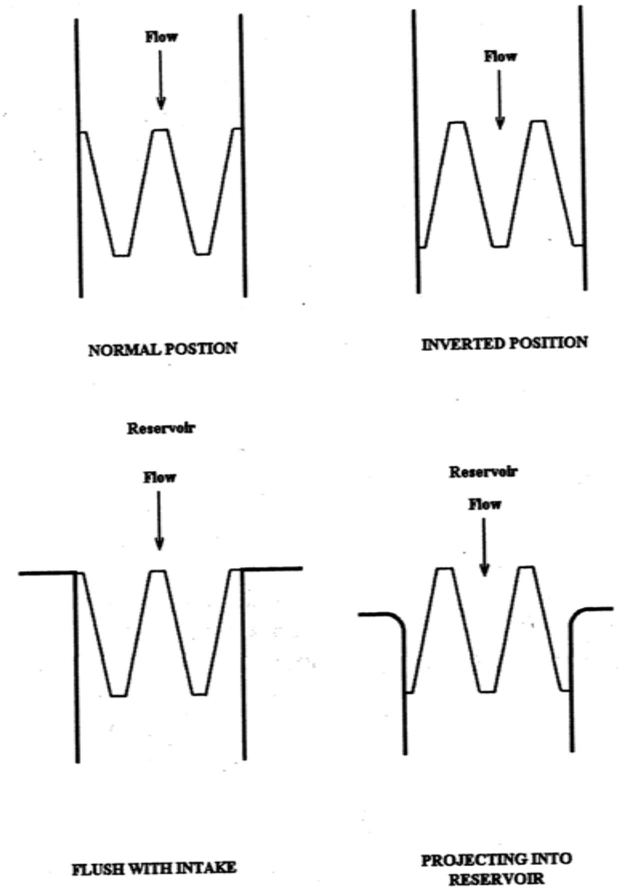


Figure 4. Labyrinth weir locations and orientations after Houston (1983)

As the labyrinth is moved into the reservoir, its capacity increases. The discharge with the labyrinth projecting into the reservoir is 20% greater than it is when in the normal position. However, a labyrinth projecting into the reservoir must use the less-efficient inverted position to tie the weir into the abutment. The curves used in the Excel spreadsheet, described below, are for a labyrinth weir placed in the normal position.

If a greater length of labyrinth is needed to pass a given discharge, the width of the approach section can be increased. For example, the width of the Avon spillway was made about 5.5 times wider than the downstream channel by creating a wide approach section. Similarly, the labyrinth width of the Kizilcapinar and Sarioglan spillways were made wider through the use of an expanded upstream approach channel. The alignment of the Avon and the Kizilcapinar spillways were curved, whereas that of the Sarioglan labyrinth was straight. Details of the alignments of these three spillways are given in Appendix A.

Downstream Channel

Considerations concerning the effects of the downstream channel geometry are given in Chapter 6, Downstream Chute.

Layout and Quantities

The dimensions of a labyrinth weir are shown in Figure 5. Stevens developed an Excel spreadsheet for URS[1] to be used in the design of labyrinth spillway installations. His spreadsheet was extensively modified to include the curves of Tullis (1994) and all the updated design limits. The spreadsheet is available in both English and metric units from falvey@members.asce.org.

[1] M. A. Stevens, PO Box 3263, Boulder, CO, 80307. Tel (303) 444-7120.

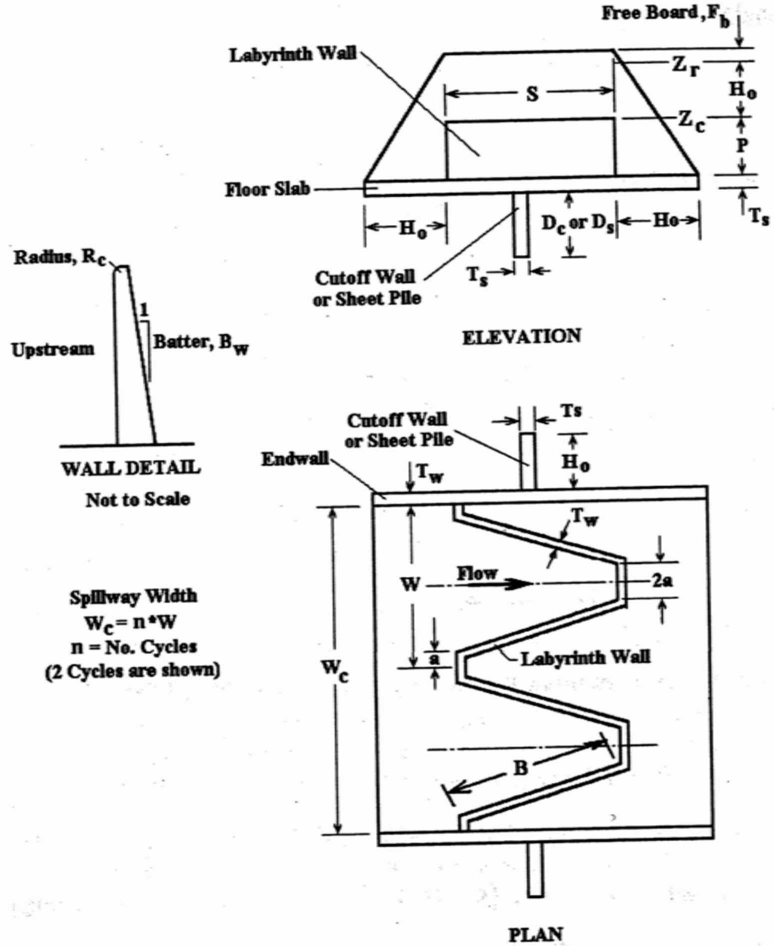


Figure 5. Definition Sketch for Labyrinth Spillway Geometry

Dimensions

The dimensions of the labyrinth weir are determined as follows:

Width of each cycle

$$W = \frac{W_c}{n} \tag{5}$$

Crest length of weir

$$L_w = \left(\frac{L}{W}\right) \cdot W = m \cdot W \tag{6}$$

Sidewall angle

$$\alpha = \tan^{-1}\left(\frac{W - 4\cdot a}{2\cdot S}\right) \tag{7}$$

Length of one leg of weir crest

$$B = \frac{L - 2\cdot(2\cdot a)}{2} \tag{8}$$

Depth of labyrinth weir

$$S = \sqrt{B^2 - \left(\frac{W - 2\cdot a}{2}\right)^2} \tag{9}$$

Head on weir

$$H_o = Z_r - Z_c \tag{10}$$

Quantities

The volume computations to estimate the materials and costs are as follows:

Weir walls

$$V_w = n\cdot L\cdot P\cdot T_w \tag{11}$$

End walls

$$V_e = (P + H_o + F_b)\cdot(S + H_o)\cdot 2\cdot T_w \tag{12}$$

Slab

$$V_s = (S + 2\cdot H_o)\cdot W_s\cdot T_s \tag{13}$$

Concrete cutoff wall

(Without sheet piles)

$$V_c = [W_s\cdot D_c + 2\cdot(D_c + T_s + P + H_0 + F_b)]\cdot T_s \tag{14}$$

(With sheet piles)

$$V_c = W_s\cdot 2\cdot D_c \tag{15}$$

Sheet piles

$$A_s = (W_s\cdot D_s) + D_s\cdot(P + H_0 + F_b) \tag{16}$$

Reinforcing bars

$$M(lb) = \frac{\gamma_s}{3.5}\cdot(V_w + V_e + V_s + V_c) \tag{17}$$

in which γ_s = the unit weight of steel.

Discharge Coefficient

The discharge coefficients are obtained from Table 1 based on the design curves of Tullis (1994). In this table, the discharge coefficient is computed from

$$C_d = A_1 + A_2\frac{H_o}{P} + A_3\left(\frac{H_o}{P}\right)^2 + A_4\left(\frac{H_o}{P}\right)^3 + A_5\left(\frac{H_o}{P}\right)^4 \tag{18}$$

in which the discharge is given by

$$Q = C_d\cdot\frac{2}{3}\sqrt{2\cdot g}\cdot L\cdot H_o^{3/2} \tag{19}$$

Interpolation for other angles should be done by first determining the coefficient for the adjacent angles and the given head ratio. Then, use linear interpolation between the two adjacent angles. Do not interpolate between the coefficients!

The discharge curves of Tullis (1994) are valid for an H_o/P of less than or equal to 0.9, for interference ratios less than those shown in Figure 7, and for sidewall angles greater than or equal to 6°.

Because the head ratio, H_o/P, should be less that 0.9, a maximum interference criterion of L_{de}/B equals 0.35 is recommended for use in the spreadsheet.

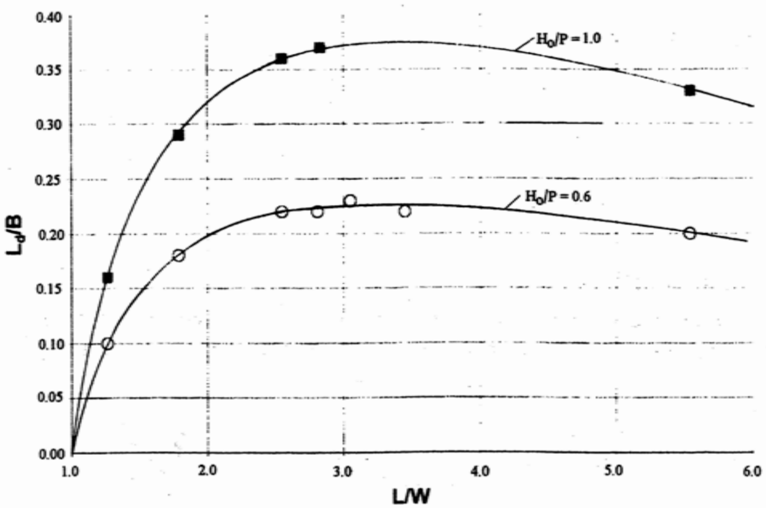


Figure 6. Maximum interference ratios for the design curves based on data from Tullis, J.P., Nosratollah, A.,.and Waldron, D., (1995). "Design of labyrinth spillways. " *American Society of Civil Engineering, Journal of Hydraulic Engineering,* 121(3), 247-255.

Design Procedure

Steps

The steps in the design procedure are based on the availability of a spreadsheet that facilitates the process.

- Determine the labyrinth's location and channel alignment based on site conditions.
- Define the maximum allowable operating head on the weir that will satisfy operational specifications.
- Define the maximum discharge to be passed at the maximum allowable operational head.
- Use the spreadsheet to determine the spillway configuration that will pass the discharge at the specified operational head. Varying the floor elevation, the magnification, and the number of cycles will determine the most economical configuration. Figure 5 in Chapter 3, Nappe Interference, shows that the smallest slab to support a labyrinth weir is the one that has the largest number of cycles. Therefore, the most economical design will be one with the smallest magnification ratio and the maximum number of cycles that does not violate the head and interference criteria. Violation of the interference criteria means that experimental conditions for which the discharge equations were developed are being exceeded. Thus, the discharge values may be in error. If the economics of the structure indicate that higher interference values are desirable, then a model study of the structure should be conducted to verify the performance at higher heads. As shown in Chapter 3, Nappe Interference, structures with interference values as high as 0.6 have performed satisfactorily.

- The designer must pay close attention to the estimated wall and slab thickness, as well as to the depth of the cut off wall. In addition, the unit prices should be as accurate as possible. These variables have a significant influence on the cost of the structure.
- Perform reservoir routing to verify that the selected design will meet the specified maximum head and discharge requirements.
- Analyze the approach flow conditions for high-velocity concentrations that may decrease the capacity of the spillway. For this analysis, a mathematical or a physical model study may be necessary.
- If either the reservoir routing or the approach flow conditions are not satisfactory, redesign the spillway by revising the approach flow width, changing the alignment, and varying the spillway input parameters using the spreadsheet.

Spreadsheet

For the Excel spreadsheet shown on the following pages, the required input is listed under the section "User Input." All other items are filled in automatically. The spreadsheet calculates the pertinent spillway dimensions, the maximum discharge, the estimated cost of the installation, a detailed discharge curve, and the labyrinth dimensions. The coordinates for one cycle are computed and plotted.

LABYRINTH WEIR DESIGN
No Approach Velocity

PROJECT:	Hyrum	**TIME:**	16:50:51
PROJECT NO.	1	**DATE:**	02-Sep-02
FLOOD CRITERIA:	PMF	**BY:**	HTF

USER INPUT

Max. Res	**Zr**	4678.0 ft	**Thickness**		
Crest el.	**Zc**	4672.0 ft	**Wall**	**Tw**	1 ft
Floor el.	**Zf**	4660.0 ft	**Slab**	**Ts**	1 ft
Spillway width	**Ws**	60.0 ft	**Cutoff Depth**		
Apex Width	**2a**	4 ft	**Sheet Pile**	**Ds**	0 ft
No. of cycles	**n**	2	**Conc Wall**	**Dc**	4 ft
Magnification	**L/W**	4.95			

CHECK ON RATIOS

L_d/B = 0.33 — **Ld/B RATIO IS OK**

H_o/P = 0.50 — **Ho/P RATIO IS OK**

L/W RATIO IS OK

Note: L_d/B must be <= 0.35
Ho/P must be <= 0.9
α must be >= 6 deg

LABYRINTH DIMENSIONS (Per Cycle)

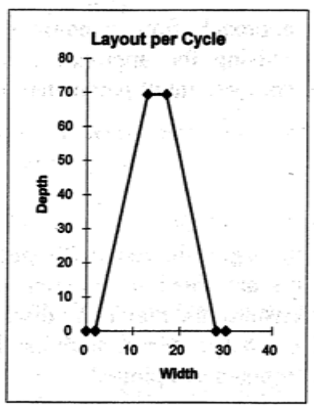
Wall Height	**P**	12 ft
Width	**W**	30.00 ft
Length	**L**	148.50 ft
Wall Length	**B**	70.25 ft
Depth	**D**	69.38 ft
Head max	**H**	6.00 ft
Wall Angle	**α**	9.01 deg
Length of Interference	L_b	22.99

CREST LAYOUT
(One Cycle)

X	Y
0	0
2.00	0
13.00	69.38
17.00	69.38
28.00	0
30.00	0

COST CALCULATION

	Unit price $/unit	Units	Cost $
Weir wall, cy			
Abutment walls, cy	350	132	46,200
Slab, cy	350	106	37,133
Concrete cutoff, cy	225	181	40,692
Sheet pile, sf	200	86	17,185
Reinforcement, lb	20	0	0
	0.65	70,682	45,943



ESTIMATED COST
$187,154

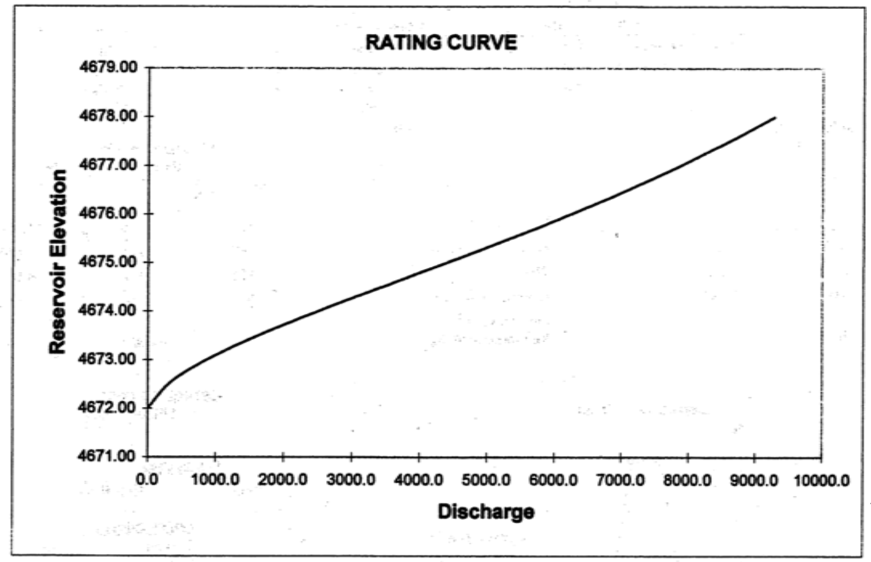
DISCHARGE
Qmax 9,285 **cfs**

COEFFICIENTS

Column	2.00
Cd lower	0.38
Cd Upper	0.44
Cd	0.40
Efficacy	2.59

RATING CURVE

HEAD	H_o/P	C_{lower}	C_{upper}	C_d	Q	RES
6.00	0.50	0.38	0.44	0.40	9285	4678.00
5.40	0.45	0.41	0.46	0.42	8455	4677.40
4.80	0.40	0.44	0.49	0.45	7559	4676.80
4.20	0.35	0.47	0.51	0.48	6580	4676.20
3.60	0.30	0.50	0.53	0.51	5518	4675.60
3.00	0.25	0.53	0.55	0.53	4394	4675.00
2.40	0.20	0.54	0.56	0.55	3247	4674.40
1.80	0.15	0.55	0.57	0.56	2140	4673.80
1.20	0.10	0.55	0.56	0.55	1155	4673.20
0.60	0.05	0.53	0.53	0.53	393	4672.60
0.00	0.00	0.49	0.49	0.49	0	4672.00



Discharge Coefficient Table Tullis et al. (1995)

	Angle wall makes with centerline α							
	6	8	12	15	18	25	35	90
A0	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49
A1	-0.24	1.08	1.06	1.00	1.32	1.51	1.69	1.46
A2	-1.20	-5.27	-4.43	-3.57	-4.13	-3.83	-4.05	-2.56
A3	2.17	6.79	5.18	3.82	4.24	3.40	3.62	1.44
A4	-1.03	-2.83	-1.97	-1.38	-1.50	-1.05	-1.10	

LABYRINTH WEIR DESIGN

PROJECT:	Serne	**TIME:**	16:54:16
PROJECT NO.	1	**DATE:**	02-Sep-02
FLOOD CRITERIA:	PMF	**BY:**	*HTF*

USER INPUT

Max. Res	**Zr**	79.6 m	**Thickness**		
Crest el.	**Zc**	78.5 m	**Wall**	**Tw**	0.5 m
Floor el.	**Zf**	76.0 m	**Slab**	**Ts**	1 m
Spillway width	**Ws**	15.0 m	**Cutoff Depth**		
Apex Width	**2a**	3 m	**Sheet Pile**	**Ds**	0 m
No. of cycles	**n**	1	**Conc Wall**	**Dc**	2 m
Magnification	**L/W**	4			

CHECK ON RATIOS

L_d/B = 0.15 **Ld/B RATIO IS OK**

H_o/P = 0.44 **Ho/P RATIO IS OK**

L/W RATIO IS OK

Note: L_d/B must be <= 0.30

Ho/P must be <= 0.9

α must be >= 6 deg

LABYRINTH DIMENSIONS (Per Cycle)

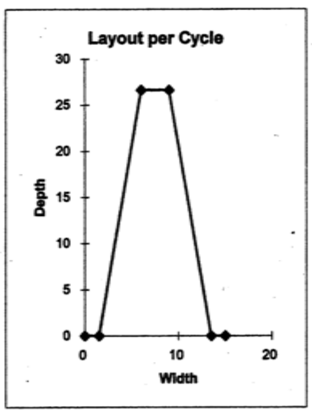
Wall Height	**P**	2.5 m
Width	**W**	15.00 m
Length	**L**	60.00 m
Wall Length	**B**	27.00 m
Depth	**D**	26.62 m
Head max	**H**	1.10 m
Wall Angle	**α**	9.59 deg
Length of Interference	**L_d**	4.09 m

CREST LAYOUT
(One Cycle)

X	**Y**
0	0
1.50	0
6.00	26.62
9.00	26.62
13.50	0
15.00	0

COST CALCULATION

	Unit price Euros/unit	**Units**	**Cost Euros**
Weir wall, m^3	150	75	11,250
Abutment walls, m^3	140	128	17,853
Slab, m^3	125	432	54,042
Concrete cutoff, m^3	300	508	152,400
Sheet pile, m^2	200	0	0
Reinforcement, kg	3.4	94,807	322,344



ESTIMATED COST
557,889 Euros

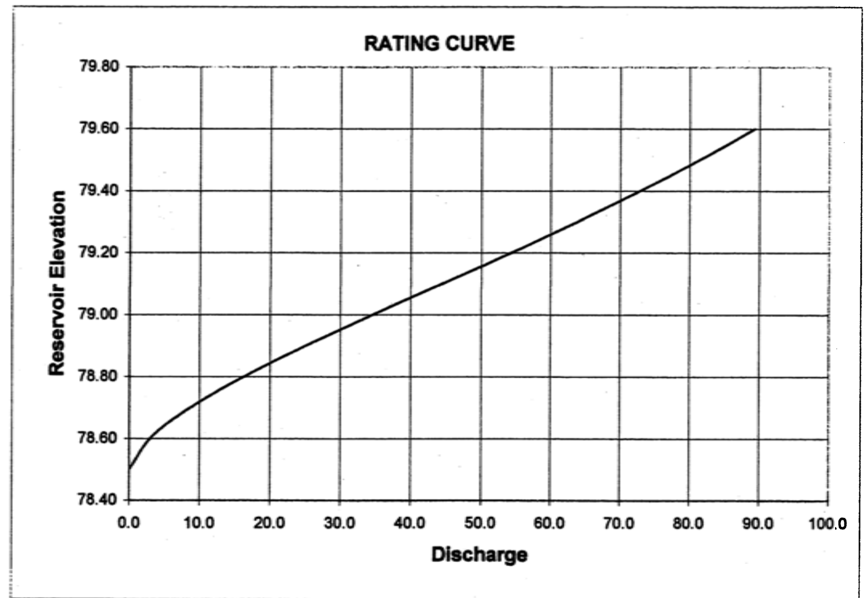
DISCHARGE
Qmax 89.3 m^3/s

COEFFICIENTS

Column	2.00
Cd lower	0.42
Cd Upper	0.47
Cd	0.44
Efficacy	2.59

RATING CURVE

HEAD	**H/P**	**Clower**	**Cupper**	**Cd**	**Q**	**RES**
1.10	0.44	0.42	0.47	0.44	89.3	79.60
0.99	0.40	0.44	0.49	0.46	80.5	79.49
0.88	0.35	0.47	0.51	0.49	71.1	79.38
0.77	0.31	0.50	0.53	0.51	61.0	79.27
0.66	0.26	0.52	0.55	0.53	50.4	79.16
0.55	0.22	0.54	0.56	0.55	39.5	79.05
0.44	0.18	0.55	0.57	0.56	28.8	78.94
0.33	0.13	0.56	0.56	0.56	18.8	78.83
0.22	0.09	0.55	0.55	0.55	10.1	78.72
0.11	0.04	0.53	0.53	0.53	3.4	78.61
0.00	0.00	0.49	0.49	0.49	0.0	78.50



Discharge Coefficient Table Tullis et al. (1995)

Angle wall makes with centerline α

	6	**8**	**12**	**15**	**18**	**25**	**35**	**90**
A0	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49
A1	-0.24	1.08	1.06	1.00	1.32	1.51	1.69	1.46
A2	-1.20	-5.27	-4.43	-3.57	-4.13	-3.83	-4.05	-2.56
A3	2.17	6.79	5.18	3.82	4.24	3.40	3.62	1.44
A4	-1.03	-2.83	-1.97	-1.38	-1.50	-1.05	-1.10	

Hydraulic Design and Analysis of Labyrinth Weirs. I: Discharge Relationships

B. M. Crookston, A.M.ASCE[1]; and B. P. Tullis, M.ASCE[2]

Abstract: A method is presented for the hydraulic design and analysis of labyrinth weirs based upon the experimental results of physical modeling. Discharge coefficient data for labyrinth weirs with quarter-round and half-round crest shapes are presented for sidewall angles ranging from 6 to 35°. Cycle efficiency is also introduced as a design aid, which compares the hydraulic performance of different cycle geometries. Geometric parameters that affect flow performance are discussed. The predictive accuracy of the design method is evaluated through comparisons to previously published labyrinth weir head-discharge data. The companion paper examines nappe behaviors that affect flow performance and presents hydraulic design considerations specific to nappe characteristics. DOI: 10.1061/(ASCE)IR.1943-4774.0000558. © 2013 American Society of Civil Engineers.

CE Database subject headings: Weirs; Water discharge; Coefficients; Hydraulics; Design; Irrigation.

Author keywords: Labyrinth weir design; Discharge coefficients; Weir crest shape; Cycle efficiency; Local submergence.

Introduction

A labyrinth weir is a linear weir that is folded in plan-view to increase the crest length for a given channel or spillway width (Fig. 1). There are an infinite number of possible labyrinth weir configurations and design variations; however, labyrinth cycles are typically placed in a linear fashion (i.e., upstream apexes align at a common channel cross section; Fig. 1), have a sidewall angle (α) less than 30°, and are oriented towards the approaching flow.

A labyrinth weir is able to pass large discharges at relatively low heads compared to traditional linear weir structures of equal width. As a result of their hydraulic performance and geometric versatility, labyrinth weirs have been placed in streams, canals, rivers, ponds, and reservoirs as headwater control structures, energy dissipaters, flow aerators, and spillways. Labyrinth weirs are well suited for spillway rehabilitation where aging infrastructure, dam safety concerns, freeboard limitations, and revised and larger probable maximum flows have required increased spillway capacity. Recently constructed examples are: Lake Brazos spillway in Texas (Vasquez et al. 2007) and Lake Townsend spillway in Greensboro, North Carolina (Tullis and Crookston 2008).

Flow Characteristics

The geometry of a labyrinth weir produces complex three-dimensional flow patterns. At very low heads, it behaves similar to a linear weir ($\alpha = 90°$, oriented normally to the approach flow direction) of equivalent length. However, as the head increases, labyrinth weir discharge efficiency, as quantified by the discharge coefficient value, begins to decline as nappe collision and local submergence regions develop (Crookston and Tullis 2012c).

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Previous Studies

Labyrinth weir head-discharge relationships have been described by various empirical equations. These relationships vary based on different definitions of the discharge coefficient, the characteristic weir length, and the upstream driving head (e.g., the inclusion of the velocity head component $V^2/2g$, described in the following). In the present study, a standard form of the weir equation, Eq. (1), was selected with the centerline length of the crest (L_c) as the characteristic weir length:

$$Q = \frac{2}{3} C_{d(\alpha^\circ)} L_c \sqrt{2g} H_T^{3/2} \qquad (1)$$

where Q = labyrinth weir discharge; $C_{d(\alpha^\circ)}$ = dimensionless discharge coefficient; $L_c = N(2l_c + A + D)$ where N = number of cycles, l_c = centerline length of the sidewall, A = inside apex length, and D = outside apex length; g = acceleration constant of gravity; and H_T = total upstream head (unsubmerged) measured relative to the crest elevation [$H_T = V^2/2g + h$ (V is the average cross-sectional velocity at the gauging location, and h is the piezometric head upstream of the weir)].

Several earlier labyrinth weir studies resulted in published design methods; a selection is presented and discussed. Hay and Taylor (1970) presented parameter guidelines, based upon research by Taylor (1968), for sharp-crested triangular and trapezoidal labyrinth weirs. Discharge rating curves for $h/P < 0.6$ were presented in terms of a labyrinth-to-linear weir discharge ratio (based on a common channel width, W, and h), requiring discharge information for a linear weir ($\alpha = 90°$) of equivalent weir height (P), wall thickness (t_w), and crest shape. The Bureau of Reclamation (USBR) conducted model studies to aid in the design of Ute Dam (Houston 1982). Discrepancies between their experimental results and the recommendations by Hay and Taylor (1970) were attributed to different definitions of upstream head [h, Hay and Taylor (1970);

Table 1. Physical Model Test Program

Model	α[a] (°)	P (mm)	$L_{c\text{-cycle}}$ (mm)	$L_{c\text{-cycle}}/w$ (−)	w/P (−)	N (−)	Crest (−)
1[b]	6	304.8	4,654.6	7.607	2.008	2	HR
2–3	6	304.8	4,654.6	7.607	2.008	2	QR, HR
4–5	8	304.8	3,544.9	5.793	2.008	2	QR, HR
6–7	10	304.8	2,879.1	4.705	2.008	2	QR, HR
8–9	12	304.8	2,435.1	3.980	2.008	2	QR, HR
10–11	15	304.8	1,991.4	3.254	2.008	2	QR, HR
12	15	152.4	1,991.4	3.254	4.015	2	QR
13	15	152.4	995.7	3.254	2.008	4	QR
14	15	304.8	995.7	3.254	1.019	4	QR
15–16	20	304.8	1,548.1	2.530	2.008	2	QR, HR
17–18	35	304.8	983.5	1.607	2.008	2	QR, HR
19–20	90	304.8	1,223.8	1.000	4.015	—	QR, HR

[a]Linear configuration and normal orientation for all models unless noted.
[b]Inverse orientation.

in which at least 10% of the data were repeated to ensure accuracy and to determine measurement repeatability.

Experimental Results

Discharge Rating Curves

Eq. (1) was used to quantify the labyrinth weir head-discharge relationship with L_c representing the characteristic weir length. The term $C_{d(\alpha°)}$ can be influenced by weir geometry (e.g., P, t_w, A, w, α, and crest shape), weir abutments, flow conditions (H_T, approaching flow angle, local submergence, and nappe interference), and nappe aeration conditions (clinging, aerated, partially aerated, and drowned). The two-cycle sectional labyrinth weir models evaluated in this study did not account for the influence of abutments on discharge. Data for $C_{d(\alpha°)}$ are presented in terms of H_T/P for nonvented trapezoidal labyrinth weirs for $6° \le \alpha \le 35°$ in Fig. 2 (quarter-round crest shape) and Fig. 3 (half-round crest shape). The data for $\alpha = 90°$ (linear) weirs are also included for comparison.

The influence of labyrinth weir orientation in the channel [distal apexes connecting to channel sidewalls as upstream apexes (referred to as normal orientation) or as downstream apexes (inverse orientation)] was evaluated by testing the $\alpha = 6°$ labyrinth weirs with both orientations. No measurable variations were observed between $C_{d(\alpha°)}$ data sets (data not presented). Consequently, the data in Figs. 2 and 3 are assumed to be applicable, independent of weir orientation. Crookston and Tullis (2012b) present additional design information and discussion regarding labyrinth weirs in reservoir applications and abutment effects on discharge.

For convenience, the labyrinth weir $C_{d(\alpha°)}$ data in Figs. 2 and 3 were curve-fit per Eq. (2), and the corresponding coefficients are presented in Tables 2 and 3. Eq. (3) was used for $\alpha = 90°$ data and the corresponding coefficients are presented in the aforementioned tables. The curves have been validated for $0.05 \le H_T/P < 0.8$–0.9; however, due to the well-behaved nature of the data and Eq. (2), the $C_{d(\alpha°)}$ curves have been extrapolated to $H_T/P = 1.0$. Eqs. (2) and (3) were selected over polynomial relationships because of their improved data representation ($R^2 \ge 0.99$) and extrapolation performance (they remain well-behaved up to $H_T/P \le 2.0$). Crookston et al. (2012) evaluated Eq. (2) for $H_T/P < 2$ via physical and numerical modeling and found that Eq. 2 may be used as a good first order approximation. When the Tullis et al. (1995) polynomial $C_{d(\alpha°)}$ relationships are extrapolated, they incorrectly compute $C_{d(\alpha°)}$ (even producing negative values) beyond the upper limit of their experimental data (experimental data limited to $H_T/P < 0.9$). For labyrinth weirs:

$$C_{d(\alpha°)} = a\left(\frac{H_T}{P}\right)^{b\left(\frac{H_T}{P}\right)^{c}} + d \quad (2)$$

For linear weirs:

$$C_{d(90°)} = \frac{1}{a + b\frac{H_T}{P} + \frac{c}{H_T/P}} + d \quad (3)$$

A comparison between the half-round and quarter-round experimental data is presented in Fig. 4 as the ratio of the half-round over

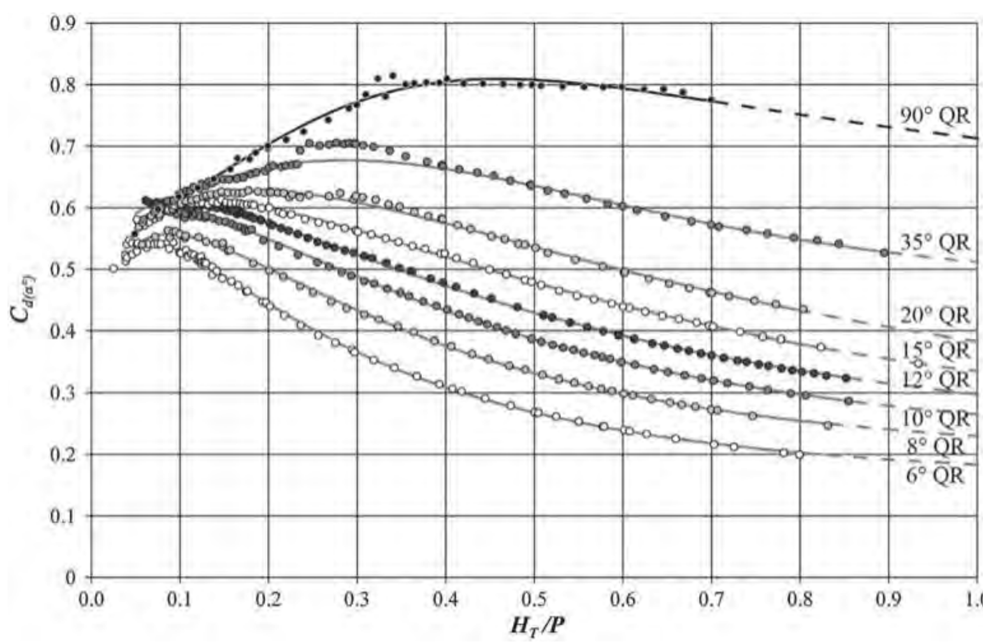


Fig. 2. Values of C_d versus H_T/P for quarter-round trapezoidal labyrinth weirs

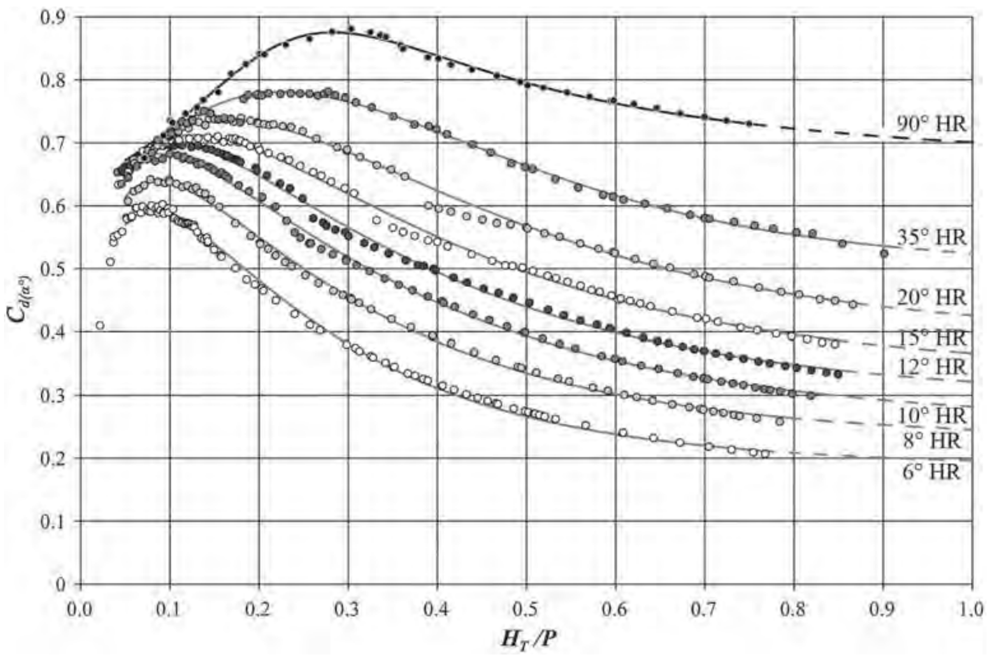


Fig. 3. Values of C_d versus H_T/P for half-round trapezoidal labyrinth weirs

Table 2. Curve-Fit Coefficients for Quarter-Round Labyrinth and Linear Weirs, Validated for $0.05 \le H_T/P < 0.9$

α	a	b	c	d
6°	0.02623	−2.681	0.3669	0.1572
8°	0.03612	−2.576	0.4104	0.1936
10°	0.06151	−2.113	0.4210	0.2030
12°	0.09303	−1.711	0.4278	0.2047
15°	0.10890	−1.723	0.5042	0.2257
20°	0.11130	−1.889	0.5982	0.2719
35°	0.03571	−3.760	0.7996	0.4759
90°	−2.3800	6.476	1.3710	0.5300

Table 3. Curve-Fit Coefficients for Half-Round Labyrinth and Linear Weirs, Validated for $0.05 \le H_T/P < 0.9$

α	a	b	c	d
6°	0.009447	−4.039	0.3955	0.1870
8°	0.017090	−3.497	0.4048	0.2286
10°	0.029900	−2.978	0.4107	0.2520
12°	0.030390	−3.102	0.4393	0.2912
15°	0.031600	−3.270	0.4849	0.3349
20°	0.033610	−3.500	0.5536	0.3923
35°	0.018550	−4.904	0.6697	0.5062
90°	−8.60900	22.650	1.8120	0.6375

the quarter-round $C_{d(\alpha^\circ)}$ values ($C_{d\text{-}HR}/C_{d\text{-}QR}$) versus H_T/P. A crest that is rounded on the downstream face helps the flow stay attached (clinging flow) to the downstream weir wall at smaller H_T/P values, thus increasing flow efficiency and discharge capacity. As the discharge and the corresponding momentum of the flow passing over the weir increase, the nappe becomes aerated and the streamlines will eventually detach from the weir crest, creating a similar nappe profile to the quarter-round crest. Once nappe detachment occurs, the gains in the half-round crest flow efficiency are lost relative to the quarter-round crest. All of the $C_{d\text{-}HR}/C_{d\text{-}QR}$ curves are anticipated to eventually converge to 1.0 with increasing

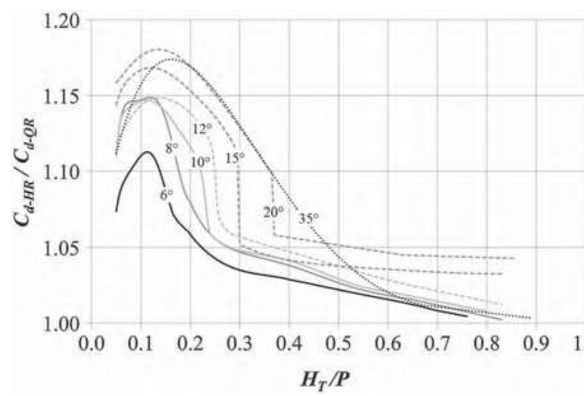


Fig. 4. Comparison of half-round and quarter-round crest shape on hydraulic performance of labyrinth weirs

H_T/P. Improving weir approach flow conditions and using a more efficient crest shape can obtain further gains in efficiency. Brazos Dam (Waco, TX), for example, features an ogee-type crest [modified half-round crest with an upstream radius of $1/3t_w$ and a downstream radius of $2/3t_w$ (Willmore 2004)].

Nappe Behavior and Artificial Aeration

Labyrinth weir flow can produce several nappe behavior phenomena that should not be overlooked: nappe aeration conditions, nappe instability (also termed flow surging), and nappe vibrations. These behaviors, conditions, and remedial actions are discussed in detail in the companion paper (Crookston and Tullis 2013). The artificial aeration, vent pipes, and nappe breakers are also discussed.

Labyrinth Design and Analyses

The recommended procedure for designing a labyrinth weir is presented in Table 4, which includes a design example for illustration

J. Irrig. Drain Eng. 2013.139:363-370.

Attachment C

United States Department of Agriculture

Natural Resources Conservation Service

Conservation Engineering Division

July 2005

Earth Dams and Reservoirs

TR–60

Earth embankments and foundations

Earth embankments constructed of soil and rock are the principal means of impounding water. The earth embankment and its foundation must withstand the anticipated loads without movements leading to failure. Measures must be provided for adequate seepage control.

Height

The design height of an earth embankment must be sufficient to prevent overtopping during passage of either the freeboard hydrograph or stability design hydrograph plus the freeboard required for frost conditions or wave action, whichever is larger. The design height must also meet the requirements for minimum auxiliary spillway depth. The design height of the dam must be increased by the amount needed to compensate for settlement.

Top width

The minimum top width of embankment is shown in table 5-1.

The width may need to be greater than the above minimums to:

- meet state and local standards;
- accommodate embankment zoning;
- provide roadway access and traffic safety; and
- provide structural stability.

An increase in top width is a major design feature in preventing breaching after embankment slumping caused by earthquake ground motion.

When the embankment top is used as a public roadway, the minimum width shall be 16 feet for one-way and 26 feet for two-way traffic. Guardrails or other safety measures shall be used and must meet the requirements of the responsible road authority.

Embankment slope stability

Analyze the stability of embankment slopes using generally accepted methods based on sound engineering principles. Document all analyses including assumptions regarding shear strength parameters for each zone of the embankment and each soil type or horizon in the foundation. Documentation should include methods used for analyses and a summary of results. Design features necessary to provide required safety factors should be noted.

Table 5-1 Minimum top width of embankment

Total height of embankment, H, (ft)	Top Width (ft)		
	All dams	Single purpose floodwater retarding	Multipurpose or other purposes
14 or less	8	N/A	N/A
15–19	10	N/A	N/A
20–24	12	N/A	N/A
25–34	14	N/A	N/A
35–95	N/A	14	(H+35)/5
Over 95	N/A	16	26

Attachment D

LABYRINTH WEIR DESIGN - Low Flow Notch
No Approach Velocity

PROJECT:	East Locust Creek		TIME:	10:17:46	
PROJECT NO.	A11-1513		DATE:	06-Aug-15	
FLOOD CRITERIA:	PMF		BY:	BHL	

USER INPUT

Units	-	English			-		
Max. Res	Zr	924.8	ft		Thickness		
Crest el.	Zc	922.3	ft	Wall Crest	Tw	1.3	ft
Floor el.	Zf	911.8	ft	Wall Bottom	Tb	2.5	ft
Spillway width	Ws	27.5	ft	Slab	Ts	4	ft
Apex Width	D	4	ft		Cutoff Depth		
No. of cycles	n	0.5	-	Sheet Pile	Ds	0	ft
Magnification	L/W	2	-	Conc Wall	Dc	4	ft

CHECK ON RATIOS

Lde/B =	0.07	Ld/B RATIO IS OK
H_o/P =	0.24	Ho/P RATIO IS OK
a =	27.07	Angle IS OK

Note: L_{de}/B must be <= 0.35
Ho/P must be <= 0.9
a must be >= 6 deg

LABYRINTH DIMENSIONS (Per Cycle)

Wall Height	P	10.5	ft
Width	W	55.00	ft
Effective Length	L_e	110.00	ft
Inside Apex	A	2.41	ft
Wall Length	L_1	53.39	ft
Depth	B	48.84	ft
Head max	H	2.50	ft
Wall Angle	a	27.07	deg
Interference Length	L_{de}	3.74	ft

CREST LAYOUT
(One Cycle)

X	Y
0.00	0.00
0.00	1.30
1.20	1.30
25.50	48.84
29.50	48.84
53.80	1.30
55.00	1.30
55.00	0.00
53.00	0.00
28.70	47.54
26.30	47.54
2.00	0.00
0.00	0.00

DISCHARGE
Qmax 2,235 cfs

COST ESTIMATE

	Unit price $/unit	Units	Cost $
	-------------------	-------------------	-------------------
Weir wall, cy	350	41	14,224
Abutment walls, cy	350	101	35,404
Slab, cy	225	207	46,601
Concrete cutoff, cy	200	47	9,333
Sheet pile, sf	20	0	0
Reinforcement, lb	0.65	55,381	35,998

ESTIMATED COST
$141,560

Slab & Weir Wa $ 60,825

			RATING CURVE						Metric
									English
HEAD	H_o/P	C_{lower}	C_{upper}	C_d	Q	RES			
ft	-	-	-	-	cfs	ft			
2.50	0.24	0.6230	0.67	0.65	753	924.80		924.8	
2.25	0.21	0.63	0.67	0.65	642	924.55		924.55	
2.00	0.19	0.63	0.66	0.64	536	924.30		924.3	
1.75	0.17	0.63	0.65	0.64	436	924.05		924.05	
1.50	0.14	0.62	0.64	0.63	342	923.80		923.8	
1.25	0.11904762	0.61	0.63	0.62	256	923.55		923.55	
1.00	0.10	0.60	0.61	0.61	179	923.30		923.3	
0.75	0.07	0.58	0.59	0.59	112	923.05		923.05	
0.50	0.05	0.55	0.57	0.56	59	922.80		922.8	
0.25	0.024	0.51	0.55	0.52737	19	922.55		922.55	
0.00	0.00	0.00	0.00	0.00	0	922.30		922.3	

Discharge Coefficient Table Tullis et al. (1995)

Angle wall makes with centerline a

	6	8	10	12	15	20	35	90
	-------	-------	-------	-------	-------	-------	-------	-------
a	0.02623	0.03612	0.06151	0.09303	0.1089	0.1113	0.03571	-2.38
b	-2.681	-2.576	-2.113	-1.711	-1.723	-1.889	-3.76	6.476
c	0.3669	0.4104	0.421	0.4278	0.5042	0.5982	0.7996	1.371
d	0.1572	0.1936	0.203	0.2047	0.2257	0.2719	0.4759	0.53

COEFFICIENTS	
Column	6.00
Cd lower	0.98
Cd Upper	2.98
Cd	1.92
Efficacy	5.40

50.0
4
1
2

LABYRINTH WEIR DESIGN - Main Spillway
No Approach Velocity

PROJECT:	East Locust Creek			**TIME:**	10:18:52
PROJECT NO.	A11-1513			**DATE:**	06-Aug-15
FLOOD CRITERIA:	PMF			**BY:**	BHL

USER INPUT

Units	**-**	English			**-**		
Max. Res	**Zr**	936.0	**ft**		**Thickness**		
Crest el.	**Zc**	924.8	**ft**	**Wall Crest**	**Tw**	1.3	**ft**
Floor el.	**Zf**	912.8	**ft**	**Wall Bottom**	**Tb**	2.5	**ft**
Spillway width	**Ws**	55.0	**ft**	**Slab**	**Ts**	4	**ft**
Apex Width	**D**	4	**ft**		**Cutoff Depth**		
No. of cycles	**n**	1	**-**	**Sheet Pile**	**Ds**	0	**ft**
Magnification	**L/W**	2	**-**	**Conc Wall**	**Dc**	4	**ft**

CHECK ON RATIOS

Lde/B =	0.31	**Ld/B RATIO IS OK**
H_o/P =	0.93	**WARNING! H/P > 0.9!**
a =	27.07	**Angle IS OK**

Note: L_{de}/B must be <= 0.35
Ho/P must be <= 0.9
a must be >= 6 deg

LABYRINTH DIMENSIONS (Per Cycle)

Wall Height	**P**	12	**ft**
Width	**W**	55.00	**ft**
Effective Length	L_e	110.00	**ft**
Inside Apex	**A**	2.41	**ft**
Wall Length	L_1	53.39	**ft**
Depth	**B**	48.84	**ft**
Head max	**H**	11.20	**ft**
Wall Angle	**a**	27.07	**deg**
Interference Length	L_{de}	16.77	**ft**

CREST LAYOUT
(One Cycle)

X	Y
0.00	0.00
0.00	1.30
1.20	1.30
25.50	48.84
29.50	48.84
53.80	1.30
55.00	1.30
55.00	0.00
53.00	0.00
28.70	47.54
26.30	47.54
2.00	0.00
0.00	0.00

DISCHARGE

Qmax 9,993 **cfs**

COST ESTIMATE

	Unit price $/unit	Units	Cost $
	-------------------	-------------------	-------------------
Weir wall, cy	350	93	32,511
Abutment walls, cy	350	204	71,570
Slab, cy	225	414	93,203
Concrete cutoff, cy	200	97	19,378
Sheet pile, sf	20	0	0
Reinforcement, lb	0.65	113,190	73,573

ESTIMATED COST
$290,235

Slab & Weir Wa $ 125,714

				RATING CURVE						Metric
										English
	HEAD	H_o/P	C_{lower}	C_{upper}	C_d	Q	RES		Rating Curve with I	
	ft	-	-	-	-	cfs	ft			
	11.20	0.93	0.3980	0.52	0.46	10064	936.00		10816.6	936
	9.80	0.82	0.43	0.54	0.48	8717	934.60		9469.2	934.6
	8.50	0.71	0.46	0.57	0.51	7482	933.30		8234.83	933.3
	7.10	0.59	0.50	0.61	0.55	6135	931.90		6887.43	931.9
	5.70	0.48	0.55	0.64	0.59	4739	930.50		5491.15	930.5
	4.40	0.36666667	0.59	0.67	0.63	3399	929.20		4151.88	929.2
	3.00	0.25	0.62	0.68	0.65	1977	927.80		2729.7	927.8
	1.60	0.13	0.62	0.64	0.63	749	926.40		1501.09	926.4
	0.20	0.02	0.49	0.54	0.51	27	925.00		779.542	925
	0.10	0.008	0.46	0.53	0.49152	9	924.90		761.689	924.9
	0.00	0.00	0.00	0.00	0.00	0	924.80		752.542	924.8

Discharge Coefficient Table Tullis et al. (1995)

Angle wall makes with centerline a

	6	8	10	12	15	20	35	90
	-------	-------	-------	-------	-------	-------	-------	-------
a	0.02623	0.03612	0.06151	0.09303	0.1089	0.1113	0.03571	-2.38
b	-2.681	-2.576	-2.113	-1.711	-1.723	-1.889	-3.76	6.476
c	0.3669	0.4104	0.421	0.4278	0.5042	0.5982	0.7996	1.371
d	0.1572	0.1936	0.203	0.2047	0.2257	0.2719	0.4759	0.53

COEFFICIENTS	
Column	6.00
Cd lower	0.39
Cd Upper	0.52
Cd	0.45
Efficacy	1.14

50.0
4
1
2

APPENDIX B
SPILLWAY CHUTE AND STILLING BASIN DESIGN REFERENCES AND CALCULATIONS

Attachment A - Design of Small Dams

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN OF SMALL DAMS

A WATER RESOURCES TECHNICAL PUBLICATION

Table 9-3.—Side channel spillway computations. Using eq(15) for design example in section 9.17(b): given Q = 2,000 ft³/s, bottom width = 10 feet, side slopes = ½:1, and bottom slope = 1 foot in 100 feet.

(1) Station	(2) Δx	(3) Elevation bottom	(4) Trial Δy	(5) Water surface elevation	(6) d	(7) A	(8) Q	(9) v	(10) Q_1+Q_2	(11) $\frac{Q_1}{g(Q_1+Q_2)}$	(12) v_1+v_2	(13) v_2-v_1	(14) Q_2-Q_1	(15) $\frac{Q_2-Q_1}{Q_1}$	(16) $\frac{v_2(Q_2-Q_1)}{Q_1}$	(17) (13)+(16)	(18) Δy = (11) ×(12)×(17)	(19) Remarks
1+00	-	100.0	-	116.34	16.34	297	2,000	6.73	-	-	-	-	-	-	-	-	-	-
0+75	25	100.25	1.00	117.34	17.09	317	1,500	4.73	3,500	0.01332	11.46	2.00	500	0.333	2.24	4.24	0.64	Too low
			.62	116.96	16.71	307	-	4.89	-	-	11.62	1.84	-	-	-	4.08	.63	OK
0+50	25	100.50	.50	117.46	16.96	313	1,000	3.19	2,500	.01244	8.08	1.70	500	.50	2.44	4.14	.42	Too low
			.42	117.38	16.88	311	-	3.22	-	-	8.11	1.67	-	-	-	4.11	.41	OK
0+25	25	100.75	.30	117.68	16.93	313	500	1.60	1,500	.01036	4.82	1.62	500	1.00	3.22	4.84	.24	Too low
			.24	117.62	16.87	311	-	1.61	-	-	4.83	1.61	-	-	-	4.83	.24	OK
0+00	15	100.90	.10	117.72	16.82	310	200	.64	700	.00888	2.25	.97	300	1.50	2.41	3.38	.07	Too low
			.07	117.69	16.79	309	-	.65	-	-	2.26	.96	-	-	-	3.37	.07	OK

reservoir water level. To obtain the assumed crest coefficient value of 3.6, excessive submergence of the overflow must be avoided. If it is assumed that a maximum of two-thirds submergence at the upstream end of the channel can be tolerated, the maximum water surface level in the channel will be ⅔H_o above the crest, or elevation 1002.0. Then at station 0+10, the channel datum water surface level elevation 117.7 will become elevation 1002.0, placing the channel floor level for station 0+00 at approximately elevation 985.3, and for station 1+00 at approximately elevation 984.3.

The design of the side channel control structure would be completed by designing the uncontrolled ogee crest by the methods shown in section 9.13, to obtain the crest coefficient value of 3.6 that was assumed.

Variations in the design can be made by assuming different bottom widths, different channel slopes, and varying control sections. A proper and economical design can usually be achieved after comparing several alternatives.

D. HYDRAULICS OF FREE-FLOW DISCHARGE CHANNELS

9.18. ***General.***—Discharge generally passes through the critical stage in the spillway control structure and enters the discharge channel as supercritical or shooting flow. To avoid a hydraulic jump below the control, the flow must remain at the supercritical stage throughout the length of the channel. The flow in the channel may be uniform or it may be accelerated or decelerated, depending on the slopes and dimensions of the channel and on the total drop. Where it is desired to minimize the grade to reduce excavation at the upstream end of a channel, the flow might be uniform or decelerating, followed by accelerating flow in the steep drop leading to the downstream river level. Flow at any point along the channel will depend upon the specific energy, $d+h_v$, available at that point. This energy will equal the total drop from the reservoir water level to the floor of the channel at the point under consideration, less the head losses accumulated to that point. The velocities and depths of flow along the channel can be fixed by selecting the grade and the cross-sectional dimensions of the channel.

The velocities and depths of free surface flow in a channel, whether it be an open channel, a conduit, or a tunnel, conform to the principle of the conservation of energy as expressed by Bernoulli's theorem, which states "the absolute energy of flow at any cross section is equal to the absolute energy at a downstream section plus intervening losses of energy." As applied to figure 9-35 this relationship can be expressed as follows:

$$\Delta Z + d_1 + h_{v_2} = d_2 + h_{v_2} + \Delta h_L \qquad (17)$$

When the channel grades are not too steep, for practical purposes the normal depth, d_n, can be considered equal to the vertical depth d. The term Δh_L includes all losses that occur in the reach of channel, such as friction, turbulence, impact, and transition losses. Because changes in most channels are made gradually, all losses except those from friction

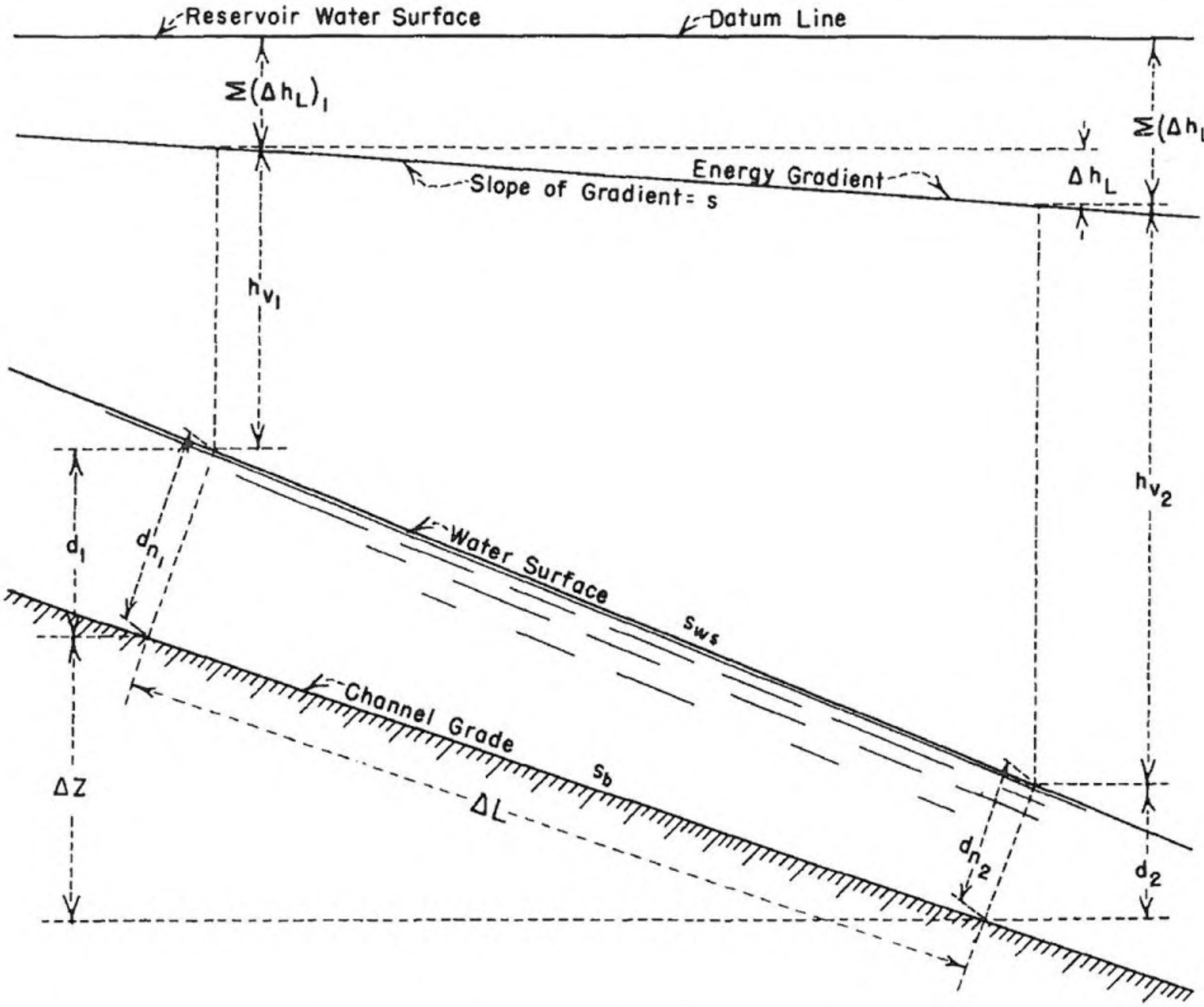


Figure 9-35.—Flow in open channels. 288-D-2421.

can ordinarily be neglected. The friction loss can then be expressed as:

$$\Delta h_L = s\Delta L \tag{18}$$

where s is the average friction slope expressed by either the Chezy or the Manning formula. For the reach ΔL, the head loss can be expressed as:

$$\Delta h_L = \left(\frac{s_1 + s_2}{2}\right) \Delta L$$

From the Manning formula (eq. (30), app. B), $s = (vn/1.486r^{2/3})^2$.

The roughness coefficient, n, will depend on the nature of the channel surface. For conservative design the frictional loss should be maximized when evaluating depths of flow and minimized when evaluating the energy content of the flow. For determining depths of flow in a concrete-lined channel, an n of about 0.014 should be assumed. For determining specific energies of flow needed to design the dissipating device, an n of about 0.008 should be assumed.

Where only rough approximations of depths and velocities of flow in a discharge channel are desired, the total head loss $\Sigma\Delta h_L$ to any point along the channel might be expressed in terms of the velocity head. Thus, at any section the relationship can be stated: reservoir water surface elevation minus floor grade elevation = $d + h_v + Kh_v$. For spillways with small drops, K can be assumed as approximately 0.2 for determining depths of flow and 0.1 or less for evaluating the energy of flow. Rough approximations of losses can also be obtained from figure B-5.

9.19. *Open Channels.*—(a) *Profile.*—The profile of an open channel is usually selected to conform to topographic and geologic site conditions. It is generally defined as straight reaches connected by vertical curves. Sharp convex and concave vertical curves would develop unsatisfactory flows in the channel and should be avoided. Convex curves should be flat enough to maintain positive pressures and thus preclude the tendency for the flow to separate from the floor. Concave curves should have a sufficiently long radius of curvature to minimize the dynamic forces on the floor brought about by the centrifugal force from a change in the direction of flow.

To avoid the tendency for the water to spring away from the floor and, thereby, reduce the surface contact pressure, the floor shape for convex curvature should be made slightly flatter than the trajectory of a free-discharging jet issuing under a head equal to the specific energy of flow as it enters the curve. The curvature should approximate a shape defined by the equation:

$$-y = x \tan\theta + \frac{x^2}{K[4(d + h_v)\cos^2\theta]} \qquad (19)$$

where θ is the slope angle of the floor upstream from the curve. Except for the factor K, the equation is that of a free-discharging trajectory issuing from an inclined orifice. To ensure positive pressure along the entire contact surface of the curve, K should be equal to or greater than 1.5.

For the concave curvature, the pressure exerted upon the floor surface by the centrifugal force of the flow varies directly with the energy of the flow and inversely with the radius of curvature. An approximate relationship of these criteria can be expressed in the equations:

$$R = \frac{2qv}{p} \text{ and } R = \frac{2dv^2}{p} \qquad (20)$$

where:

- R = the minimum radius of curvature, in feet,
- q = the discharge, in cubic feet per second per foot of width,
- v = the velocity, in feet per second,
- d = the depth of flow, in feet, and
- p = the normal dynamic pressure exerted on the floor, in pounds per square foot.

An assumed value of p = 1,000 will normally produce an acceptable radius; however, in no case should the radius be less than $10d$. For the reverse curve at the lower end of the ogee crest, radii of not less than $5d$ have been found acceptable.

(b) *Convergence and Divergence.*—The best hydraulic performance in a discharge channel is obtained when the confining sidewalls are parallel and the distribution of flow across the channel is maintained uniform. However, economy may dictate a channel section narrower or wider than either the crest or the terminal structure, thereby requiring converging or diverging transitions to fit the various components together. Sidewall convergence must be made gradual to avoid cross waves, wave runup on the walls, and uneven distribution of flow across the channel. Similarly, the rate of divergence of the sidewalls must be limited or else the flow will not spread to occupy the entire width of the channel uniformly. This will result in undesirable flow conditions at the terminal structure.

The inertial and gravitational forces of streamlined kinetic flow in a channel can be expressed by the Froude number parameter, $v/(gd)^{1/2}$. Variations from streamlined flow caused by outside interferences that cause an expansion or a contraction of the flow can also be related to this parameter. Experiments have shown that an angular variation of the flow boundaries not exceeding that produced by the equation,

$$\tan\alpha = \frac{1}{3F} \qquad (21)$$

will provide an acceptable transition for either a contracting or an expanding channel. In this equation, $F = v/(gd)^{1/2}$, and α is the angular variation of the sidewall with respect to the channel centerline; v and d are the velocity and depth at the start of the transition. Figure 9-36 is a nomograph from which the tangent of the flare angle or the flare angle in degrees may be obtained for known values of depth and velocity of flow.

(c) *Channel Freeboard.*—In a channel conducting flow at the supercritical stage, the surface roughness, wave action, air bulking, splash, and spray are related to the velocity and energy content of the flow. Expressed in terms of v and d, the energy per foot of width $qh_v = v^3d/2g$. Therefore the relationship of velocity and depth to the flow energy also can be expressed in terms of v and $d^{1/3}$. An empirical expression based on this relationship that gives a reasonable indication of desirable freeboard values is:

$$\text{Freeboard (in feet)} = 2.0 + 0.025v\sqrt[3]{d} \qquad (22)$$

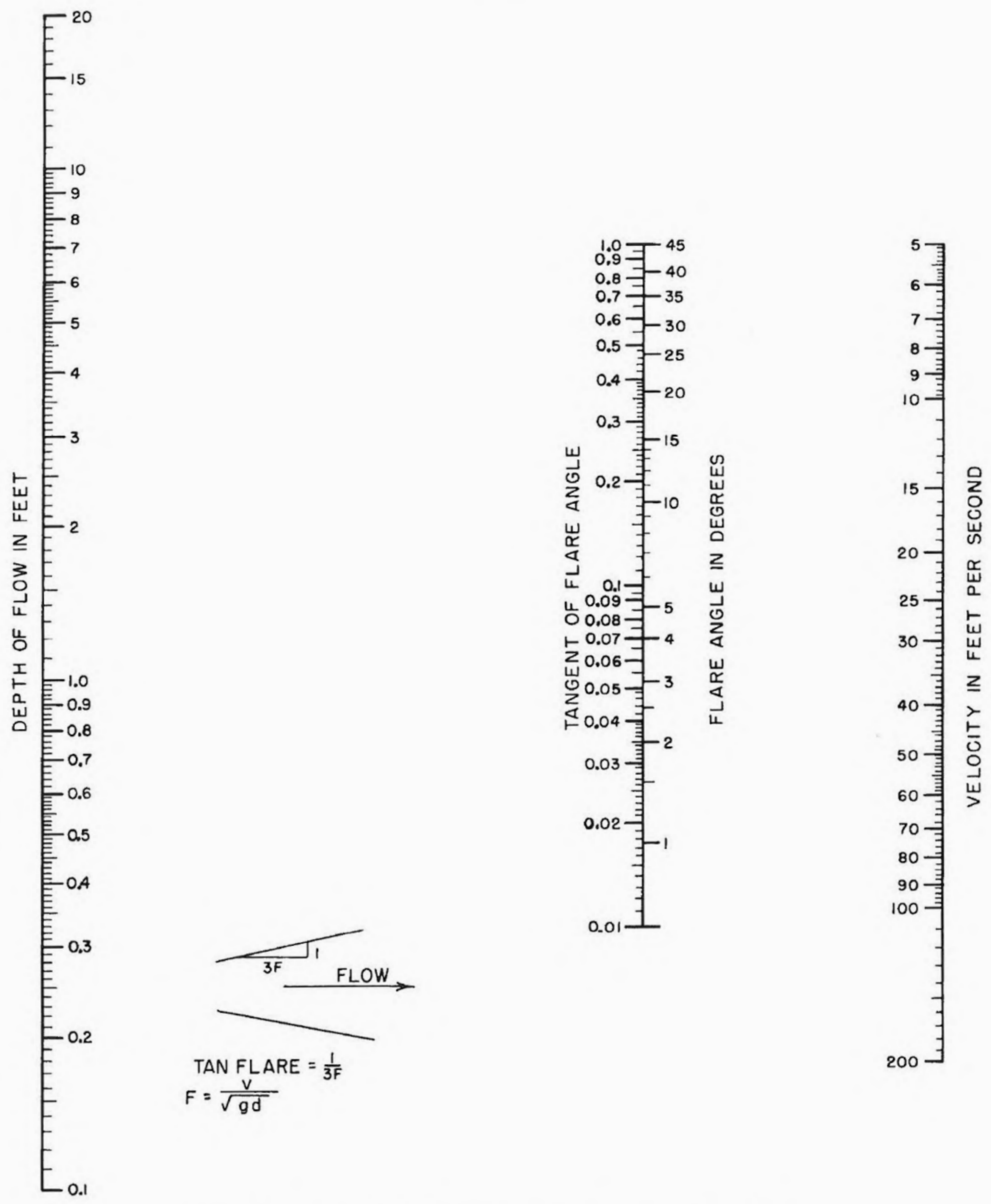


Figure 9-36.—Flare angle for divergent or convergent channels. 288–D–2422.

E. HYDRAULICS OF TERMINAL STRUCTURES

9.20. *Deflector Buckets.*—Where the spillway discharge may be safely delivered directly to the river without providing a dissipating or stilling device, the jet is often projected beyond the structure by a deflector bucket or lip. Flow from these deflectors leaves the structure as a free-discharging upturned jet and falls into the stream channel some distance from the end of the spillway. The path the jet assumes depends on the energy of flow available at the lip and the angle at which the jet leaves the bucket.

With the origin of the coordinates taken at the end of the lip, the path of the trajectory is given by the equation:

$$y = x \tan \theta - \frac{x^2}{K[4(d + h_v) \cos^2 \theta]} \tag{23}$$

where:

θ = angle of the edge of the lip with the horizontal, and

K = a factor, equal to 1, for the theoretical jet.

To compensate for loss of energy and the velocity reduction caused by air resistance, internal turbulences, and disintegration of the jet, $K = 0.9$ should be assumed.

The horizontal range of the jet at the level of the lip is obtained by making $y = 0$ in equation (23). Then, $x = 4K(d+h_v)\tan\theta\cos^2\theta = 2K(d+h_v)\sin 2\theta$. The maximum value of x will be $2K(d + h_v)$ when $\theta = 45°$. However, the angle of the lip is influenced by the bucket radius and the height of the lip above the bucket invert; ordinarily the exit angle should not be more than 30°.

The bucket radius should be made long enough to maintain concentric flow as the water moves around the curve. The rate of curvature must be limited, similar to that of a vertical curve in a discharge channel (sec. 9.19), so that the floor pressures will not alter the streamline distribution of the flow. The minimum radius of curvature, R, can be determined from equation (20), except that values of $p \leq 1{,}000$ lb/ft^2 will produce values of the radius that have proved satisfactory in practice. However, the radius should not be less than $5d$, five times the depth of water. Structurally, the cantilever bucket must be strong enough to withstand this normal dynamic force in addition to the other applied forces.

9.21. *Hydraulic-Jump Basins.*—(a) *General.*—Where the energy of flow in a spillway must be dissipated before the discharge is returned to the downstream river channel, the hydraulic-jump stilling basin is an effective device for reducing the exit velocity to a tranquil state. The jump that will occur in such a stilling basin has distinctive characteristics and assumes a definite form, depending on the relation between the energy of flow that must be dissipated and the depth of the flow.

A comprehensive series of tests have been performed by the Bureau of Reclamation [15] to determine the properties of the hydraulic jump. The jump form and the flow characteristics can be related to the kinetic flow factor, v^2/gd, of the discharge entering the basin; to the critical depth of flow, d_c ; or to the Froude number parameter, $v/(gd)^{1/2}$. Forms of the hydraulic-jump phenomena for various ranges of the Froude number are illustrated on figure 9-37.

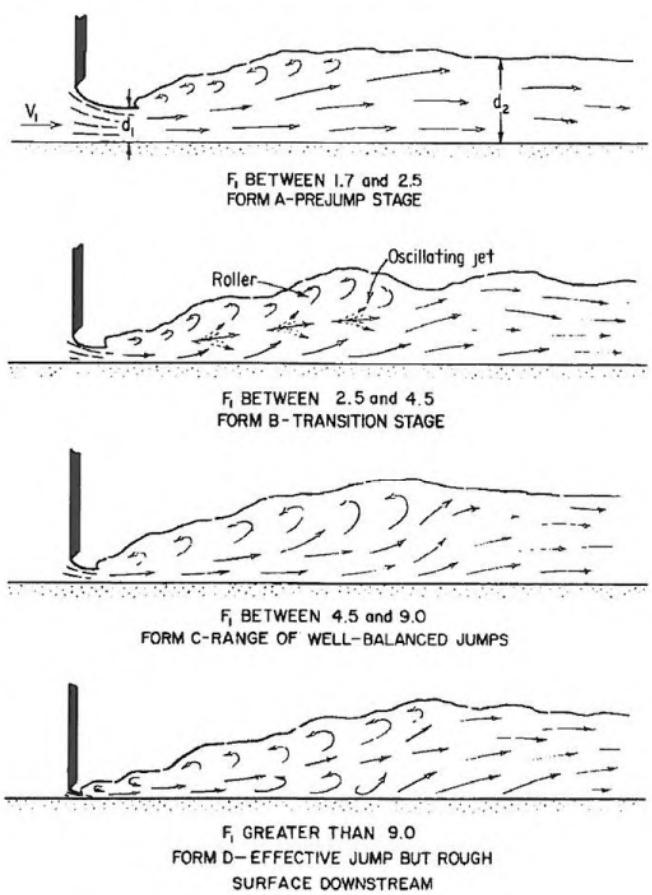


Figure 9-37.—Characteristic forms of hydraulic jump related to the Froude number. 288-D-2423.

When the Froude number of the incoming flow is 1.0, the flow is at critical depth and a hydraulic jump cannot form. For Froude numbers from 1.0 to about 1.7, the incoming flow is only slightly below critical depth, and the change from this low stage to the high stage flow is gradual and manifests itself only by a slightly ruffled water surface. As the Froude number approaches 1.7, a series of small rollers begins to develop on the surface. These become more intense with increasingly higher values of the number. Other than the surface roller phenomena, relatively smooth flows prevail throughout the Froude number range up to about 2.5. Stilling action for the range of Froude numbers from 1.7 to 2.5 is shown as form A on figure 9-37. Forms B, C, and D on figure 9-37 show characteristic forms at hydraulic jumps related to higher Froude numbers.

For Froude numbers between 2.5 and 4.5, an oscillating form of jump occurs. The entering jet intermittently flows near the bottom and then along the surface of the downstream channel. This oscillating flow causes objectionable surface waves that carry far beyond the end of the basin. The action represented through this range of flows is designated as form B on figure 9-37.

For Froude numbers between 4.5 and 9, a stable and well-balanced jump occurs. Turbulence is confined to the main body of the jump, and the water surface downstream is comparatively smooth. As the Froude number increases above 9, the turbulence within the jump and the surface roller becomes increasingly active, resulting in a rough water surface with strong surface waves downstream from the jump. Stilling action for Froude numbers between 4.5 and 9 is designed as form C on figure 9-37, and that above 9 is designated as form D.

Figure 9-38 plots relationships of conjugate depths and velocities for the hydraulic jump in a rectangular channel. The ranges for the various forms of jump described above are also indicated on the figure.

(b) *Basin Design in Relation to Froude Numbers.*—Stilling basin designs suitable to provide stilling action for the various forms of jump are described in the following paragraphs.

(1) *Basins for Froude Numbers Less Than 1.7.*—For a Froude number of 1.7, the conjugate depth, d_2, is about twice the incoming depth, or about 40 percent greater than the critical depth. The exit velocity, v_2, is about one-half the incoming velocity, or 30 percent less than the critical velocity. No special stilling basin is needed to still flows where the Froude number of the incoming flow is less than 1.7, except that the channel lengths beyond the point where the depth starts to change should be not less than about $4d_2$. No baffles or other dissipating devices are needed. These basins, designated type I, are not shown here (see [15]).

(2) *Basins for Froude Numbers Between 1.7 and 2.5.*—Flow phenomena for these basins will be in the form designated as the prejump stage, as shown on figure 9-37. Because such flows are not attended by active turbulence, baffles or sills are not required. The basin should be long enough to contain the flow prism while it is undergoing retardation. Conjugate depths and basin lengths shown on figure B-15 will provide acceptable basins. These basins, designated type I, are not shown here (see [15]).

(3) *Basins for Froude Numbers Between 2.5 and 4.5.*—Flows for these basins are considered to be in the transition flow stage because a true hydraulic jump does not fully develop. Stilling basins that accommodate these flows are the least effective in providing satisfactory dissipation because the attendant wave action ordinarily cannot be controlled by the usual basin devices. Waves generated by the flow phenomena will persist beyond the end of the basin and must often be dampened by means apart from the basin.

Where a stilling device must be provided to dissipate flows for this range of Froude number, the basin shown on figure 9-39(A), which is designated a type IV basin, has proved relatively effective for dissipating the bulk of the energy of flow. However, the wave action propagated by the oscillating flow cannot be entirely dampened. Auxiliary wave dampeners or wave suppressors must sometimes be used to provide smooth surface flow downstream.

Because of the tendency of the jump to sweep out and as an aid in suppressing wave action, the water depths in the basin should be about 10 percent greater than the computed conjugate depth.

Often, the need to design this type of basin can be avoided by selecting stilling basin dimensions that will provide flow conditions that fall outside the range of transition flow. For example, with an 800-ft^3/s capacity spillway where the specific energy at the upstream end of the basin is about 15 feet and the velocity into the basin is about 30 ft/s, the Froude number will be 3.2 for a basin width of 10 feet. The Froude number can be raised to 4.6 by widening the basin to 20 feet. The selection of basin

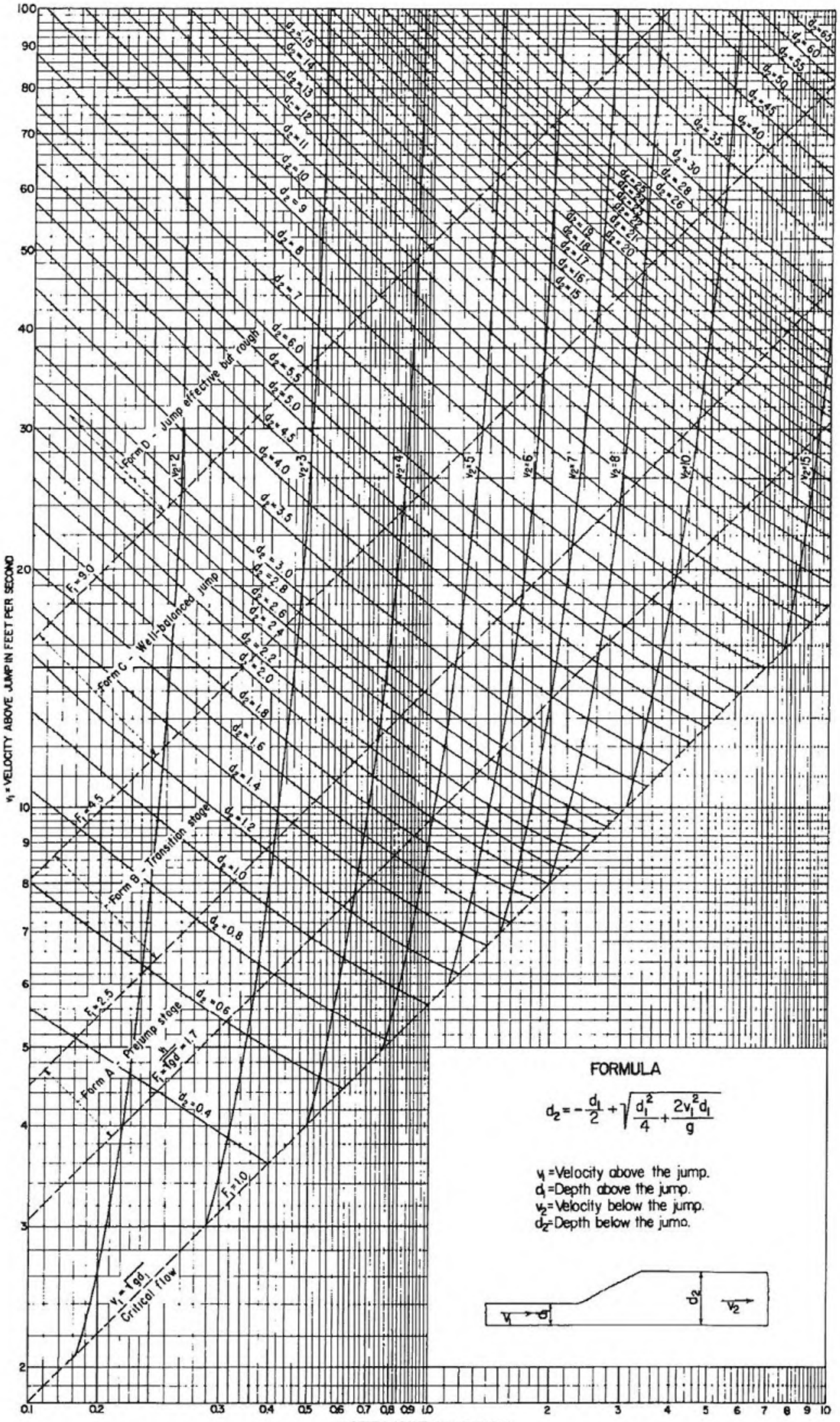


Figure 9-38.—Relations between variables in hydraulic jump for rectangular channel. 288-D-2424.

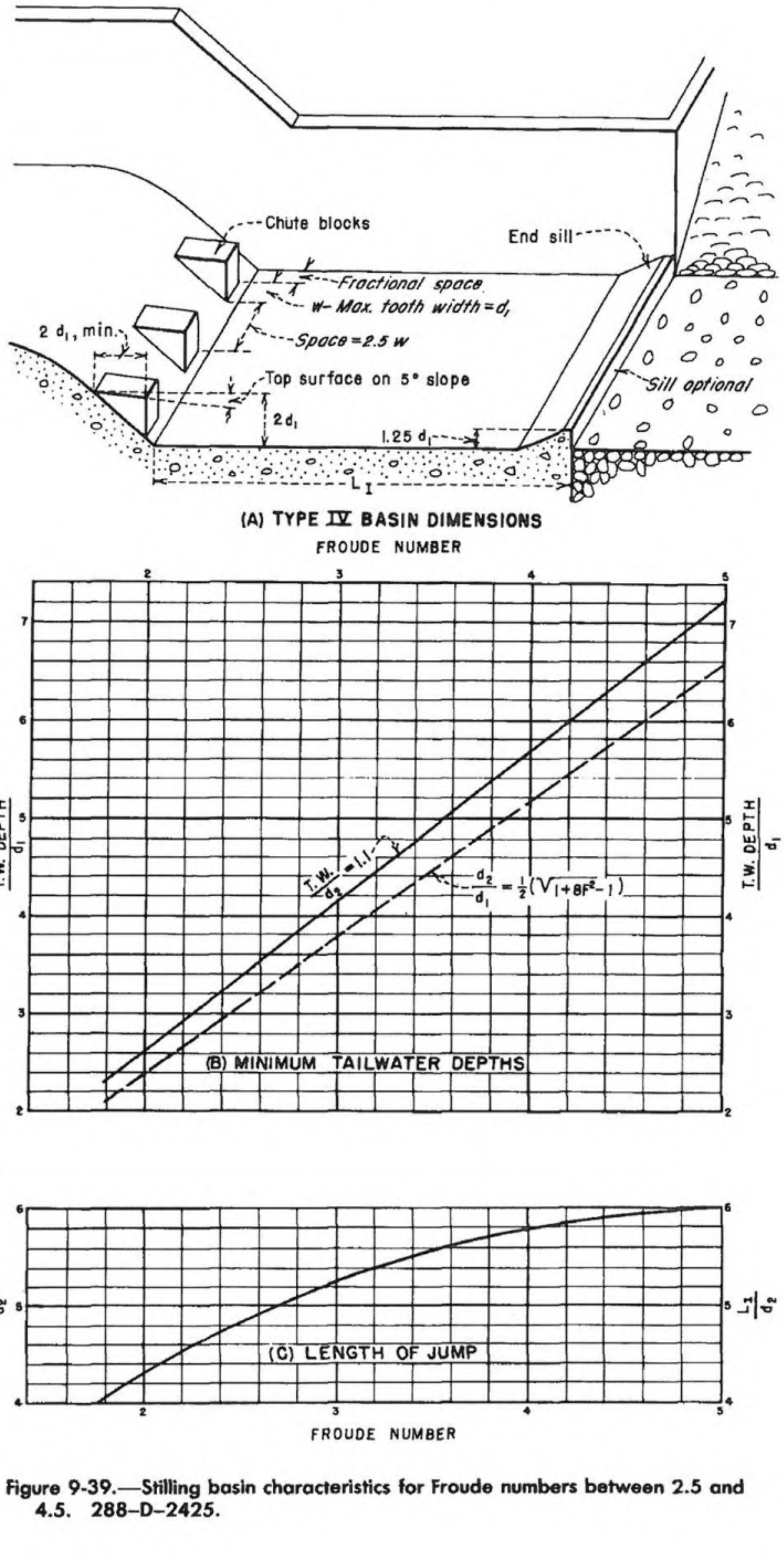


Figure 9-39.—Stilling basin characteristics for Froude numbers between 2.5 and 4.5. 288–D–2425.

width then becomes a matter of economics as well as hydraulic performance.

(4) *Alternative Low Froude Number Stilling Basins.*—Type IV basins are fairly effective at low Froude number flows for small canals and for structures with small unit discharges. However, recent model tests have developed designs quite different from the type IV basin design, even though the type IV basin design was included in the initial tests.

Palmetto Bend Dam stilling basin [22] is an example of a low Froude number structure, modeled in the Bureau of Reclamation Hydraulics Laboratory, whose recommended design is quite different from type IV design. The type IV design has large deflector blocks, similar to but larger than chute blocks, and an optional solid end sill; the Palmetto Bend design has no chute blocks, but has large baffle piers and a dentated end sill.

The foregoing generalized designs have not been suitable for some Bureau applications, and the increased use of low Froude number stilling basins has created a need for additional data on this type of design. A study was initiated to develop generalized criteria for the design of low Froude number hydraulic-jump stilling basins. The criteria and guidelines from previous studies were combined with the results of this study to formulate the design guidelines recommended for low Froude number stilling basins [23]. However, it should be noted that a hydraulic-jump stilling basin is not an efficient energy dissipator at low Froude numbers; that is, the efficiency of a hydraulic-jump basin is less than 50 percent in this Froude number range. Alternative energy dissipators, such as the baffled apron chute or spillway, should be considered for these conditions.

The recommended design has chute blocks, baffle piers, and a dentated end sill. All design data are presented on figure 9-40. The length is rather short, approximately three times d_2 (the conjugate depth after the jump). The size and spacing of the chute blocks and baffle piers are a function of d_1 (incoming depth) and the Froude number. The dentated end sill is proportioned according to d_2 and the Froude number. The end sill is placed at or near the downstream end of the stilling basin. Erosion tests were not included in the development of this basin. Observations of flow patterns near the invert downstream from the basin indicated that no erosion problem should exist. However, if hydraulic model tests are performed to confirm a design based on these criteria, erosion tests should be included. Tests should be made over a full range of discharges to determine whether abrasive materials will move upstream into the basin and to determine the erosion potential downstream from the basin. If the inflow velocity is greater than 50 ft/s, hydraulic model studies should be performed.

(5) *Basins for Froude Numbers Higher Than 4.5.*—For these basins, a true hydraulic jump will form. The elements of the jump will vary according to the Foude number, as shown on figure B-15. The installation of accessory devices such as blocks, baffles, and sills along the floor of the basin produce a stabilizing effect on the jump, which permits shortening the basin and provides a safety factor against sweepout caused by inadequate tailwater depth.

The basin shown on figure 9-41, which is designated a type III basin, can be adopted where incoming velocities do not exceed 60 ft/s. The type III basin uses chute blocks, impact baffle blocks, and an end sill to shorten the jump length and to dissipate the high-velocity flow within the shortened basin length. This basin relies on dissipation of energy by the impact blocks and on the turbulence of the jump phenomena for its effectiveness. Because of the large impact forces to which the baffles are subjected by the impingement of high incoming velocities and because of the possibility of cavitation along the surfaces of the blocks and floor, the use of this basin must be limited to heads where the velocity does not exceed 60 ft/s.

Cognizance must be taken of the added loads placed on the structure floor by the dynamic force brought against the upstream face of the baffle blocks. This dynamic force will approximate that of a jet impinging upon a plane normal to the direction of flow. The force, in pounds, may be expressed by the formula:

$$\text{Force} = 2wA(d_1+h_{v_1}) \tag{24}$$

where:

w = unit weight of water, in pounds per cubic foot,

A = area of the upstream face of the block, in square feet, and

$(d_1+h_{v_1})$ = the specific energy of the flow entering the basin, in feet.

Negative pressure on the back face of the blocks

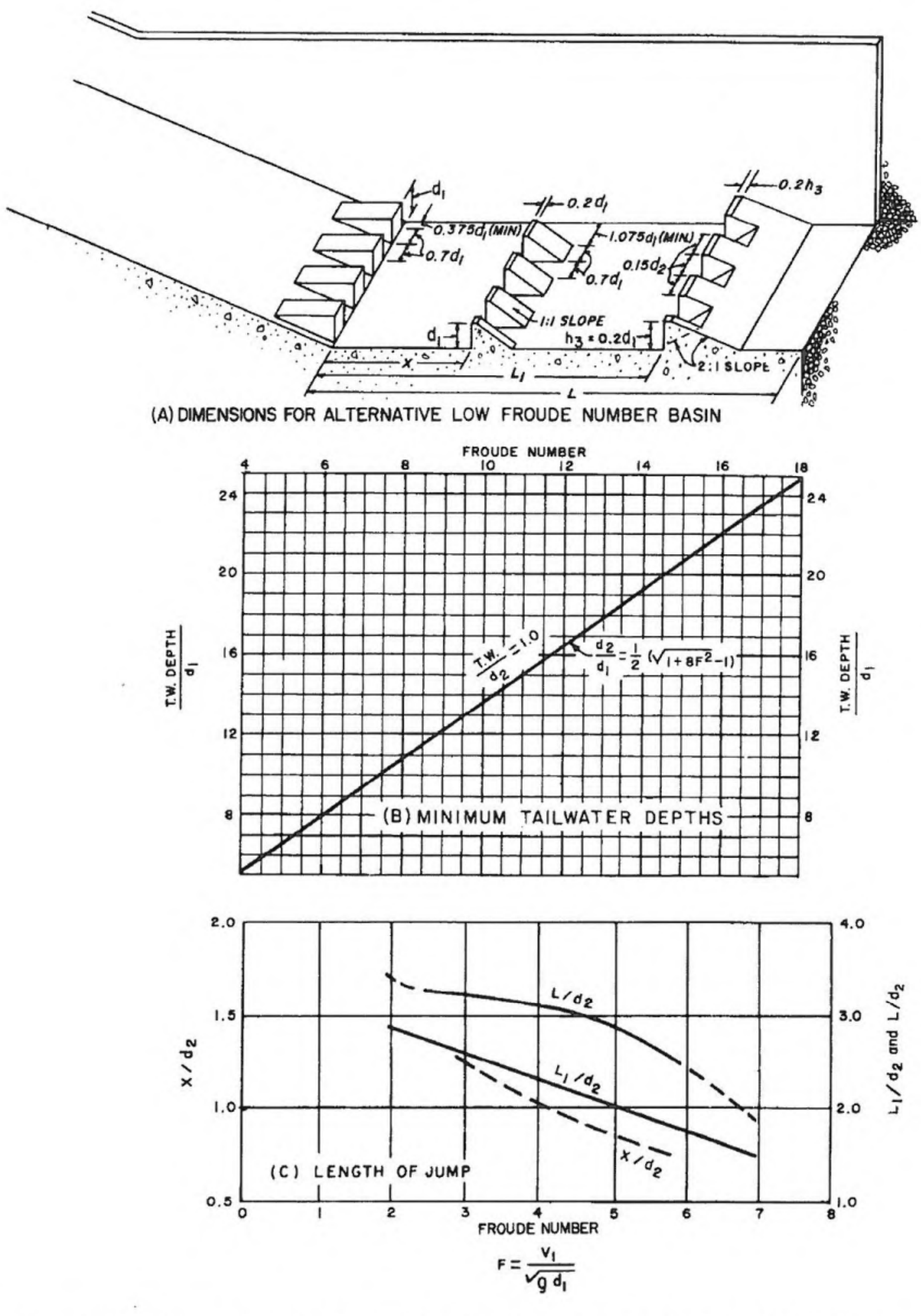


Figure 9-40.—Characteristics for alternative low Froude number stilling basins. 103–D–1876.

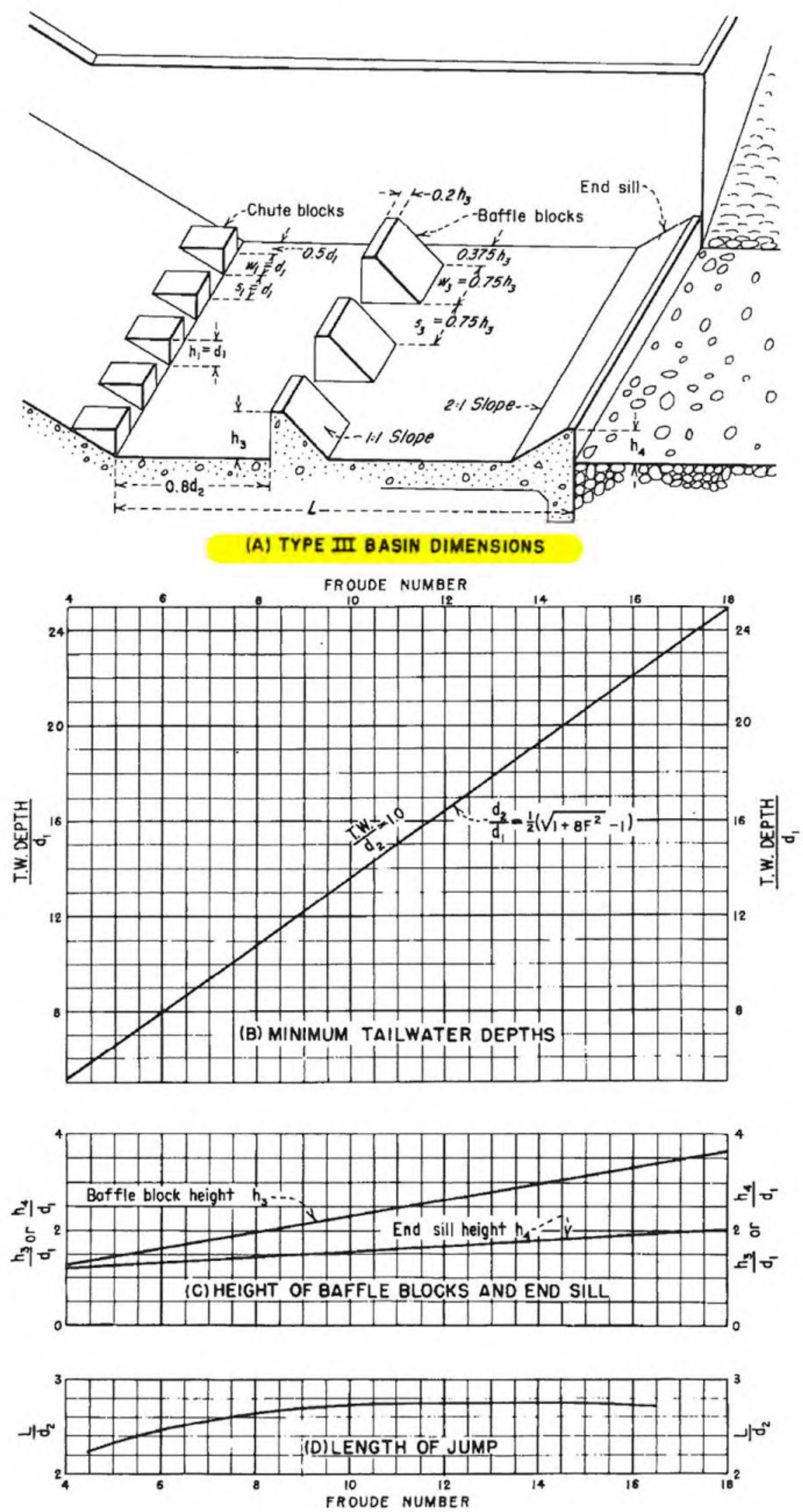


Figure 9-41.—Stilling basin characteristics for Froude numbers above 4.5 where incoming velocity, $V_1 \leq$ 60 ft/s. 288–D–2426.

will further increase the total load. However, because the baffle blocks are placed a distance equal to $0.8d_2$ beyond the start of the jump, there will be some cushioning effect by the time the incoming jet reaches the blocks, and the force will be less than that indicated by the above equation. If the full force computed by equation (24) is used, the negative pressure force may be neglected.

Where incoming velocities exceed 60 ft/s, or where impact baffle blocks are not used, the type II basin (fig. 9-42) may be adopted. Because the dissipation is accomplished primarily by hydraulic-jump action, the basin length will be greater than that indicated for the type III basin. However, the chute blocks and dentated end sill will still effectively reduce the length. Because of the reduced margin of safety against sweepout, the water depth in the basin should be about 5 percent greater than the computed conjugate depth.

(c) *Rectangular Versus Trapezoidal Stilling Basin.*—The use of a trapezoidal stilling basin instead of a rectangular basin may often be proposed where economy favors sloped side lining over vertical wall construction. Model tests have shown, however, that the hydraulic-jump action in a trapezoidal basin is much less complete and less stable than it is in the rectangular basin. In a trapezoidal basin, the water in the triangular areas along the sides of the basin adjacent to the jump does not oppose the incoming high-velocity jet. The jump, which tends to occur vertically, cannot spread sufficiently to occupy the side areas. Consequently, the jump will form only in the central portion of the basin, while areas along the outside will be occupied by upstream-moving flows that ravel off the jump or come from the lower end of the basin. The eddy or horizontal roller action resulting from this phenomenon tends to interfere and interrupt the jump action to the extent that there is incomplete dissipation of the energy and severe scouring can occur beyond the basin. For good hydraulic performance, the sidewalls of a stilling basin should be vertical or as close to vertical as practicable.

(d) *Basin Depths Versus Hydraulic Heads.*—The nomograph on figure 9-43 can help determine approximate basin depths for various basin widths and for various differences between reservoir and tailwater levels. Plots are shown for the condition of no loss of head to the upstream end of the stilling basin, and for 10, 20, and 30 percent loss as scales A, B, C, and D, respectively. The required conjugate depths, d_2, will depend on the specific energy available at the entrance of the basin, as determined by the procedure discussed in section 9.18. Where the specific energy is known, the head loss in the channel upstream can be related to the velocity head, the percentage loss can be determined, and the approximate conjugate depth can be read for the nomograph. Where head losses have not been computed, a quick approximation of the head losses can be obtained from figure B-5. Where only a rough determination of basin depths is needed, the choice of the loss to be applied for various spillway designs may be generalized as follows:

(1) For a design of an overflow spillway where the basin is directly downstream from the crest, or where the chute is not longer than the hydraulic head, consider no loss of head.
(2) For a design of a channel spillway where the channel length is between one and five times the hydraulic head, consider 10 percent loss of head.
(3) For a design of a spillway where the channel length exceeds five times the hydraulic head, consider 20 percent loss of head.

The nomograph on figure 9-43 gives values of the conjugate depth of the hydraulic jump. Tailwater depths for the various types of basin described should be increased as noted earlier in this section.

(e) *Tailwater Considerations.*—Determination of the tailwater rating curve, which gives the stage-discharge relationship of the natural stream below the dam, is discussed in appendix B, part B. Tailwater rating curves for the regime of river below a dam are fixed by the natural conditions along the stream and ordinarily cannot be altered by the spillway design or by the release characteristics. As discussed in section 9.7(d), the retrogression or aggradation of the river below the dam, which will affect the ultimate stage-discharge conditions, must be recognized in selecting the tailwater rating curve to be used for stilling basin design. Usually, river flows that approach the maximum design discharges do not occur, and an estimate of the tailwater rating curve must either be extrapolated from known conditions or computed on a basis of assumed or empirical criteria. Thus, the tailwater rating curve is, at best, only approximate, and safety factors must be included in the design to compensate for variations in tailwater.

For a jump-type stilling basin, downstream water

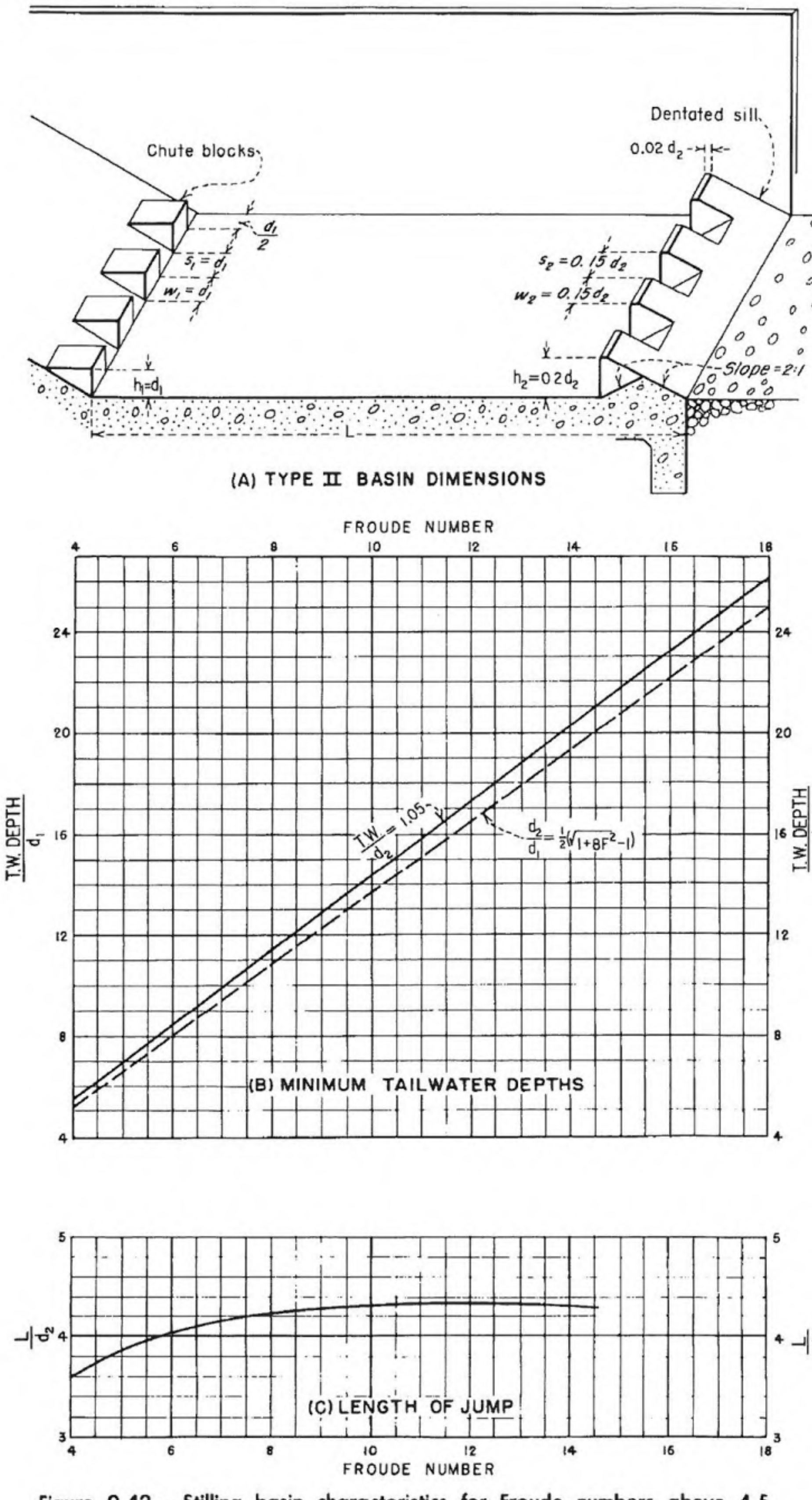


Figure 9-42.—Stilling basin characteristics for Froude numbers above 4.5. 288-D-2427.

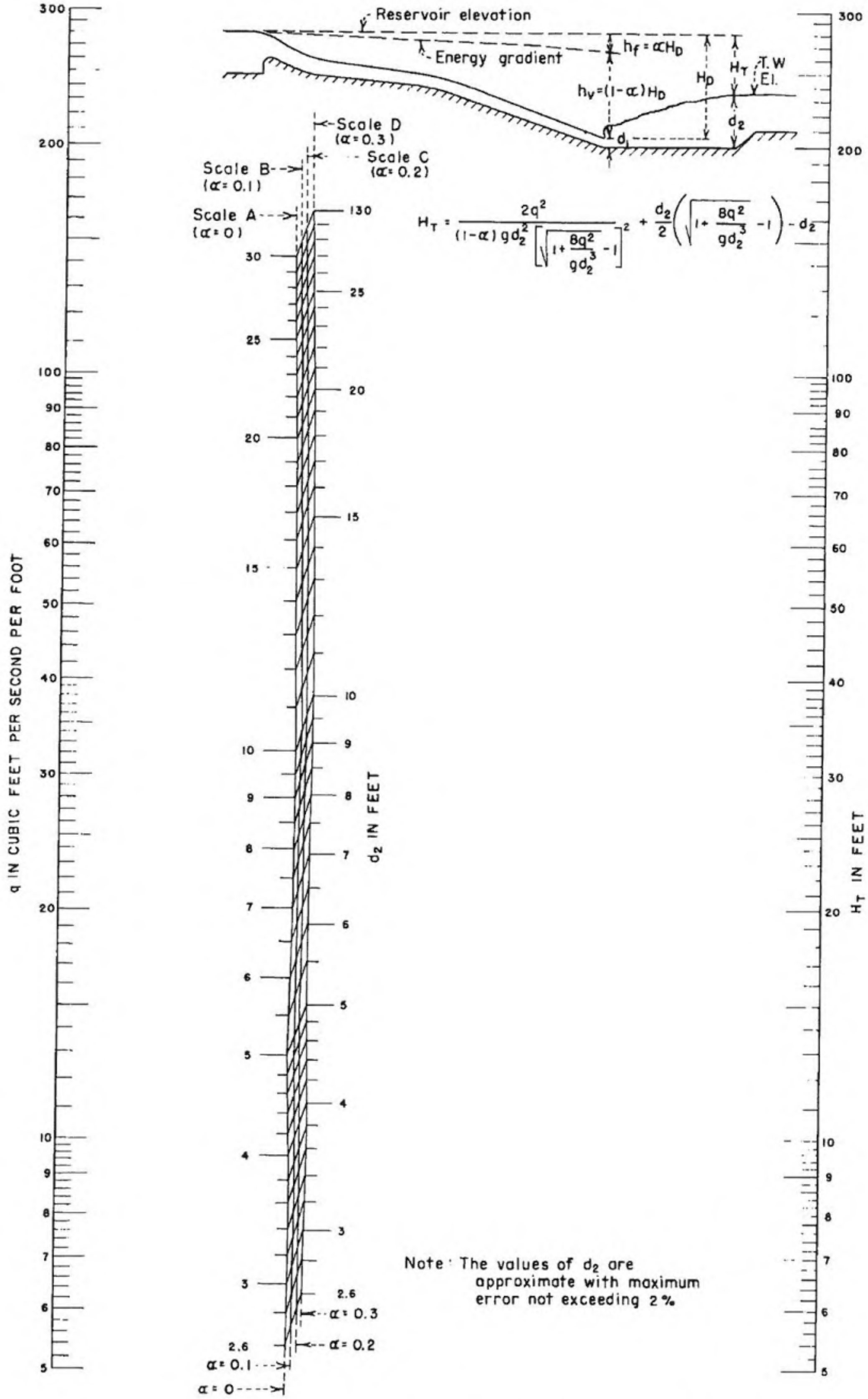


Figure 9-43.—Stilling basin depths versus hydraulic heads for various channel losses. 288-D-2428.

levels for various discharges must conform to the tailwater rating curve. The basin floor level must therefore be selected to provide jump depths that most nearly agree with the tailwater depths. For a given basin design, the tailwater depth for each discharge seldom corresponds to the conjugate depth needed to form a perfect jump. Thus, the relative shapes and relationships of the tailwater curve to the depth curve will determine the required minimum depth to the basin floor. This is shown on figure 9-44(A) where the tailwater rating curve is shown as curve 1, and a conjugate depth versus discharge curve for a basin of certain width is represented by curve 3. Because the basin must be deep enough to provide for full conjugate depth (or some greater depth to provide a safety factor) at the maximum spillway design discharge, the curves will intersect at point D. For lesser discharges the tailwater depth will be greater than the required conjugate depth, thus providing an excess of tailwater, which is conducive to the formation of a "drowned jump." (With the drowned jump condition, instead of achieving good jump-type dissipation by the intermingling of the upstream and downstream flows, the incoming jet plunges to the bottom and carries along the entire length of the basin floor at high velocity.) If the basin floor is higher than indicated by the position of curve 3 on figure 9-44, the depth curve and tailwater rating curve will intersect to the left of point D. This indicates an excess of tailwater for smaller discharges and a deficiency of tailwater for higher discharges.

As an alternative to the selected basin represented by curve 3, a wider basin might be considered for which conjugate depth curve 2 will apply. This design will provide a shallower basin, in which the ideal jump depths will more nearly match the tailwater depths for all discharges. The choice of basin widths, of course, involves consideration of economics, as well as of hydraulic performance.

Where a tailwater rating curve shaped similar to that represented by curve 4 on figure 9-44(B) is encountered, the level of the stilling basin floor must be determined for some discharge other than the maximum design capacity. If the tailwater curve intersects the required water surface elevation at the maximum design capacity, as in figure 9-44(A), there would be insufficient tailwater depth for most smaller discharges. In this case the basin floor elevation is selected so that there will be sufficient tailwater depth for all discharges. For a basin of

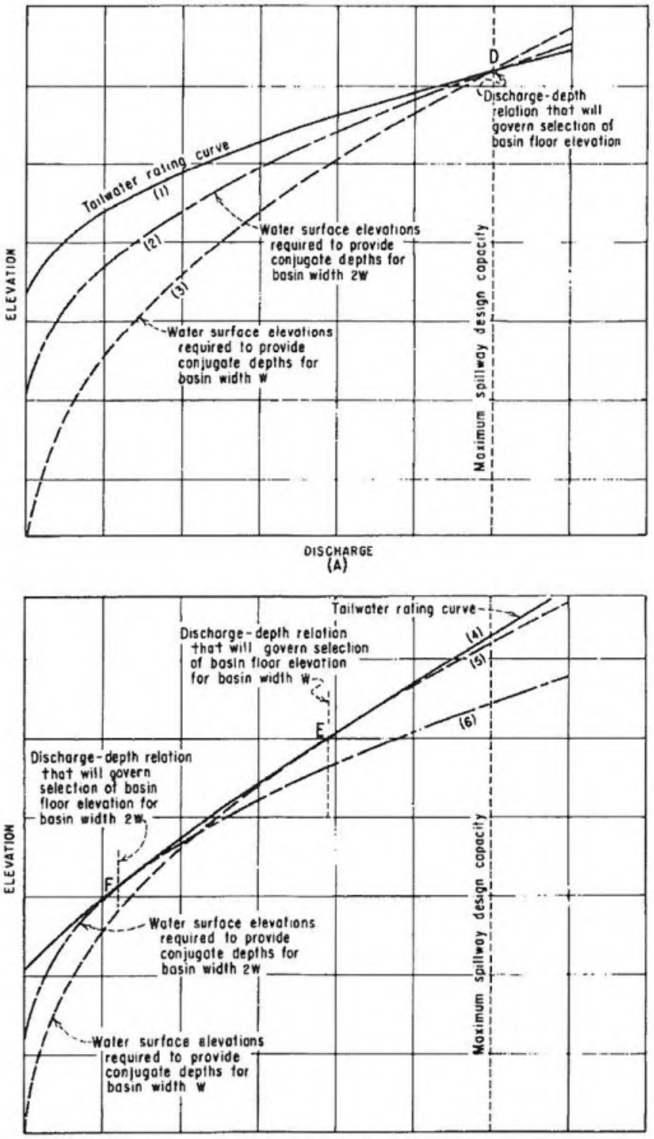


Figure 9-44.—Relationships of conjugate depth curves to tailwater rating curves. 288–D–2429.

width W, the floor level should be selected so that the two curves would coincide at the discharge represented by point E on the figure 9-44(B). For all other discharges the tailwater depth will be greater than that needed to form a satisfactory jump. Similarly, if a basin width of $2W$ were considered, the basin floor level would be selected so that curve 6 would intersect the tailwater curve at point F. Here also, the selection of basin widths should be based on economics as well as on hydraulic performance.

Where exact conjugate depth conditions for forming the jump cannot be attained, the relative desirability of having insufficient tailwater as compared with having excessive tailwater should be considered. With insufficient tailwater the back pressure will be deficient and sweepout of the basin will occur. With an excess of tailwater the jump will be formed, and energy dissipation within the basin will be complete until the drowned-jump phenomenon becomes critical. Chute blocks, baffles, and end sills will also assist in energy dissipation, even with a drowned jump.

(f) *Stilling Basin Freeboard.*—Freeboard is ordinarily provided so that the stilling basin walls will not be overtopped by surges, splash and spray, and wave action set up by the turbulence of the jump. The surface roughness of the flow is related to the energy dissipated in the jump and to the depth of flow in the basin. The following empirical expression provides values that have proved satisfactory for most basins:

$$\text{Freeboard in feet} = 0.1(v_1 + d_2) \qquad (25)$$

9.22. *Submerged Bucket Dissipators.*—When the tailwater depth is too great for the formation of a hydraulic jump, the high energy can be dissipated by the use of a submerged bucket deflector. The hydraulic behavior in this type of dissipator is manifested primarily by the formation of two rollers: one occurs on the surface, moves counterclockwise, and is contained within the region above the curved bucket; the other is a ground roller, moves clockwise, and is situated downstream from the bucket. The movements of these rollers, along with the intermingling of the incoming flows, effectively dissipate the high energy of the water and prevent excessive scouring downstream from the bucket.

Two types of roller buckets have been developed and model tested [15]. Their shape and dimensions are shown on figure 9-45. The general nature of the dissipating action for each type is represented on figure 9-46. The hydraulic actions of the two buckets have the same characteristics, but distinctive features of their flows differ to the extent that each has certain limitations. The high-velocity flow leaving the deflector lip of the solid bucket is directed upward (fig. 9-46(A)). This creates a high boil on the water surface and a violent ground roller moving clockwise downstream from the bucket. This ground roller continuously pulls loose material back towards the lip of the bucket and keeps some of the intermingling material in a constant state of agitation. The typical scour pattern that results from this action is shown on figure 9-47. The high-velocity jet leaves the lip of a slotted bucket at a flatter angle, and only a part of the high-velocity flow finds its way to the surface (fig. 9-46(B)). Thus, a less violent surface boil occurs, and there is a better dissipation of flow in the region above the ground roller. This results in less concentration of high-energy flow throughout the bucket and a smoother downstream flow.

Use of a solid bucket dissipator may be objectionable because of the abrasion on the concrete surfaces caused by material that is swept back along the lip of the deflector by the ground roller. In addition, the more turbulent surface roughness induced by the severe surface boil carries farther down the river, causing objectionable eddy currents that contribute to riverbank sloughing. Although the slotted bucket provides better energy dissipation with less severe surface and streambed disturbances, it is more sensitive to sweepout at lower tailwaters and is conducive to a diving and scouring action at excessive tailwaters. This is not the case with the solid bucket. Thus, the tailwater range that provides good performance with the slotted bucket is much narrower than that of the solid bucket. A

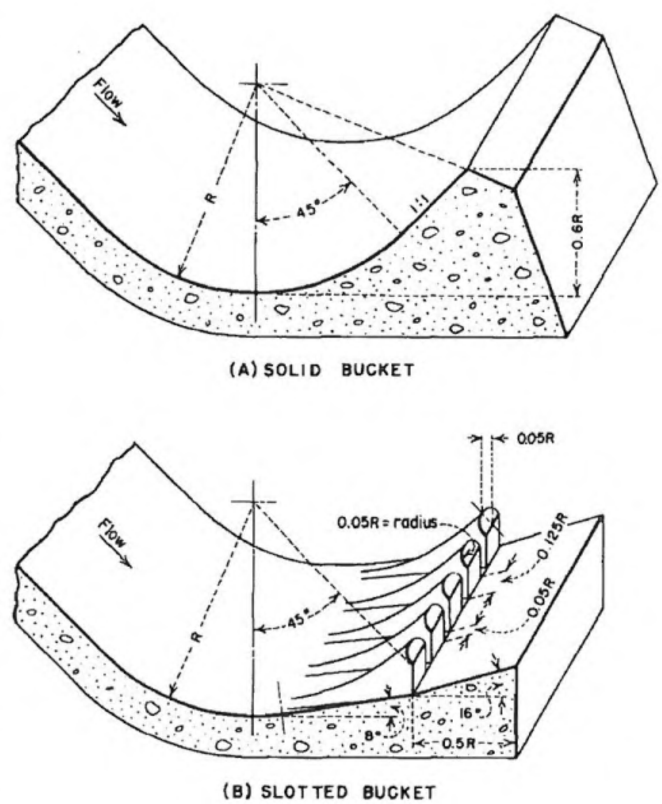
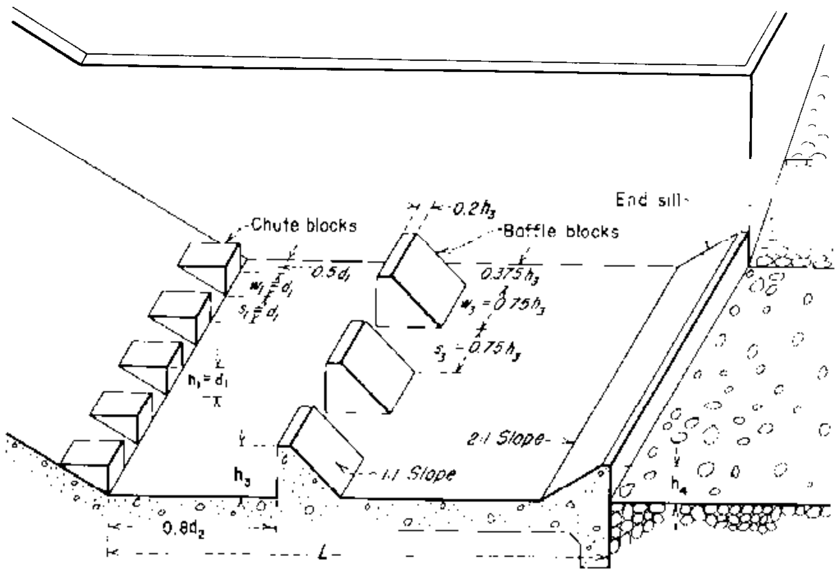


Figure 9-45.—Submerged buckets. 288-D-2430.

ATTACHMENT B - Stilling Basin Design

d2 = 24.51
d1= 4.08
v1=52.6
Fr=4.6



(A) TYPE III BASIN DIMENSIONS

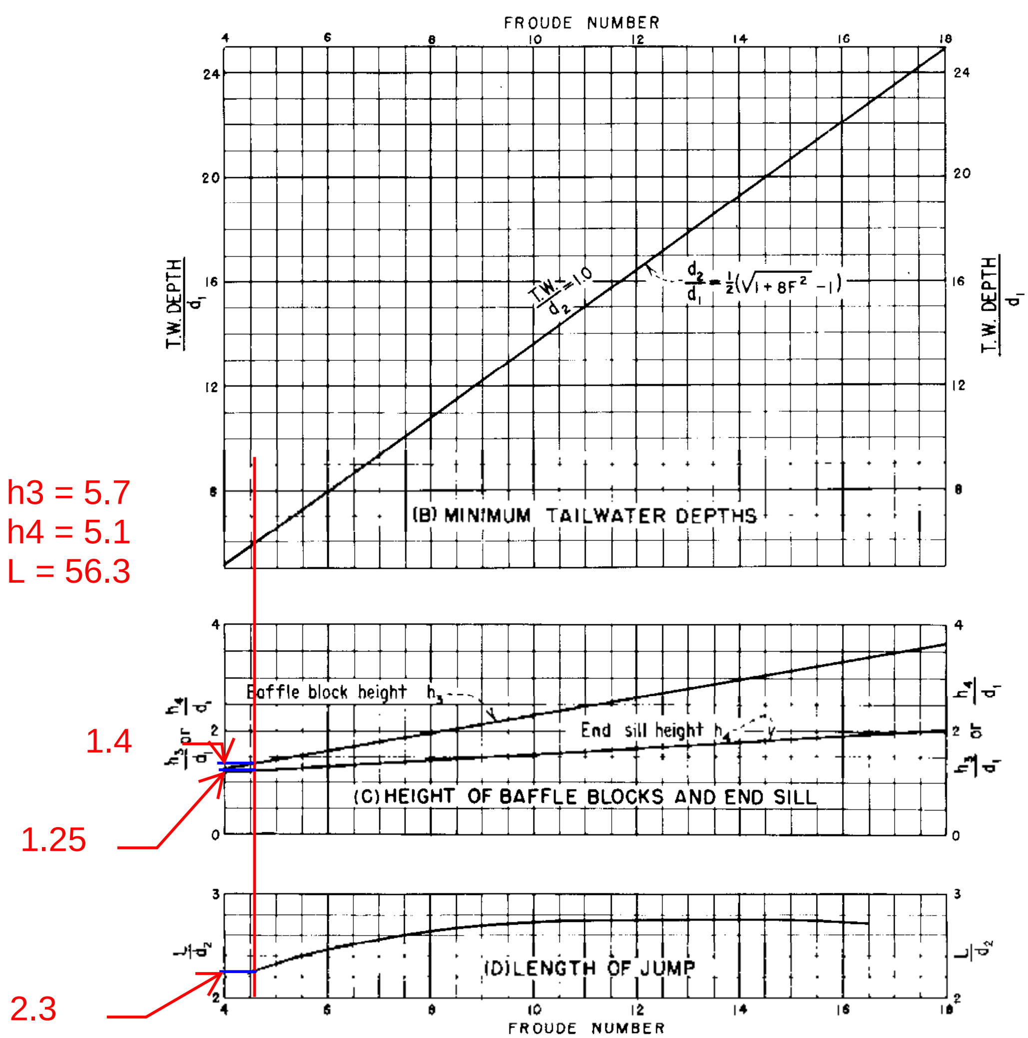
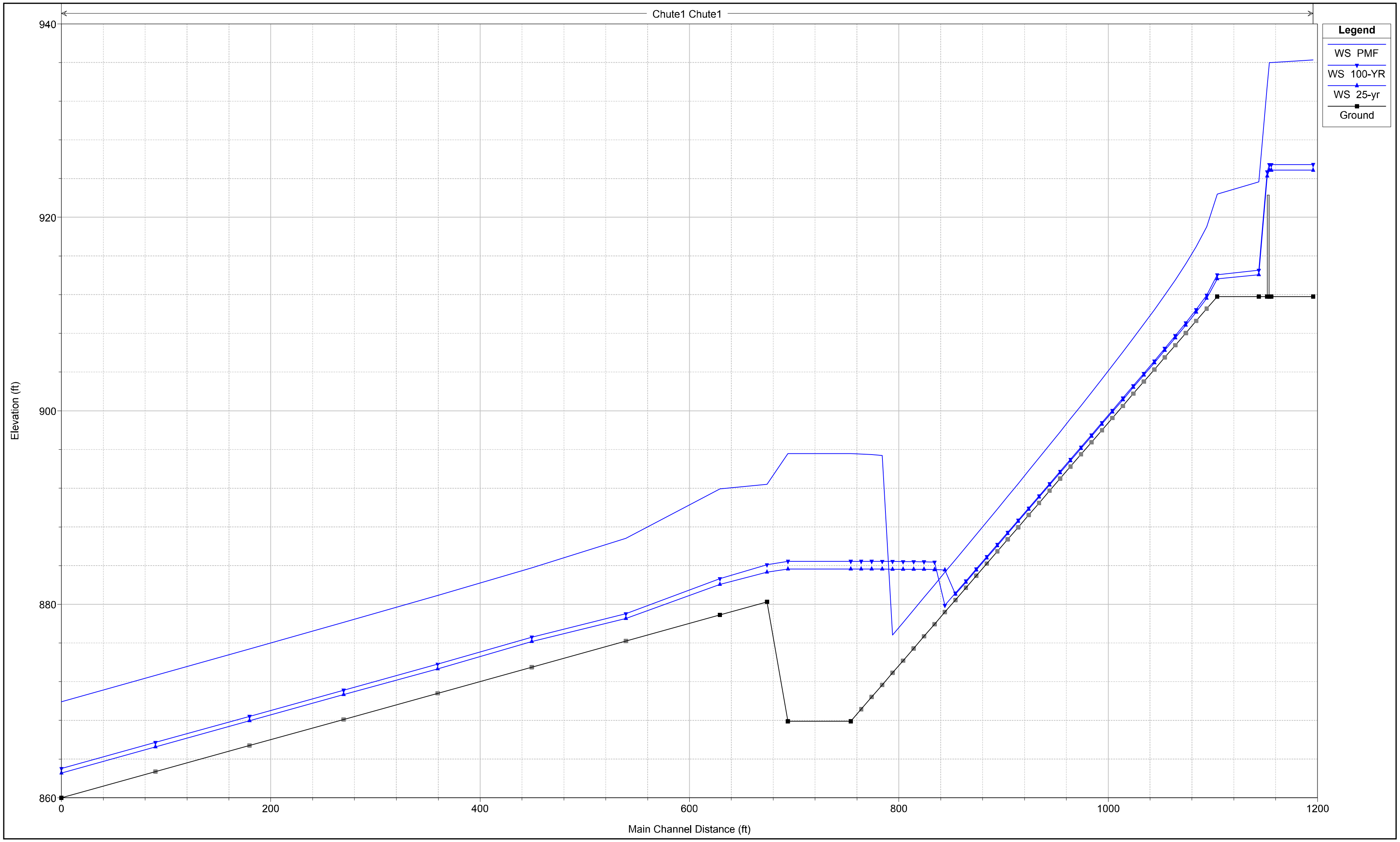
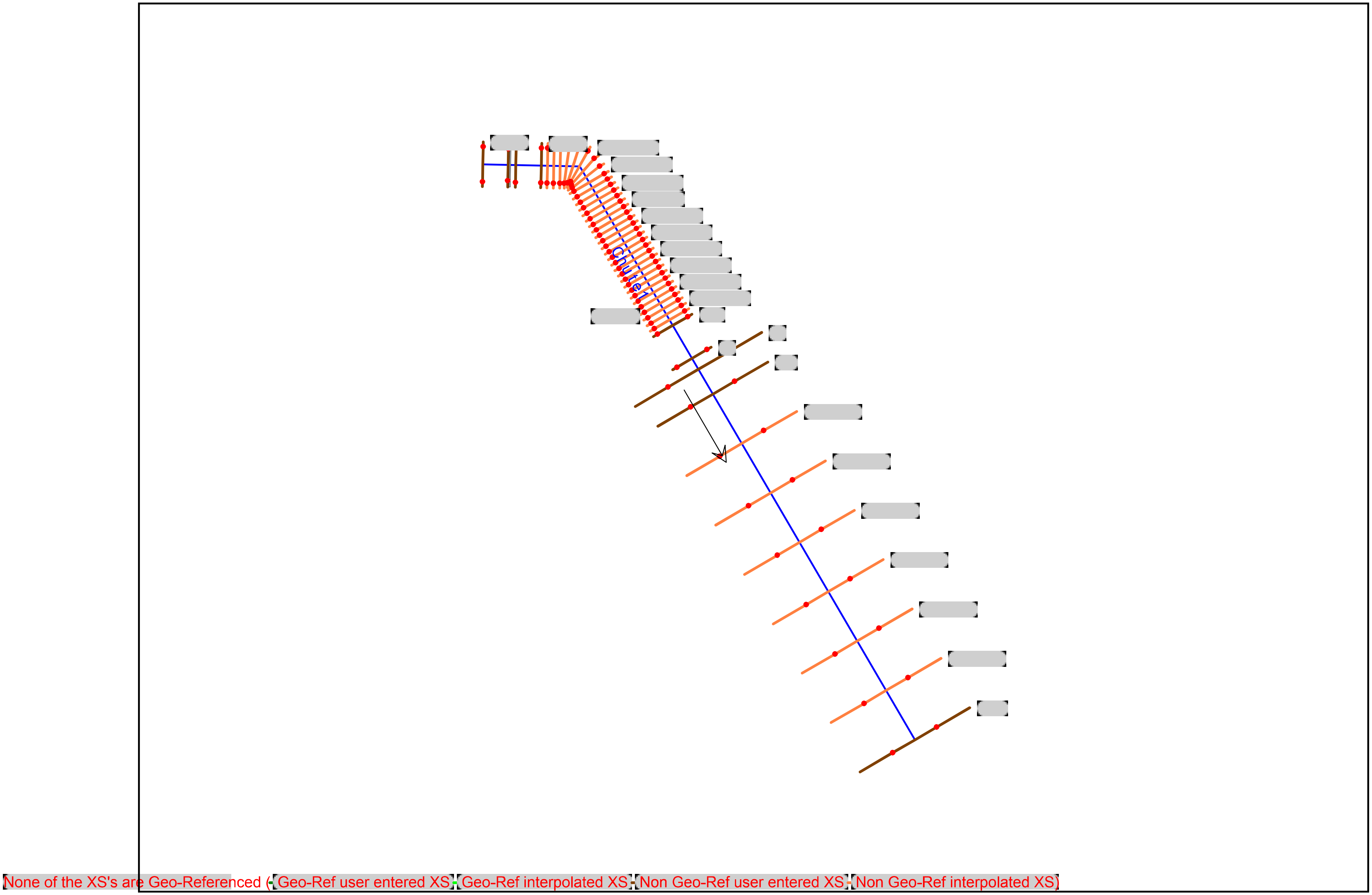


Figure 9-41.—Stilling basin characteristics for Froude numbers above 4.5 where incoming velocity, $V_1 \leq 60$ ft/s. 288–D–2426.

Freeboard = 0.1(52.6 + 24.51) = 7.7 ft

Attachment C - HEC-RAS Output




Chute1
None of the XS's are Geo-Referenced (
Geo-Ref user entered XS
Geo-Ref interpolated XS
Non Geo-Ref user entered XS
Non Geo-Ref interpolated XS)

HEC-RAS Plan: Plan 01 River: Chute1 Reach: Chute1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	542.5	PMF	10817.00	911.80	936.26	922.42	937.27	0.000217	8.04	1345.65	55.02	0.29
Chute1	542.5	25-yr	783.00	911.80	924.87	913.64	924.89	0.000007	1.09	719.17	55.01	0.05
Chute1	542.5	100-YR	1065.00	911.80	925.45	914.06	925.48	0.000011	1.42	750.96	55.01	0.07
Chute1	502.65	PMF	10817.00	911.80	935.99	923.12	937.23	0.000287	8.94	1209.74	50.02	0.32
Chute1	502.65	25-yr	783.00	911.80	924.87	913.76	924.89	0.000008	1.20	653.57	50.01	0.06
Chute1	502.65	100-YR	1065.00	911.80	925.44	914.21	925.48	0.000014	1.56	682.31	50.01	0.07
Chute1	496.65		Inl Struct									
Chute1	490.65	PMF	10817.00	911.80	923.66	922.40	927.93	0.001670	16.58	652.46	55.01	0.85
Chute1	490.65	25-yr	783.00	911.80	914.07		914.68	0.001489	6.27	124.96	55.00	0.73
Chute1	490.65	100-YR	1065.00	911.80	914.53	914.06	915.31	0.001516	7.08	150.40	55.00	0.75
Chute1	450.8	PMF	10817.00	911.80	922.40	922.40	927.74	0.002328	18.56	582.93	55.01	1.00
Chute1	450.8	25-yr	783.00	911.80	913.64	913.64	914.57	0.002951	7.74	101.19	55.00	1.01
Chute1	450.8	100-YR	1065.00	911.80	914.06	914.06	915.20	0.002806	8.57	124.26	55.00	1.00
Chute1	440.777*	PMF	10817.00	910.55	919.01	921.17	927.40	0.004562	23.24	465.53	55.01	1.41
Chute1	440.777*	25-yr	783.00	910.55	911.63	912.39	914.34	0.016954	13.21	59.26	55.00	2.24
Chute1	440.777*	100-YR	1065.00	910.55	911.94	912.81	914.96	0.013671	13.95	76.35	55.00	2.09
Chute1	430.754*	PMF	10817.00	909.29	916.97	919.91	927.16	0.006136	25.62	422.16	55.01	1.63
Chute1	430.754*	25-yr	783.00	909.29	910.20	911.13	914.01	0.029629	15.66	50.00	55.00	2.89
Chute1	430.754*	100-YR	1065.00	909.29	910.47	911.55	914.67	0.023423	16.44	64.78	55.00	2.67
Chute1	420.731*	PMF	10817.00	908.04	915.19	918.66	926.94	0.007615	27.50	393.29	55.01	1.81
Chute1	420.731*	25-yr	783.00	908.04	908.86	909.88	913.57	0.042024	17.41	44.97	55.00	3.39
Chute1	420.731*	100-YR	1065.00	908.04	909.10	910.30	914.29	0.033143	18.28	58.27	55.00	3.13
Chute1	410.708*	PMF	10817.00	906.78	913.53	917.40	926.71	0.009084	29.13	371.29	55.01	1.98
Chute1	410.708*	25-yr	783.00	906.78	907.54	908.62	913.01	0.053876	18.78	41.70	55.00	3.80
Chute1	410.708*	100-YR	1065.00	906.78	907.76	909.04	913.82	0.042790	19.75	53.91	55.00	3.52
Chute1	400.685*	PMF	10817.00	905.53	911.96	916.15	926.48	0.010539	30.57	353.79	55.01	2.12
Chute1	400.685*	25-yr	783.00	905.53	906.25	907.37	912.36	0.064624	19.84	39.46	55.00	4.13
Chute1	400.685*	100-YR	1065.00	905.53	906.45	907.79	913.27	0.051975	20.96	50.82	55.00	3.84

HEC-RAS Plan: Plan 01 River: Chute1 Reach: Chute1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	390.662*	PMF	10817.00	904.27	910.43	914.89	926.24	0.012014	31.90	339.09	55.01	2.26
Chute1	390.662*	25-yr	783.00	904.27	904.96	906.11	911.61	0.074377	20.70	37.82	55.00	4.40
Chute1	390.662*	100-YR	1065.00	904.27	905.15	906.53	912.65	0.060693	21.97	48.48	55.00	4.12
Chute1	380.64*	PMF	10817.00	903.02	908.96	913.64	925.99	0.013483	33.11	326.67	55.00	2.39
Chute1	380.64*	25-yr	783.00	903.02	903.69	904.86	910.78	0.082726	21.38	36.62	55.00	4.62
Chute1	380.64*	100-YR	1065.00	903.02	903.87	905.28	911.94	0.068604	22.80	46.71	55.00	4.36
Chute1	370.617*	PMF	10817.00	901.77	907.51	912.39	925.72	0.014961	34.24	315.87	55.00	2.52
Chute1	370.617*	25-yr	783.00	901.77	902.42	903.61	909.88	0.089887	21.93	35.71	55.00	4.80
Chute1	370.617*	100-YR	1065.00	901.77	902.59	904.03	911.17	0.075796	23.50	45.32	55.00	4.56
Chute1	360.594*	PMF	10817.00	900.51	906.08	911.13	925.44	0.016456	35.31	306.34	55.00	2.64
Chute1	360.594*	25-yr	783.00	900.51	901.15	902.35	908.92	0.096168	22.38	34.99	55.00	4.94
Chute1	360.594*	100-YR	1065.00	900.51	901.31	902.77	910.33	0.082364	24.10	44.19	55.00	4.74
Chute1	350.571*	PMF	10817.00	899.26	904.68	909.88	925.16	0.017963	36.32	297.84	55.00	2.75
Chute1	350.571*	25-yr	783.00	899.26	899.89	901.10	907.91	0.101156	22.72	34.46	55.00	5.06
Chute1	350.571*	100-YR	1065.00	899.26	900.05	901.52	909.44	0.088070	24.60	43.30	55.00	4.89
Chute1	340.548*	PMF	10817.00	898.00	903.28	908.62	924.86	0.019490	37.28	290.13	55.00	2.86
Chute1	340.548*	25-yr	783.00	898.00	898.62	899.84	906.86	0.105805	23.03	33.99	55.00	5.16
Chute1	340.548*	100-YR	1065.00	898.00	898.77	900.26	908.50	0.093340	25.03	42.54	55.00	5.02
Chute1	330.525*	PMF	10817.00	896.75	901.90	907.37	924.55	0.021016	38.19	283.22	55.00	2.97
Chute1	330.525*	25-yr	783.00	896.75	897.36	898.59	905.76	0.109252	23.26	33.66	55.00	5.24
Chute1	330.525*	100-YR	1065.00	896.75	897.51	899.01	907.53	0.097882	25.40	41.93	55.00	5.13
Chute1	320.502*	PMF	10817.00	895.49	900.52	906.11	924.23	0.022561	39.07	276.86	55.00	3.07
Chute1	320.502*	25-yr	783.00	895.49	896.10	897.33	904.64	0.112282	23.45	33.38	55.00	5.31
Chute1	320.502*	100-YR	1065.00	895.49	896.24	897.75	906.50	0.101833	25.71	41.43	55.00	5.22
Chute1	310.48*	PMF	10817.00	894.24	899.17	904.86	923.89	0.024097	39.90	271.09	55.00	3.17
Chute1	310.48*	25-yr	783.00	894.24	894.84	896.08	903.49	0.114638	23.60	33.18	55.00	5.36
Chute1	310.48*	100-YR	1065.00	894.24	894.99	896.50	905.44	0.105102	25.95	41.04	55.00	5.29
Chute1	300.457*	PMF	10817.00	892.99	897.82	903.61	923.54	0.025641	40.70	265.77	55.00	3.26
Chute1	300.457*	25-yr	783.00	892.99	893.59	894.83	902.33	0.116741	23.73	33.00	55.00	5.40

HEC-RAS Plan: Plan 01 River: Chute1 Reach: Chute1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	300.457*	100-YR	1065.00	892.99	893.73	895.25	904.36	0.107980	26.17	40.70	55.00	5.36
Chute1	290.434*	PMF	10817.00	891.73	896.47	902.35	923.19	0.027207	41.48	260.79	55.00	3.36
Chute1	290.434*	25-yr	783.00	891.73	892.33	893.57	901.16	0.118588	23.84	32.84	55.00	5.44
Chute1	290.434*	100-YR	1065.00	891.73	892.46	893.99	903.25	0.110596	26.36	40.41	55.00	5.42
Chute1	280.411*	PMF	10817.00	890.48	895.14	901.10	922.81	0.028753	42.21	256.24	55.00	3.45
Chute1	280.411*	25-yr	783.00	890.48	891.08	892.32	899.95	0.119680	23.91	32.75	55.00	5.46
Chute1	280.411*	100-YR	1065.00	890.48	891.21	892.74	902.12	0.112723	26.51	40.18	55.00	5.47
Chute1	270.388*	PMF	10817.00	889.22	893.80	899.84	922.42	0.030313	42.93	251.96	55.00	3.53
Chute1	270.388*	25-yr	783.00	889.22	889.81	891.06	898.74	0.120787	23.98	32.66	55.00	5.48
Chute1	270.388*	100-YR	1065.00	889.22	889.95	891.48	900.97	0.114690	26.65	39.96	55.00	5.51
Chute1	260.365*	PMF	10817.00	887.97	892.48	898.59	922.02	0.031864	43.62	248.01	55.00	3.62
Chute1	260.365*	25-yr	783.00	887.97	888.56	889.81	897.52	0.121573	24.02	32.59	55.00	5.50
Chute1	260.365*	100-YR	1065.00	887.97	888.69	890.23	899.81	0.116301	26.76	39.80	55.00	5.54
Chute1	250.342*	PMF	10817.00	886.71	891.15	897.33	921.61	0.033428	44.29	244.25	55.00	3.70
Chute1	250.342*	25-yr	783.00	886.71	887.30	888.55	896.30	0.122408	24.07	32.53	55.00	5.52
Chute1	250.342*	100-YR	1065.00	886.71	887.43	888.97	898.64	0.117811	26.87	39.64	55.00	5.58
Chute1	240.32*	PMF	10817.00	885.46	889.84	896.08	921.18	0.034966	44.92	240.78	55.00	3.78
Chute1	240.32*	25-yr	783.00	885.46	886.05	887.30	895.07	0.122828	24.10	32.49	55.00	5.53
Chute1	240.32*	100-YR	1065.00	885.46	886.18	887.72	897.44	0.118775	26.93	39.54	55.00	5.60
Chute1	230.297*	PMF	10817.00	884.21	888.53	894.83	920.73	0.036502	45.54	237.53	55.00	3.86
Chute1	230.297*	25-yr	783.00	884.21	884.80	886.05	893.83	0.123182	24.12	32.46	55.00	5.53
Chute1	230.297*	100-YR	1065.00	884.21	884.93	886.47	896.24	0.119657	26.99	39.45	55.00	5.62
Chute1	220.274*	PMF	10817.00	882.95	887.21	893.57	920.27	0.038046	46.14	234.43	55.00	3.94
Chute1	220.274*	25-yr	783.00	882.95	883.54	884.79	892.59	0.123648	24.15	32.42	55.00	5.54
Chute1	220.274*	100-YR	1065.00	882.95	883.67	885.21	895.03	0.120572	27.06	39.36	55.00	5.64
Chute1	210.251*	PMF	10817.00	881.70	885.91	892.32	919.79	0.039551	46.71	231.56	55.00	4.01
Chute1	210.251*	25-yr	783.00	881.70	882.29	883.54	891.35	0.123776	24.16	32.41	55.00	5.55
Chute1	210.251*	100-YR	1065.00	881.70	882.41	883.96	893.81	0.121121	27.09	39.31	55.00	5.65

HEC-RAS Plan: Plan 01 River: Chute1 Reach: Chute1 (Continued)

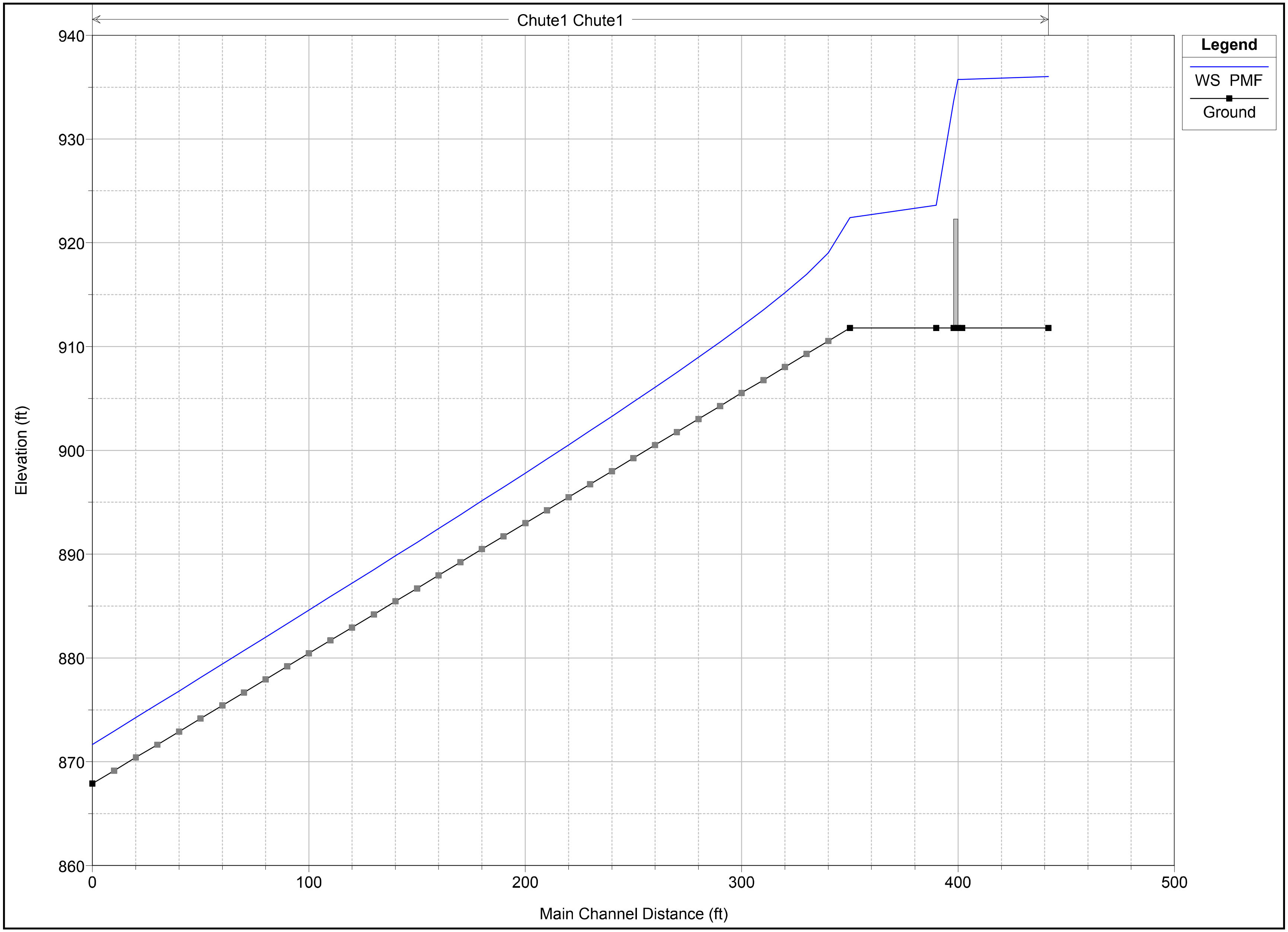
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	200.228*	PMF	10817.00	880.44	884.60	891.06	919.31	0.041078	47.28	228.79	55.00	4.09
Chute1	200.228*	25-yr	783.00	880.44	881.03	882.28	890.11	0.124159	24.18	32.38	55.00	5.55
Chute1	200.228*	100-YR	1065.00	880.44	881.15	882.70	892.59	0.121842	27.14	39.24	55.00	5.66
Chute1	190.205*	PMF	10817.00	879.19	883.30	889.81	918.81	0.042589	47.82	226.19	55.00	4.16
Chute1	190.205*	25-yr	783.00	879.19	883.54	881.03	883.71	0.000187	3.27	239.23	55.00	0.28
Chute1	190.205*	100-YR	1065.00	879.19	879.90	881.45	891.36	0.122259	27.17	39.20	55.00	5.67
Chute1	180.182*	PMF	10817.00	877.93	882.00	888.55	918.31	0.044114	48.36	223.69	55.00	4.23
Chute1	180.182*	25-yr	783.00	877.93	883.59		883.68	0.000082	2.52	311.07	55.00	0.19
Chute1	180.182*	100-YR	1065.00	877.93	884.36	880.19	884.50	0.000102	3.01	353.70	55.01	0.21
Chute1	170.16*	PMF	10817.00	876.68	880.70	887.30	917.78	0.045603	48.87	221.36	55.00	4.29
Chute1	170.16*	25-yr	783.00	876.68	883.61		883.67	0.000044	2.05	381.05	55.01	0.14
Chute1	170.16*	100-YR	1065.00	876.68	884.39		884.49	0.000059	2.51	424.07	55.01	0.16
Chute1	160.137*	PMF	10817.00	875.43	879.41	886.05	917.24	0.047067	49.36	219.15	55.00	4.36
Chute1	160.137*	25-yr	783.00	875.43	883.62		883.67	0.000026	1.74	450.51	55.01	0.11
Chute1	160.137*	100-YR	1065.00	875.43	884.41		884.48	0.000037	2.16	493.79	55.01	0.13
Chute1	150.114*	PMF	10817.00	874.17	878.12	884.79	916.69	0.048546	49.84	217.03	55.00	4.42
Chute1	150.114*	25-yr	783.00	874.17	883.63		883.66	0.000017	1.51	520.26	55.01	0.09
Chute1	150.114*	100-YR	1065.00	874.17	884.42		884.47	0.000025	1.89	563.73	55.01	0.10
Chute1	140.091*	PMF	10817.00	872.92	876.83	883.54	916.13	0.049990	50.31	215.02	55.00	4.48
Chute1	140.091*	25-yr	783.00	872.92	883.63		883.66	0.000012	1.33	589.31	55.01	0.07
Chute1	140.091*	100-YR	1065.00	872.92	884.43		884.47	0.000018	1.68	632.92	55.01	0.09
Chute1	130.068*	PMF	10817.00	871.66	895.39	882.28	896.45	0.000235	8.29	1305.26	55.02	0.30
Chute1	130.068*	25-yr	783.00	871.66	883.64		883.66	0.000009	1.19	658.83	55.01	0.06
Chute1	130.068*	100-YR	1065.00	871.66	884.43		884.47	0.000013	1.52	702.54	55.01	0.07
Chute1	120.045*	PMF	10817.00	870.41	895.46		896.42	0.000197	7.84	1386.53	70.02	0.28
Chute1	120.045*	25-yr	783.00	870.41	883.64		883.66	0.000006	1.08	727.74	55.01	0.05
Chute1	120.045*	100-YR	1065.00	870.41	884.44		884.47	0.000010	1.38	771.53	55.01	0.06
Chute1	110.022*	PMF	10817.00	869.15	895.53		896.39	0.000165	7.42	1479.69	70.02	0.25
Chute1	110.022*	25-yr	783.00	869.15	883.64		883.66	0.000005	0.98	797.16	55.01	0.05

HEC-RAS Plan: Plan 01 River: Chute1 Reach: Chute1 (Continued)

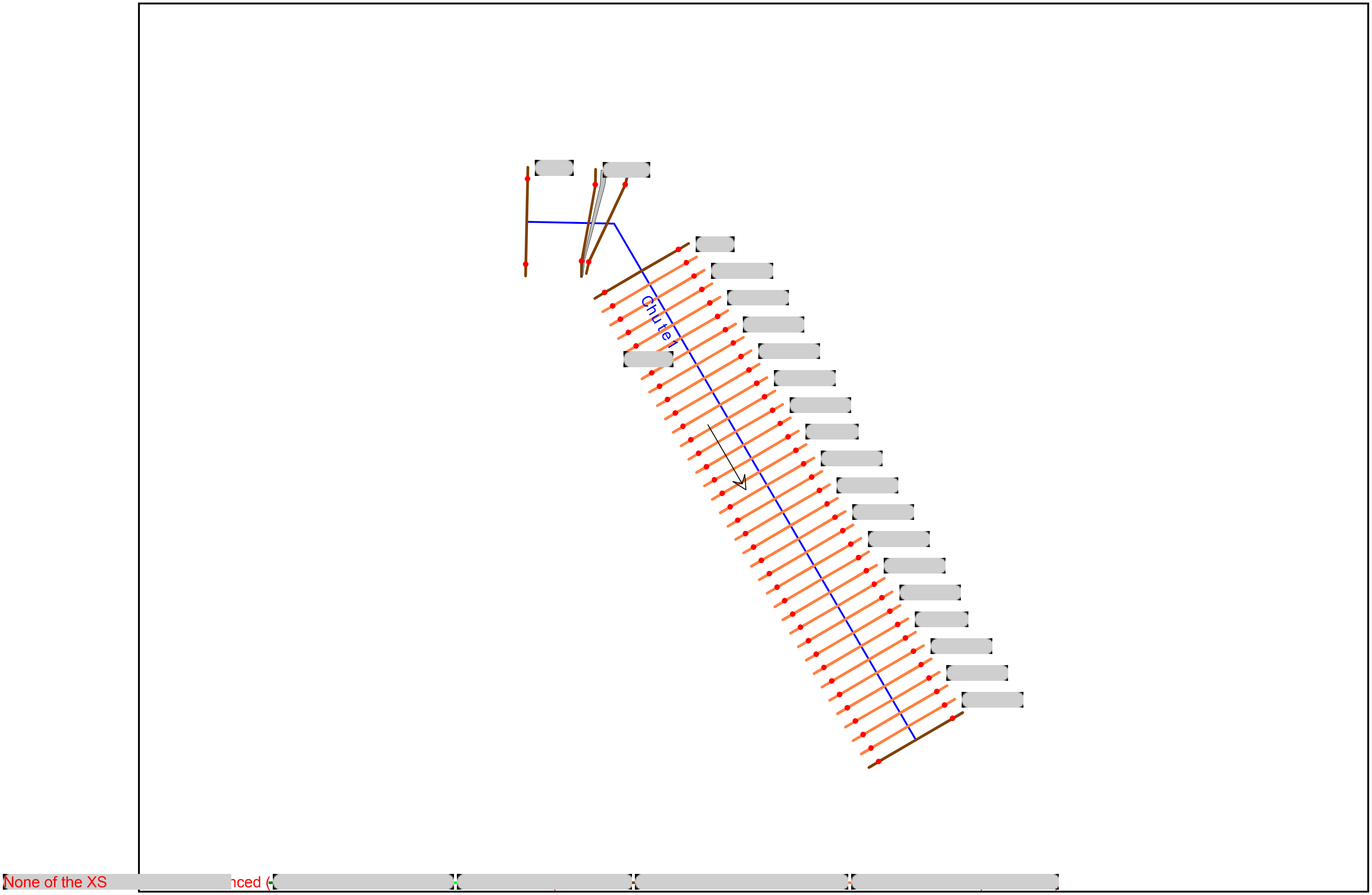
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	110.022*	100-YR	1065.00	869.15	884.44		884.46	0.000008	1.27	841.02	55.01	0.06
Chute1	100	PMF	10817.00	867.90	895.58		896.36	0.000155	7.10	1522.65	55.02	0.24
Chute1	100	25-yr	783.00	867.90	883.64		883.66	0.000004	0.90	865.99	55.01	0.04
Chute1	100	100-YR	1065.00	867.90	884.44		884.46	0.000006	1.17	909.90	55.01	0.05
Chute1	40	PMF	10817.00	867.90	895.57		896.35	0.000156	7.11	1522.09	55.02	0.24
Chute1	40	25-yr	783.00	867.90	883.64		883.66	0.000004	0.90	865.97	55.01	0.04
Chute1	40	100-YR	1065.00	867.90	884.44		884.46	0.000006	1.17	909.88	55.01	0.05
Chute1	20	PMF	10817.00	880.25	892.41	889.80	896.06	0.003870	11.02	977.90	116.33	0.65
Chute1	20	25-yr	783.00	880.25	883.32		883.63	0.002928	4.42	177.34	65.37	0.47
Chute1	20	100-YR	1065.00	880.25	884.10		884.43	0.002494	4.64	229.29	69.23	0.45
Chute1	-25	PMF	10817.00	878.90	891.92	891.92	895.79	0.008582	15.81	702.90	120.96	0.95
Chute1	-25	25-yr	783.00	878.90	882.04	882.04	883.28	0.014024	8.94	87.58	35.72	1.01
Chute1	-25	100-YR	1065.00	878.90	882.66	882.66	884.10	0.013399	9.64	110.51	38.80	1.01
Chute1	-114.85*	PMF	10817.00	876.20	886.82	889.22	894.25	0.022335	21.88	494.35	73.10	1.48
Chute1	-114.85*	25-yr	783.00	876.20	878.54	879.34	881.14	0.040846	12.95	60.45	31.69	1.65
Chute1	-114.85*	100-YR	1065.00	876.20	879.02	879.97	882.05	0.038744	13.99	76.14	34.08	1.65
Chute1	-204.71*	PMF	10817.00	873.50	883.77	886.52	892.02	0.025704	23.05	469.33	71.37	1.58
Chute1	-204.71*	25-yr	783.00	873.50	876.16	876.64	878.06	0.025765	11.05	70.86	33.30	1.33
Chute1	-204.71*	100-YR	1065.00	873.50	876.61	877.28	878.97	0.027050	12.34	86.31	35.54	1.40
Chute1	-294.57*	PMF	10817.00	870.80	880.91	883.82	889.59	0.027540	23.64	457.53	70.54	1.64
Chute1	-294.57*	25-yr	783.00	870.80	873.32	873.94	875.49	0.031218	11.81	66.31	32.61	1.46
Chute1	-294.57*	100-YR	1065.00	870.80	873.81	874.56	876.38	0.030381	12.85	82.87	35.05	1.47
Chute1	-384.42*	PMF	10817.00	868.10	878.12	881.12	887.04	0.028591	23.97	451.24	70.09	1.66
Chute1	-384.42*	25-yr	783.00	868.10	870.66	871.24	872.74	0.029350	11.56	67.74	32.82	1.42
Chute1	-384.42*	100-YR	1065.00	868.10	871.13	871.86	873.66	0.029848	12.77	83.39	35.13	1.46
Chute1	-474.28*	PMF	10817.00	865.40	875.37	878.42	884.43	0.029164	24.15	447.94	69.85	1.68
Chute1	-474.28*	25-yr	783.00	865.40	867.95	868.54	870.06	0.030111	11.66	67.14	32.73	1.43
Chute1	-474.28*	100-YR	1065.00	865.40	868.42	869.16	870.97	0.030155	12.82	83.09	35.08	1.47

HEC-RAS Plan: Plan 01   River: Chute1   Reach: Chute1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	-564.14*	PMF	10817.00	862.70	872.64	875.72	881.78	0.029529	24.26	445.89	69.71	1.69
Chute1	-564.14*	25-yr	783.00	862.70	865.25	865.84	867.36	0.030062	11.65	67.18	32.74	1.43
Chute1	-564.14*	100-YR	1065.00	862.70	865.72	866.46	868.26	0.030004	12.79	83.24	35.10	1.46
Chute1	-654	PMF	10817.00	860.00	869.92	873.02	879.11	0.029743	24.32	444.70	69.62	1.70
Chute1	-654	25-yr	783.00	860.00	862.55	863.14	864.66	0.030062	11.65	67.18	32.74	1.43
Chute1	-654	100-YR	1065.00	860.00	863.02	863.76	865.56	0.030004	12.79	83.24	35.10	1.46



Chute1 Chute1
Legend
WS PMF
Ground
Elevation (ft)
Main Channel Distance (ft)
940
930
920
910
900
890
880
870
860
0
100
200
300
400
500


Chute 1
None of the XS
nced (

HEC-RAS Plan: chute2 River: Chute1 Reach: Chute1 Profile: PMF

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	542.5	PMF	10817.00	911.80	936.03	922.42	937.05	0.000222	8.12	1332.73	55.02	0.29
Chute1	502.65	PMF	10817.00	911.80	935.75	923.12	937.02	0.000295	9.03	1197.67	50.02	0.33
Chute1	496.65		Inl Struct									
Chute1	490.65	PMF	10817.00	911.80	923.62	922.42	927.92	0.001688	16.64	650.06	55.01	0.85
Chute1	450.8	PMF	10817.00	911.80	922.42	922.42	927.74	0.002315	18.52	584.03	55.01	1.00
Chute1	440.777*	PMF	10817.00	910.55	919.01	921.17	927.40	0.004561	23.24	465.53	55.01	1.41
Chute1	430.754*	PMF	10817.00	909.29	916.97	919.91	927.16	0.006135	25.62	422.16	55.01	1.63
Chute1	420.731*	PMF	10817.00	908.04	915.19	918.66	926.94	0.007615	27.50	393.29	55.01	1.81
Chute1	410.708*	PMF	10817.00	906.78	913.53	917.40	926.71	0.009084	29.13	371.29	55.01	1.98
Chute1	400.685*	PMF	10817.00	905.53	911.96	916.15	926.48	0.010538	30.57	353.79	55.01	2.12
Chute1	390.662*	PMF	10817.00	904.27	910.43	914.89	926.24	0.012013	31.90	339.09	55.00	2.26
Chute1	380.64*	PMF	10817.00	903.02	908.96	913.64	925.99	0.013483	33.11	326.67	55.00	2.39
Chute1	370.617*	PMF	10817.00	901.77	907.51	912.39	925.72	0.014964	34.25	315.87	55.00	2.52
Chute1	360.594*	PMF	10817.00	900.51	906.08	911.13	925.44	0.016457	35.31	306.33	55.00	2.64
Chute1	350.571*	PMF	10817.00	899.26	904.68	909.88	925.16	0.017961	36.32	297.84	55.00	2.75
Chute1	340.548*	PMF	10817.00	898.00	903.28	908.62	924.86	0.019490	37.28	290.13	55.00	2.86
Chute1	330.525*	PMF	10817.00	896.75	901.90	907.37	924.55	0.021016	38.19	283.22	55.00	2.97
Chute1	320.502*	PMF	10817.00	895.49	900.52	906.11	924.23	0.022558	39.07	276.86	55.00	3.07
Chute1	310.48*	PMF	10817.00	894.24	899.17	904.86	923.89	0.024096	39.90	271.09	55.00	3.17
Chute1	300.457*	PMF	10817.00	892.99	897.82	903.61	923.54	0.025642	40.70	265.77	55.00	3.26
Chute1	290.434*	PMF	10817.00	891.73	896.47	902.35	923.18	0.027205	41.48	260.80	55.00	3.36
Chute1	280.411*	PMF	10817.00	890.48	895.14	901.10	922.81	0.028752	42.21	256.24	55.00	3.45
Chute1	270.388*	PMF	10817.00	889.22	893.80	899.84	922.42	0.030312	42.93	251.96	55.00	3.53
Chute1	260.365*	PMF	10817.00	887.97	892.48	898.59	922.02	0.031864	43.62	248.01	55.00	3.62
Chute1	250.342*	PMF	10817.00	886.71	891.15	897.33	921.61	0.033429	44.29	244.25	55.00	3.70
Chute1	240.32*	PMF	10817.00	885.46	889.84	896.08	921.18	0.034966	44.92	240.79	55.00	3.78
Chute1	230.297*	PMF	10817.00	884.21	888.53	894.83	920.73	0.036501	45.54	237.53	55.00	3.86
Chute1	220.274*	PMF	10817.00	882.95	887.21	893.57	920.27	0.038045	46.14	234.43	55.00	3.94
Chute1	210.251*	PMF	10817.00	881.70	885.91	892.32	919.79	0.039546	46.71	231.56	55.00	4.01
Chute1	200.228*	PMF	10817.00	880.44	884.60	891.06	919.31	0.041079	47.28	228.79	55.00	4.09
Chute1	190.205*	PMF	10817.00	879.19	883.30	889.81	918.81	0.042586	47.82	226.20	55.00	4.16
Chute1	180.182*	PMF	10817.00	877.93	882.00	888.55	918.31	0.044118	48.36	223.69	55.00	4.23
Chute1	170.16*	PMF	10817.00	876.68	880.70	887.30	917.78	0.045603	48.86	221.37	55.00	4.29
Chute1	160.137*	PMF	10817.00	875.43	879.41	886.05	917.24	0.047073	49.36	219.16	55.00	4.36
Chute1	150.114*	PMF	10817.00	874.17	878.12	884.79	916.69	0.048547	49.84	217.03	55.00	4.42
Chute1	140.091*	PMF	10817.00	872.92	876.83	883.54	916.13	0.049985	50.31	215.03	55.00	4.48
Chute1	130.068*	PMF	10817.00	871.66	875.53	882.28	915.55	0.051439	50.76	213.09	55.00	4.54
Chute1	120.045*	PMF	10817.00	870.41	874.25	881.03	914.96	0.052857	51.20	211.27	55.00	4.60

HEC-RAS Plan: chute2 River: Chute1 Reach: Chute1 Profile: PMF (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chute1	110.022*	PMF	10817.00	869.15	872.96	879.77	914.35	0.054278	51.63	209.51	55.00	4.66
Chute1	100	PMF	10817.00	867.90	871.68	878.52	913.73	0.055672	52.04	207.85	55.00	4.72

Attachment D - Chute Freeboard Calculation

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Freeboard	Required Wall Height	Flow depth
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)				
Chute1	542.5	PF 1	10817	911.8	936.56	922.42	937.54	0.00021	7.94	1362.08	55.02	0.28			
Chute1	502.65	PF 1	10817	911.8	936.3	923.12	937.51	0.000278	8.83	1225.06	50.02	0.31			
Chute1	496.65		Labrynith Weir Spillway												
Chute1	490.65	PF 1	10817	911.8	923.62	922.42	927.92	0.001688	16.64	650.06	55.01	0.85	2.95	14.77	11.82
Chute1	450.8	PF 1	10817	911.8	922.42	922.42	927.74	0.002315	18.52	584.02	55.01	1	3.02	13.64	10.62
Chute1	440.777*	PF 1	10817	910.55	919.01	921.17	927.4	0.004562	23.24	465.53	55.01	1.41	3.18	11.64	8.46
Chute1	430.754*	PF 1	10817	909.29	916.97	919.91	927.16	0.006136	25.62	422.16	55.01	1.63	3.26	10.94	7.68
Chute1	420.731*	PF 1	10817	908.04	915.19	918.66	926.94	0.007615	27.5	393.29	55.01	1.81	3.32	10.47	7.15
Chute1	410.708*	PF 1	10817	906.78	913.53	917.4	926.71	0.009084	29.13	371.29	55.01	1.98	3.38	10.13	6.75
Chute1	400.685*	PF 1	10817	905.53	911.96	916.15	926.48	0.010539	30.57	353.79	55.01	2.12	3.42	9.85	6.43
Chute1	390.662*	PF 1	10817	904.27	910.43	914.89	926.24	0.012014	31.9	339.09	55.01	2.26	3.46	9.62	6.16
Chute1	380.64*	PF 1	10817	903.02	908.96	913.64	925.99	0.013483	33.11	326.67	55	2.39	3.50	9.44	5.94
Chute1	370.617*	PF 1	10817	901.77	907.51	912.39	925.72	0.014961	34.24	315.87	55	2.52	3.53	9.27	5.74
Chute1	360.594*	PF 1	10817	900.51	906.08	911.13	925.44	0.016456	35.31	306.34	55	2.64	3.56	9.13	5.57
Chute1	350.571*	PF 1	10817	899.26	904.68	909.88	925.16	0.017963	36.32	297.84	55	2.75	3.59	9.01	5.42
Chute1	340.548*	PF 1	10817	898	903.28	908.62	924.86	0.01949	37.28	290.13	55	2.86	3.62	8.90	5.28
Chute1	330.525*	PF 1	10817	896.75	901.9	907.37	924.55	0.021016	38.19	283.22	55	2.97	3.65	8.80	5.15
Chute1	320.502*	PF 1	10817	895.49	900.52	906.11	924.23	0.022561	39.07	276.86	55	3.07	3.67	8.70	5.03
Chute1	310.48*	PF 1	10817	894.24	899.17	904.86	923.89	0.024097	39.9	271.09	55	3.17	3.70	8.63	4.93
Chute1	300.457*	PF 1	10817	892.99	897.82	903.61	923.54	0.025641	40.7	265.77	55	3.26	3.72	8.55	4.83
Chute1	290.434*	PF 1	10817	891.73	896.47	902.35	923.19	0.027207	41.48	260.79	55	3.36	3.74	8.48	4.74
Chute1	280.411*	PF 1	10817	890.48	895.14	901.1	922.81	0.028753	42.21	256.24	55	3.45	3.76	8.42	4.66
Chute1	270.388*	PF 1	10817	889.22	893.8	899.84	922.42	0.030313	42.93	251.96	55	3.53	3.78	8.36	4.58
Chute1	260.365*	PF 1	10817	887.97	892.48	898.59	922.02	0.031864	43.62	248.01	55	3.62	3.80	8.31	4.51
Chute1	250.342*	PF 1	10817	886.71	891.15	897.33	921.61	0.033428	44.29	244.25	55	3.7	3.82	8.26	4.44
Chute1	240.32*	PF 1	10817	885.46	889.84	896.08	921.18	0.034966	44.92	240.78	55	3.78	3.84	8.22	4.38
Chute1	230.297*	PF 1	10817	884.21	888.53	894.83	920.73	0.036502	45.54	237.53	55	3.86	3.85	8.17	4.32
Chute1	220.274*	PF 1	10817	882.95	887.21	893.57	920.27	0.038046	46.14	234.43	55	3.94	3.87	8.13	4.26
Chute1	210.251*	PF 1	10817	881.7	885.91	892.32	919.79	0.039551	46.71	231.56	55	4.01	3.89	8.10	4.21
Chute1	200.228*	PF 1	10817	880.44	884.6	891.06	919.31	0.041078	47.28	228.79	55	4.09	3.90	8.06	4.16
Chute1	190.205*	PF 1	10817	879.19	883.3	889.81	918.81	0.042589	47.82	226.19	55	4.16	3.91	8.02	4.11
Chute1	180.182*	PF 1	10817	877.93	882	888.55	918.31	0.044114	48.36	223.69	55	4.23	3.93	8.00	4.07
Chute1	170.16*	PF 1	10817	876.68	880.7	887.3	917.78	0.045603	48.87	221.36	55	4.29	3.94	7.96	4.02
Chute1	160.137*	PF 1	10817	875.43	879.41	886.05	917.24	0.047067	49.36	219.15	55	4.36	3.96	7.94	3.98
Chute1	150.114*	PF 1	10817	874.17	878.12	884.79	916.69	0.048546	49.84	217.03	55	4.42	3.97	7.92	3.95
Chute1	140.091*	PF 1	10817	872.92	876.83	883.54	916.13	0.04999	50.31	215.02	55	4.48	3.98	7.89	3.91
Chute1	130.068*	PF 1	10817	871.66	875.53	882.28	915.55	0.05144	50.76	213.09	55	4.54	3.99	7.86	3.87
Chute1	120.045*	PF 1	10817	870.41	874.25	881.03	914.96	0.052864	51.2	211.27	55	4.6	4.00	7.84	3.84
Chute1	110.022*	PF 1	10817	869.15	872.96	879.77	914.35	0.054282	51.63	209.51	55	4.66	4.02	7.83	3.81
Chute1	100	PF 1	10817	867.9	871.68	878.52	913.73	0.05567	52.04	207.85	55	4.72	4.03	7.81	3.78

Attachment E - Riprap Sizing

RIP RAP REVETMENT SIZING

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Project Name:	East Locust Creek	**Stream Name:**	
Project #	A11-1513	**Chain Name**	
Date Created:	8/19/2015	**Calculated:**	
Path=	*F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Spillway\[RipRap Revetm*	**Checked:**	

Hydraulic Engineering Circular No. 11 - Design of Riprap Revetment 1989

VARIABLES		DESCRIPTION
$V_a =$	13	Average Velocity in Main Channel (ft/sec)
$d_{avg} =$	3.10	Average Depth in Main Channel (ft)
$\theta =$	21.80	Bank Angle with Horizontal (deg) (See Bank Angle Sheet)
$\varphi =$	42.0	Material Angle Repose (deg) (See Angle of Repose Sheet)

Section 4.1.1.1 Equation (6)

$$K_1 = \left[1 - \left(\frac{(\sin\theta)^2}{(\sin\varphi)^2}\right)\right]^{0.5}$$

K_1 = 0.8318

$$D_{50} = \frac{0.001 \times V_a^3}{d_{avg}^{0.5} \times K_1^{1.5}}$$

D_{50} = 1.64

Coorection Factor

VARIABLES		DESCRIPTION
SF =	1.10	Stabity Facter (See Stablity Factor Sheet)
$S_s =$	2.65	Specific Gravity of Rip Rap

$$C = \frac{1.61 \times SF^{1.5}}{(S_s - 1)^{1.5}}$$

C = 0.88

CD_{50} = 1.44

EM_1110-2-1601 Hyraulic Design of Flood Control Channels 1994

East Locust Creek Reservoir Water Budget Summary

Appendix C.

East Locust Creek Reservoir Hydrology Report

EAST LOCUST CREEK RESERVOIR HYDROLOGY REPORT

PREPARED FOR

NORTH CENTRAL MISSOURI REGIONAL WATER COMMISSION

MILAN, MISSOURI

STATE OF MISSOURI
BRIAN H. LADD
NUMBER
PE-2014012020
5-24-17
PROFESSIONAL ENGINEER

JUNE 2015
UPDATED MARCH 2016

OA PROJECT NO. A11-1513

OLSSON ASSOCIATES

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A reservoir on East Locust Creek near Milan, Missouri is proposed to provide flood control, recreation and water supply for the surrounding community. The proposed reservoir has been in the planning stages since the 1980s. The design of the proposed reservoir will follow the Natural Resource Conservation Service (NRCS) guidelines for dam construction. This report addresses the hydrology for the contributing drainage area of the proposed reservoir. The report also summarizes the calculation of the storms for sizing the primary and auxiliary spillway and the freeboard required above these spillways.

Watershed Description

The proposed reservoir on East Locust Creek is situated in north-central Missouri. The watershed for the proposed reservoir drains portions of Putnam and Sullivan Counties, Figure 1. The contributing drainage area of the proposed reservoir on East Locust Creek is approximately 32.6 square miles. The watershed for the proposed reservoir was determined by URS in 2013 using Surdex Corporation LiDAR (June 2009). The watershed area was reviewed, validated, and used for this analysis. The watershed is largely rural and contains mostly agricultural lands. Figures 2 and 3 show the extents of the watershed. The table below shows the land uses, as classified by The National Land Cover Database published by the U.S. Geological Survey, 2011. The normal pool of the proposed reservoir is included as open water.

Table 1 - Land Use above Proposed Reservoir

Land Use in East Locust Creek Watershed Upstream of Proposed Reservoir	
Hay/Pasture	56%
Deciduous Forest	20%
Open Water	12%
Developed, Open Space	4%
Cultivated Crops	2%
Other Land Uses	6%

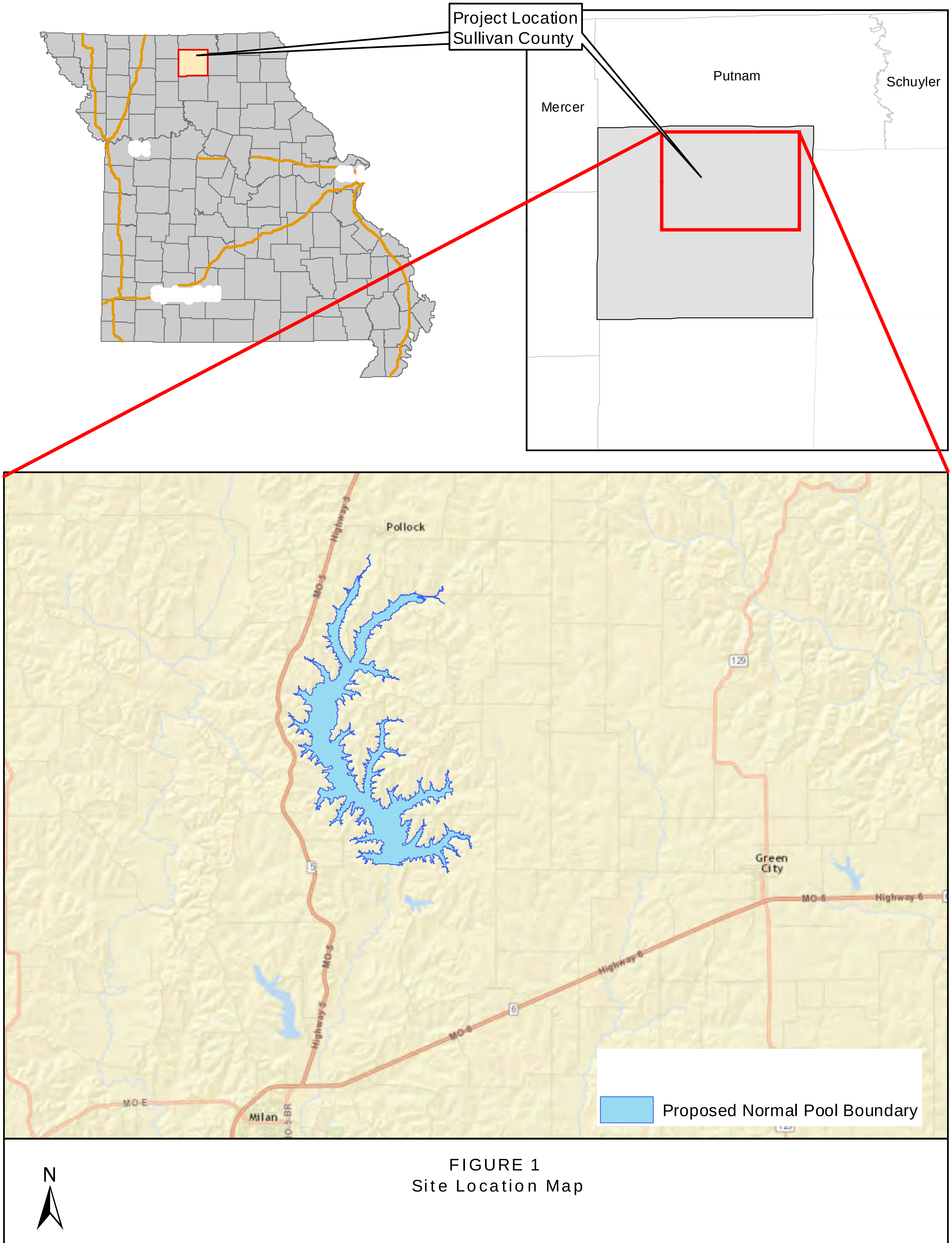

FIGURE 1
Site Location Map

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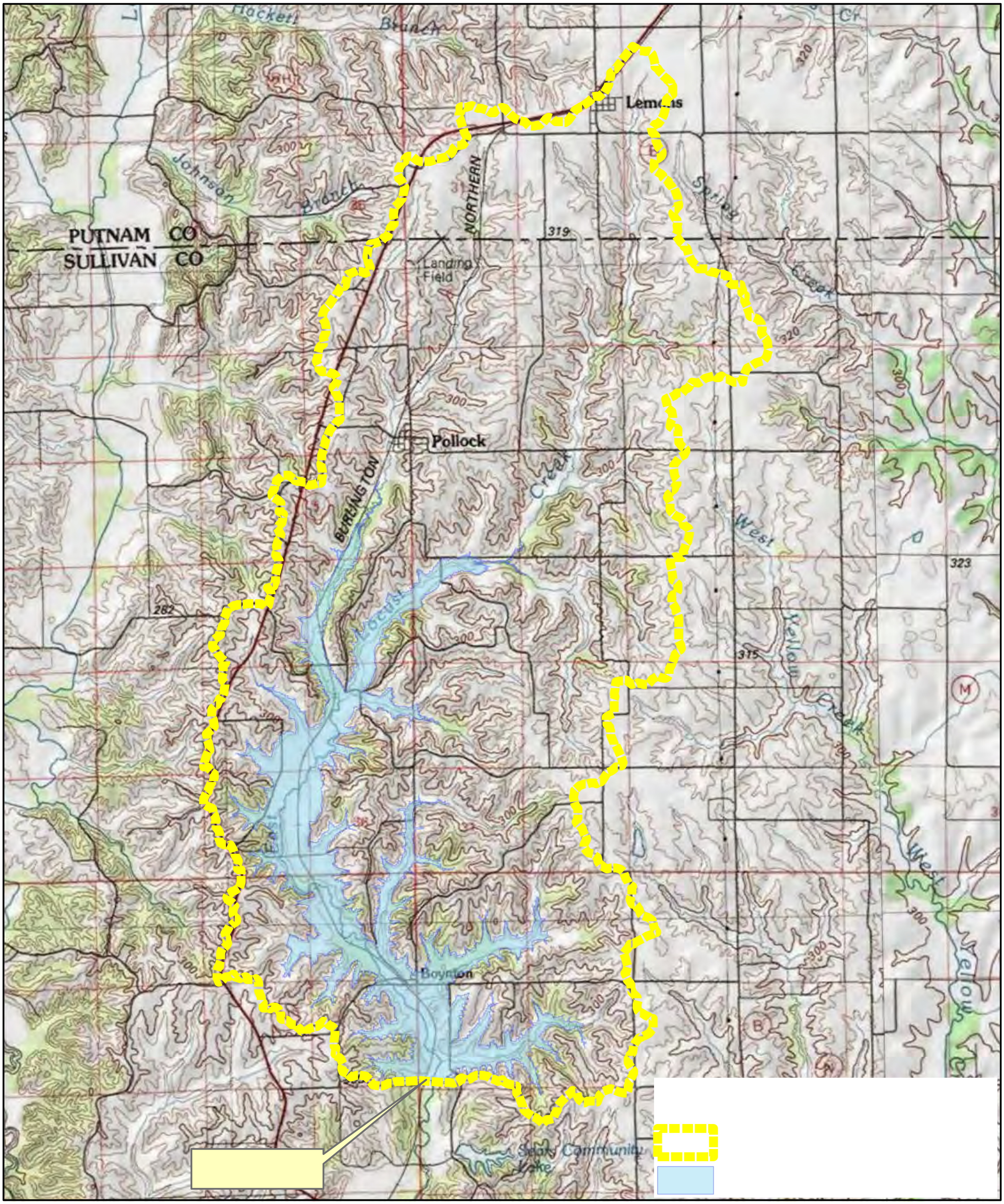

FIGURE 2
Watershed Map

N

Miles
0 0.5 1

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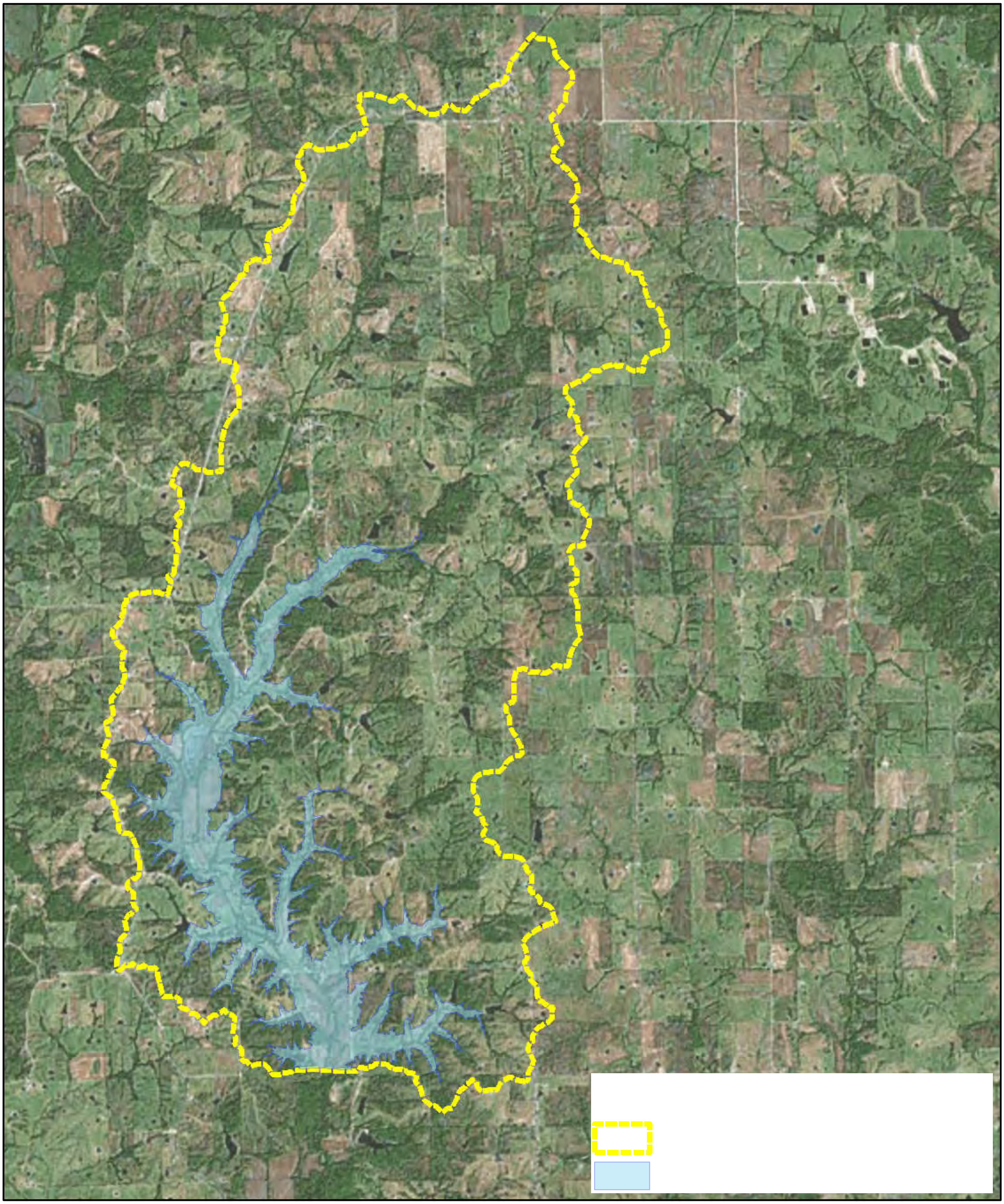

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N

Miles
0 0.5 1

FIGURE 3
Aerial Map

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The watershed and proposed dam were modeled using the United States Department of Agriculture (USDA) SITES program, version 3.5. Per the guidance in the Natural Resources Conservation Service Technical Release 60, “Earth Dams and Reservoirs” issued July, 2005 (TR-60), and the National Engineering Handbook (NEH), Part 630 - Hydrology, USDA NRCS, version 2010, the watershed was analyzed as one watershed and was not broken into smaller watersheds.

Curve Number

To calculate the curve number for the watershed, the hydraulic soil group from the Natural Resources Conservation Service SSURGO Soil Data (accessed 2015) and land use data were compiled. The soil and land use information can be seen in Figures 4 and 5. The curve number tables, located in TR-55, were used to obtain a curve number for each land use and soil group. Table 2 below provides the curve number for the corresponding land use and hydraulic soil group.

Table 2 – Curve Number Table

Land Use Description	**Soil Group and Curve Number**			
	A	B	C	D
Open Water	98	98	98	98
Hay/Pasture	49	69	79	84
Barren Land & Cultivated Crops	77	86	91	94
Roads	98	98	98	98
Farmsteads	59	74	82	86
Mixed Forest	43	65	76	82
Deciduous Forest & Evergreen Forest & Woody Wetlands	36	60	73	79
Shrub/Scrub	35	56	70	77
Developed, Open Space	39	61	74	80
Developed, Low Intensity	68	79	86	89
Emergent Herbaceous Wetlands	48	67	77	83

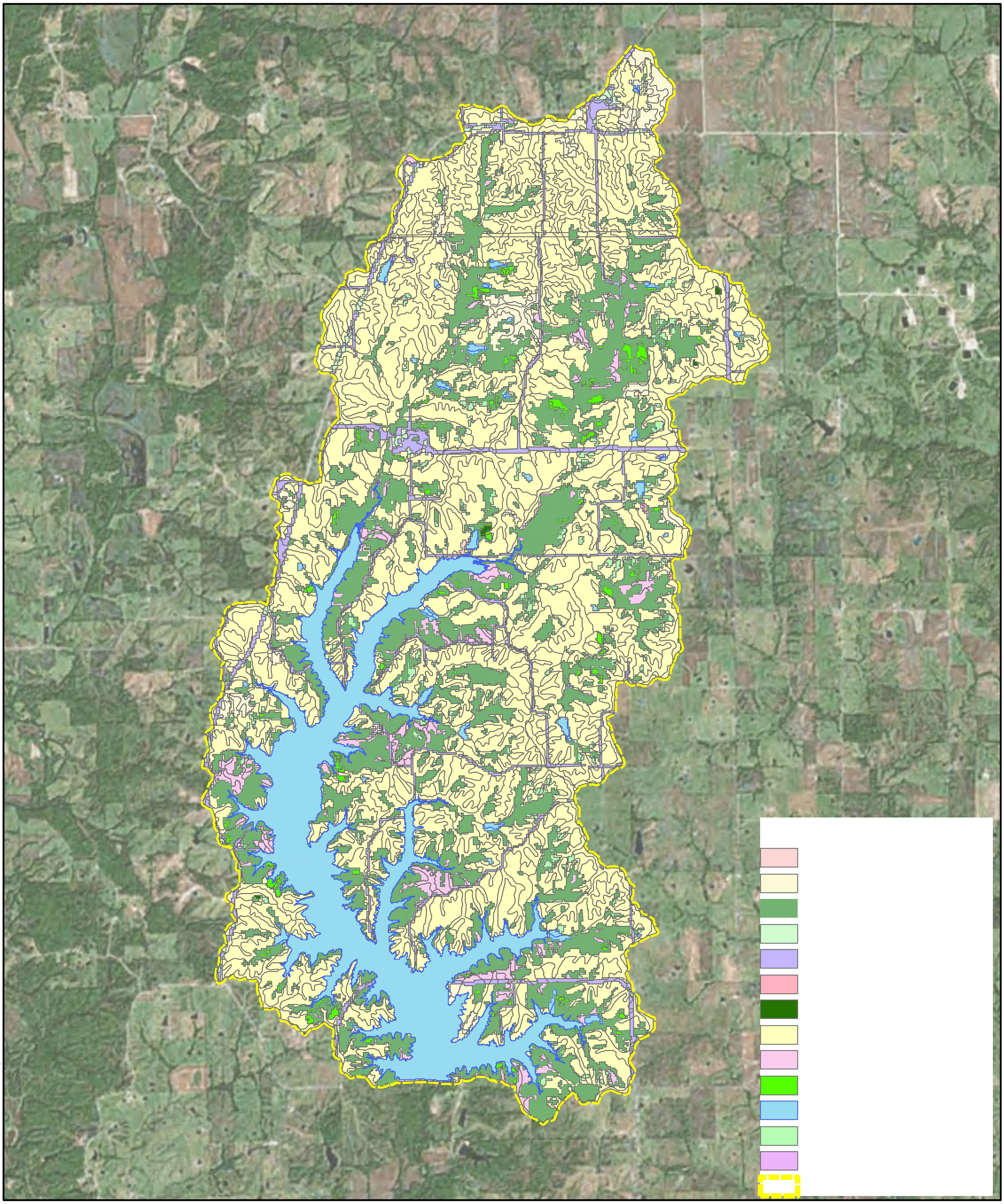

FIGURE 5
Landuse Map

Feet
0 5,000 10,000

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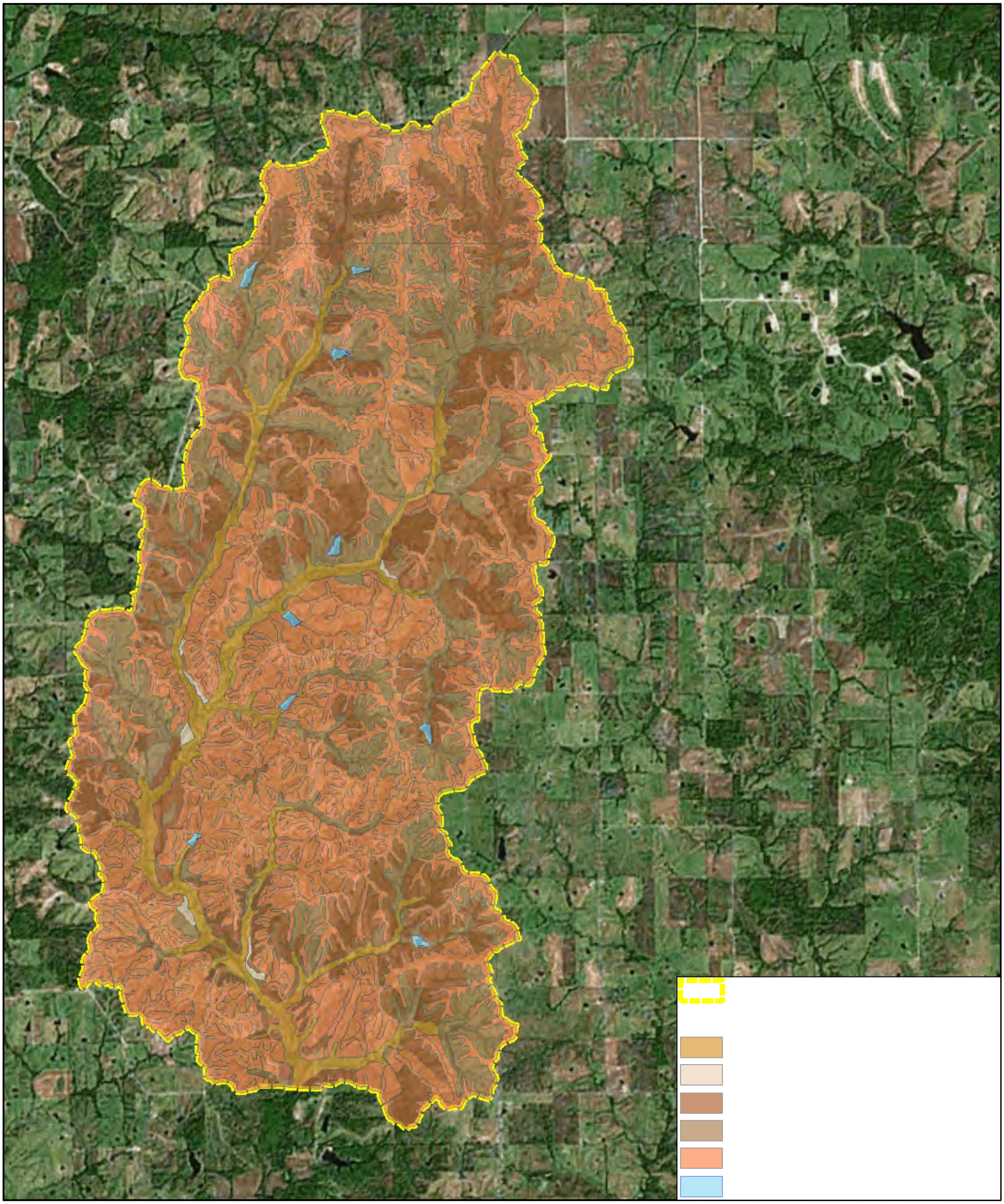

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FIGURE 4
Soil Survey Map

N

Miles
0 0.5 1

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A weighted curve number was calculated for the watershed and input into SITES. The weighted curve number for the watershed draining to the proposed reservoir is 82.1.

Climate Area Zone

Climate Zone 2 was selected for the watershed. Climate Zone 2 corresponds to a humid and sub-humid climate.

Time of Concentration

The velocity method in the NEH was selected for calculating the time of concentration. The velocity method uses different overland flow types to compute the time of concentration. The flow type at the top of the watershed is sheet flow and is limited to the first 300 feet on the upstream side of the watershed. The travel time for sheet flow is calculated with the modified Manning's equation provided in the NEH. The shallow concentrated flow occurs after sheet flow and represents the runoff collecting in swales or gullies. The tables in the NEH were used to calculate the travel time for shallow concentrated flow. Shallow concentrated flow eventually accumulates and becomes open channel flow. To calculate the travel time for the open channel flow, a representative cross section was taken from the LiDAR and a velocity for the cross section was calculated using Manning's equation. The flow path for the watershed included flow through the proposed reservoir. The wave velocity equation (NEH eq. 15-11) was used to calculate the travel time through the proposed reservoir. The cumulative time of concentration for the watershed is the sum of the travel times computed above, or 3.68 hours. The flow path and types of flow for the watershed can be seen in Figure 6.

Rainfall Data

Design Storm Simulation

The SCS curve number method requires the input of rainfall total depths to calculate a peak runoff value. The National Oceanic and Atmospheric Admiration (NOAA) Atlas 14 Precipitation-Frequency Atlas of the United States, Volume 8, Version 2.0 (2013) was used for the 24-hour rainfall depths for SITES modeling. The 24-hour rainfall depths are used to size the principal

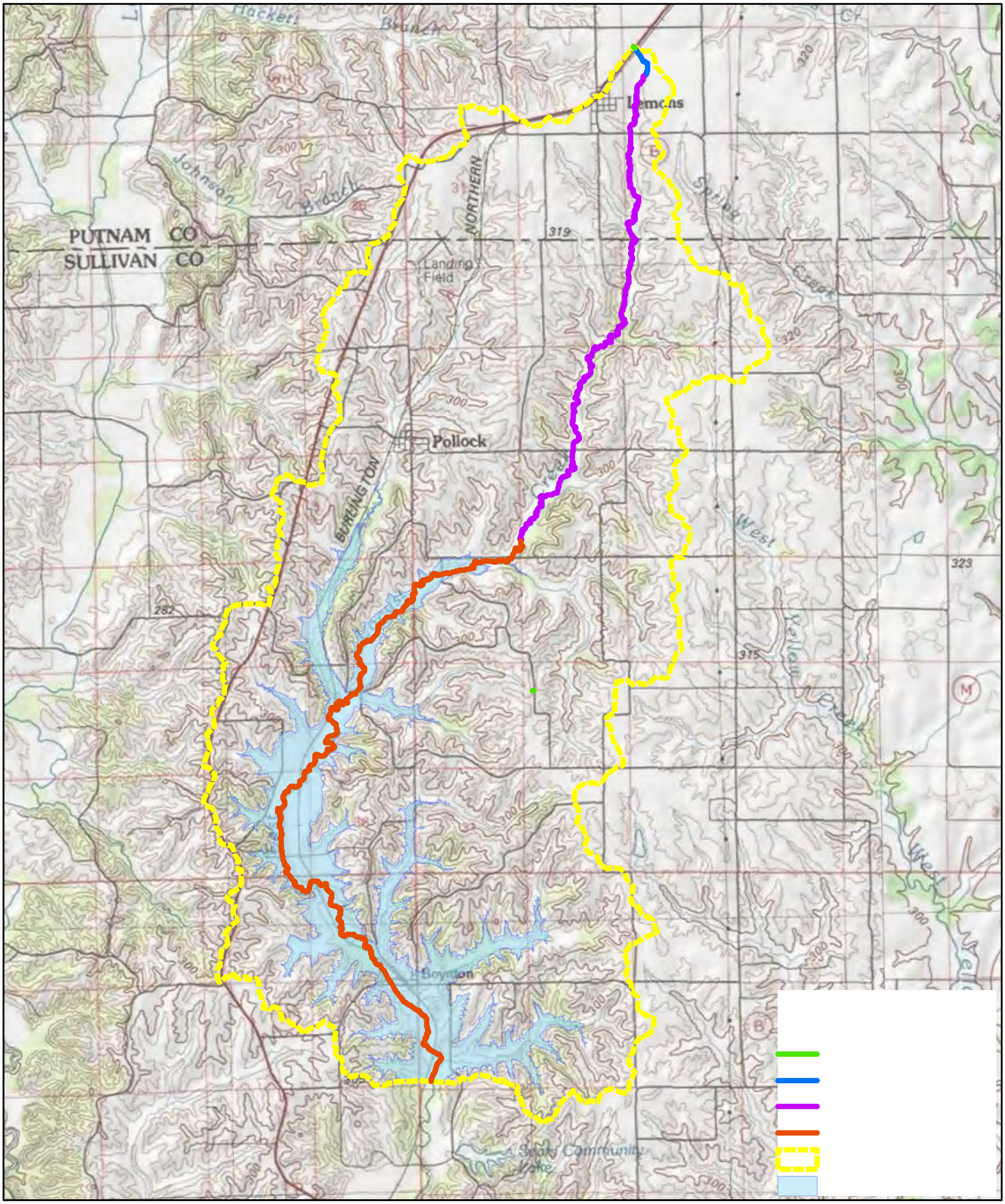

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FIGURE 6
Time of Concentration and Flow Type

N

Miles
0 0.5 1

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spillway, and determine the elevation for the auxiliary spillway. The rainfall data from the station closest to the reservoir at Milan, Missouri was used for the SITES analysis. The rainfall depths for the analysis were areally adjusted by 0.959 for the 32.6 square miles drainage area per Table 2-3 in TR-60. Table 3 provides the adjusted rainfall depth for each return period analyzed.

Table 3 – Areally Adjusted 24-hr Rainfall Depths

Return Period	Adjusted Rainfall Depth (in)
1%	7.74
2%	6.75
10%	4.75
50%	3.23

The values from the SITES model with the parameters estimated for the contributing watershed are below in Table 4.

Table 4. SITES Peak Runoff Rates

Return Period	SITES Output (cfs)
1%	24,360
2%	20,393
10%	12,416
50%	6,649

Probable Maximum Precipitation

TR-60 was utilized to determine the design storm for the principal spillway, auxiliary spillway, and freeboard for the proposed dam. The proposed reservoir is classified as a high hazard dam, per TR-60. To determine the size of the auxiliary spillway and the freeboard required from the auxiliary spillway to the top of the dam, the Stability Design Hydrograph (SDH) and Freeboard Hydrograph (FBH) were used. Routing the SDH and FBH require the input of the Probable Maximum Precipitation (PMP) rainfall depth. The PMP depths for the proposed reservoir were estimated from Hydrometeorological Report No. 51 (HMR – 51) published by NOAA, 1978. PMP rainfall depths for the 6-hour, 12-hour, and 24-hour durations were estimated from HMR – 51. The SDH value was computed from the equation SDH = P_{100}+0.26(PMP-P_{100}) and the FBH runoff depth is equal to the PMP rainfall depth for high hazard dams. Table 5 shows the rainfall depths for evaluating the auxiliary spillway and the required freeboard for the top of the dam.

Table 5 – Rainfall Depths for SDH and FBH Hydrographs

Storm Duration (hr)	PMP (in)	SDH (in)	FBH (in)
6	26.9	11.63	26.9
12	31.8	NA	31.8
24	33.4	14.66	33.4

Hydrometeorological Report No. 52 (HMR – 52), published by NOAA in 1982, was used to route the FBH storm. The 24-hour storm "5-point" rainfall distribution mass curve was used for the FBH modeling. The 5-point rainfall distribution method divides the 24-hour distribution into four quadrants with 5 points. The values in the 5 point distribution are calculated as a percentage of the total rainfall for the 6-hour and 24-hour storms. The first point in the distribution is 0, the second point is calculated by the equation (24_{HR-PMP} - 6_{HR-PMP})/(24_{HR-PMP} /2), the third point is calculated by (6_{HR-PMP})/(24_{HR-PMP}), the fourth point is the (12_{HR-PMP} - 6_{HR-PMP})/ 24_{HR-PMP}, the fifth point is the remainder of the rainfall for the storm.

Calibration Approach

The primary hydrologic method of analysis for the watershed is the Curve Number method using the SITES modeling software. To determine if the SITES model results represent the actual runoff rates for the contributing drainage area with reasonable accuracy, the SITES model output was compared to gages in the same region, rural regression equations, and sub watersheds within the contributing drainage area of the proposed reservoir.

Gage Data

Historical data for the contributing watershed was not available. However several gages near the area are available and have similar land use to the East Locust Creek watershed. To determine the flood flow frequency from the gage data for the return periods required for the design of the reservoir, the Corps of Engineers HEC-SSP ver. 2.0 software was utilized. HEC-SSP uses Bulletin 17B “Guidelines for Determining Flood Flow Frequency” (1982) as the basis for analysis. The flood flow frequency was calculated from the available data for seven gage stations. The location of the gage stations can be seen in Figure 7. The flood flow frequency for each station was weighted based on guidance provided in “The National Streamflow Statistics Program: Chapter 6 of Book 4, Hydrologic Analysis and Interpretation, Section A - Statistical Analysis” Published by U.S. Department of the Interior and U.S. Geological Survey. The recommendation for weighting the gage values uses the exponents from the Regression equations for the state of Missouri. The below values were weighted according to the procedure in the publication. Table 6 shows the comparison of the peak flood flow rates computed from gage stations to the values from the SITES model.

Table 6 - Weighted Gage Runoff Data

			Return Period			
		Drainage Area (mi^2)	1%	2%	10%	50%
Gage Station Locations	Linneus	550	4,829	4,359	3,103	1,465
	Reger	232	5,949	4,926	2,842	1,088
	Atlanta	23	12,594	10,188	5,457	1,787
	Chula	72	3,642	3,345	2,603	1,664
	Bethany	95	5,039	4,322	2,757	1,247
	Mendon	14	4,826	4,659	3,877	1,924
	Bedford	85	5,592	5,387	4,612	2,827
	SITES Output	32.6	24,360	20,393	12,416	6,649
	SITES Output No Lake	32.6	12,285	10,161	6,006	3,096

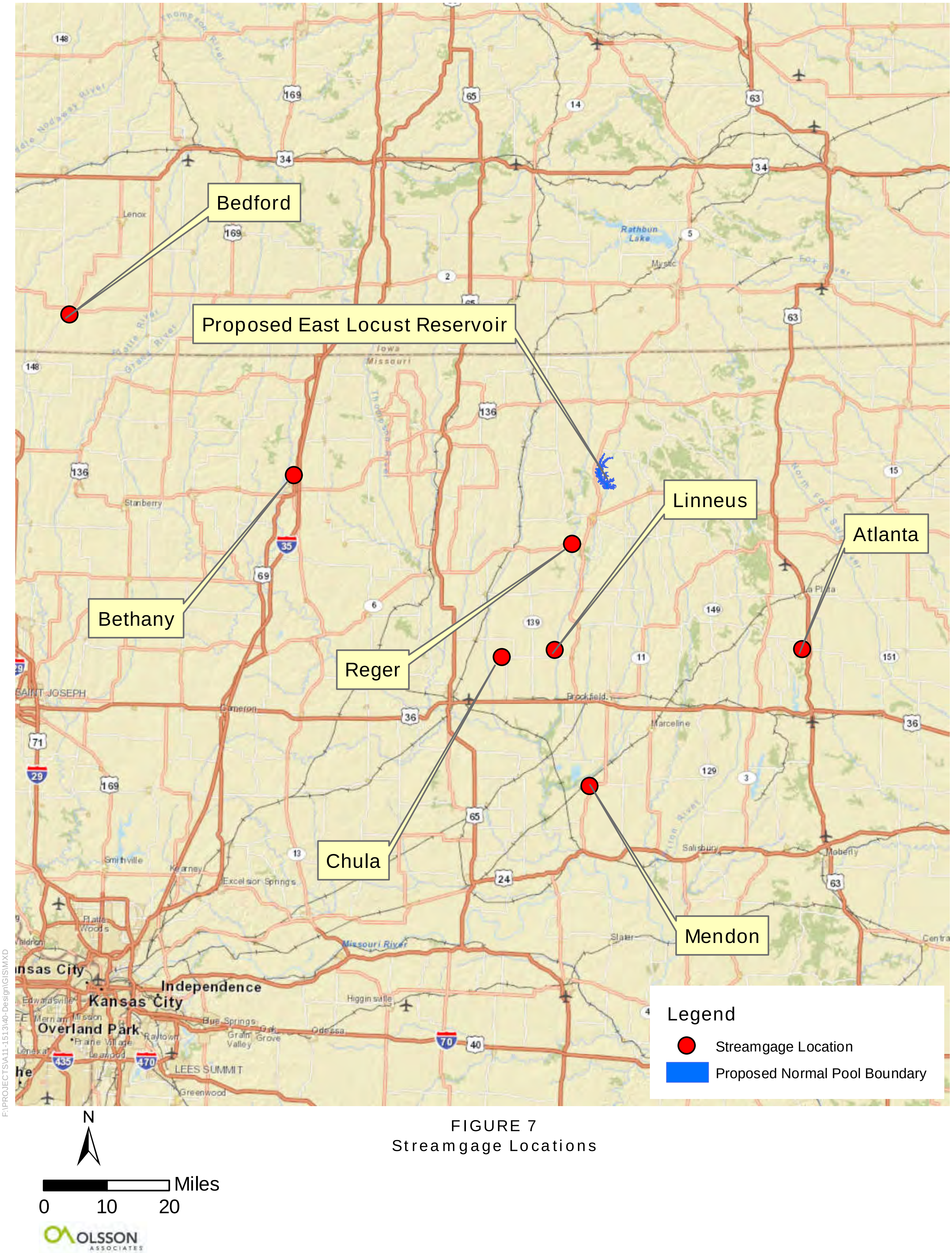

FIGURE 7
Streamgage Locations

To compare the runoff rate for the proposed basin to the gage data, runoff rates were computed for the contributing watershed without the proposed reservoir. The curve number and time of concentration were computed using the same procedures used for the proposed conditions watershed, as described above. The curve number for the watershed without the reservoir is 79 and the time of concentration is 8.53 hours.

The National Streamflow Statistics Program also recommends limiting comparisons of gage data to locations that are between 0.5 and 1.5 times the drainage area of the proposed reservoir. The gages near Atlanta and Mendon have a ratio to the proposed reservoir of 0.71 and 0.43 respectively. The other gages all have a ratio above 2.2.

The shape of a watershed can also have an impact on the runoff rates for a given watershed. To determine the approximate length to width ratio of the watersheds, the centroid of the watershed was found and the lengths of lines passing through the centroid along the flow path and perpendicular to the flow path were calculated. The length to width ratios for those watersheds that are close in shape and size to the contributing watershed, are shown in Table 7.

Table 7 – Length to Width Ratio

	Length to Width Ratio
Atlanta	6.0
Mendon	2.2
Proposed Reservoir	2.6

The higher length to width ratio of the gage near Atlanta could be part of the reason that the flows at the Atlanta gage are higher than other weighted gages. The Mendon gage has a length to width ratio closer to that of the contributing drainage area of the proposed reservoir; however, the flow rates computed for the contributing drainage area (with no lake) are 2.5 times those

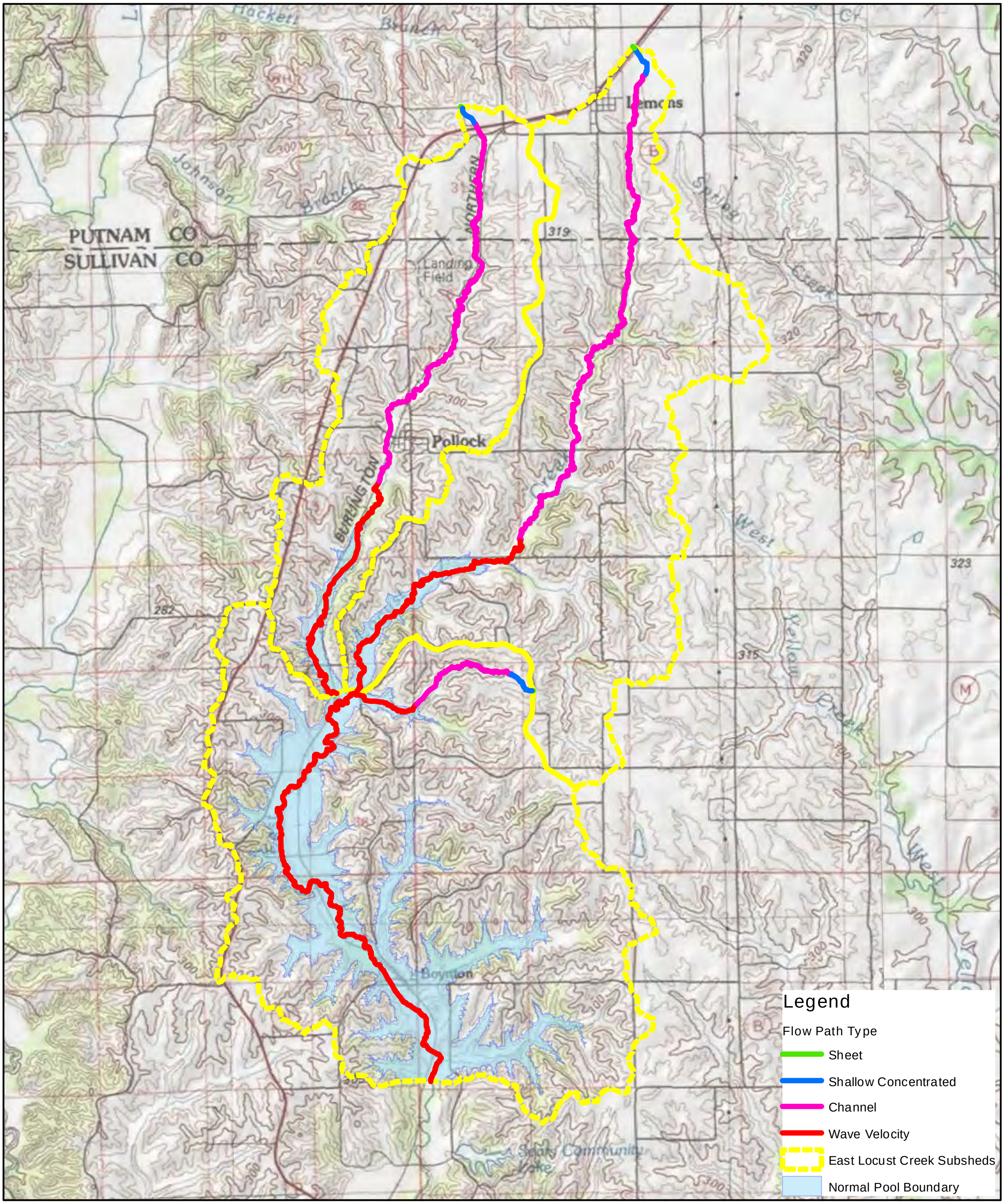

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FIGURE 8
HEC-HMS Watersheds and Flow Types

Miles
0 0.5 1

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derived from the gaged data. A detailed study of the gaged watersheds was not completed for this study and many factors can contribute to lower runoff rates for an individual watershed.

Regional Regression Equations

Regional regression equations were also used to compute runoff rates which were compared to the runoff rates from the SITES model. The USGS "Techniques for Estimating the 2-to 500-Year Flood Discharges on Unregulated Streams in Rural Missouri" (1995) was used for the regional regression equations. The regional regression equations were generated using the least-squares regression technique. Runoff rates from 278 selected streamflow-gaging stations were compared to basin characteristics in order to determine which basin characteristics were statistically significant (USGS, 1995).The equations for Region 1 were used for estimating the peak flows. The regional regression equations require the area and slope of the watershed. The slope of the watershed between the 10% and 85% point on the watershed is 9.81 ft/mile (URS, 2014).

Table 8 - Regional Regression Equations Peak Runoff

Return Period	Regional Regression Equations (cfs)	SITES Output (cfs)
1%	8,043	24,360
2%	6,863	20,393
10%	4,313	12,416
50%	1,886	6,649

HEC-HMS Model Development

A U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) version 3.5 model was developed. Computed peak runoff rates from HEC-HMS were also compared to the calculated values from SITES. The curve number method was also used in the model development. For the development of the HEC-HMS model, the contributing watershed of the proposed reservoir was broken into 3 sub-watersheds, Figure 8.

The same techniques of computing the curve number, time of concentration, and rainfall depth that were used for the SITES model were used for the HEC-HMS model. The HEC-HMS model includes routing between the watersheds that was not included in the SITES model. The Lag method was used for routing within the watershed to the proposed reservoir. The lag time in the watershed was calculated using a representative cross section from the LiDAR information, a velocity was calculated using Manning’s equation. The velocity was used to compute a lag time for the channel between the sub-areas.

Table 9 - HEC-HMS Peak Flows

Return Period	HEC-HMS (cfs)	SITES Output (cfs)
1%	31,327	24,360
2%	26,192	20,393
10%	15,982	12,416
50%	8,599	6,649

TR-60 guidance recommends that that highest value calculated for the watershed be used in the analysis of the dam and spillway. The HEC-HMS output is significantly higher than both the gage data and the rural regression equations. The TR-60 guidance does not recommend dividing the watershed into subareas unless the drainage area is over 50 square miles. While the HEC-HMS flows are higher than then SITES output the land use in the contributing watershed for the proposed reservoir does not differ significantly and does not warrant dividing the watershed into subareas. Therefore, the SITES output values will be used for the design and analysis of the proposed spillway and dam.

Appendix A

Hydrologic Parameters

East Locust - Time of Concentration Estimation

Time of Concentration (Tc) = $T_{sheet} + T_{shallow\ conc} + T_{channel} + T_{wave\ velocity}$

Reference: National Engineering Handbook, Part 630 Hydrology, Chapter 15, USDA NRCS, May 2010

T_{sheet}

$$T_t = \frac{0.007(n\ell)^{0.8}}{(P_2)^{0.5}\, S^{0.4}} \qquad \text{(eq. 15–8)}$$

where:
T_t = travel time, h
n = Manning's roughness coefficient (table 15–1)
ℓ = sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
S = slope of land surface, ft/ft

n =	0.17	
L =	300	ft
P_2 =	3.3	in
Begin elev =	1076.97	ft
End elev =	1075.52	ft
s =	0.005	ft/ft
T_{sheet} =	**0.755**	hr

$T_{shallow}$

Velocity =

v = 6.962*(slope)$^{0.5}$
(for short grass pasture)

(Table 15-3 and Figure 15-4)
(NEH Part 360 Hydrology, Chapter 15)

$T_{shallow}$ =	L (ft)/v (ft) * 1/3600	
Begin elev =	1075.52	ft
End elev =	1062.99	ft
s =	0.00931	ft/ft
v =	0.67	ft/s
$L_{shallow}$ =	1346.47	ft
$T_{shallow}$ =	**0.557**	hr

Travel time (T_t) is the time it takes water to travel from one location to another. Travel time between two points is determined using the following relationship:

$$T_t = \frac{\ell}{3{,}600V} \qquad \text{(eq. 15–1)}$$

where:
T_t = travel time, h
ℓ = distance between the two points under consideration, ft
V = average velocity of flow between the two points, ft/s
3,600 = conversion factor, s to h

The Total Time of Concentration is the sum of all travel times:

(b) Velocity method

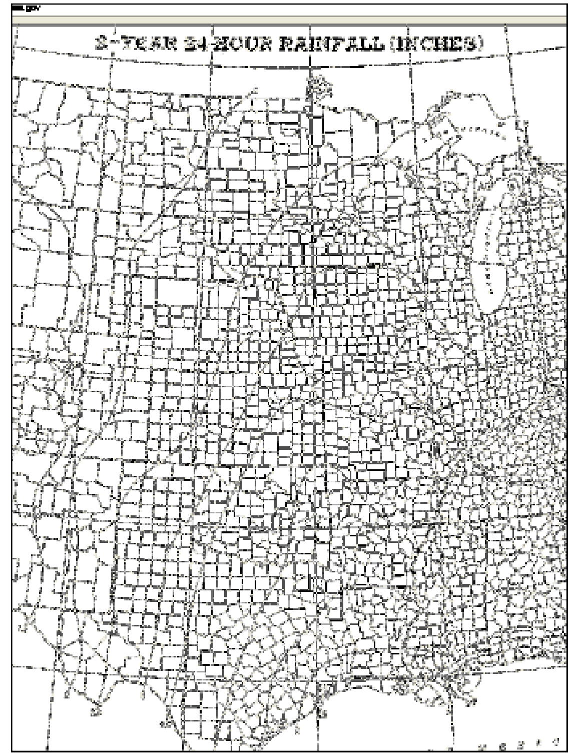
Another method for determining time of concentration normally used within the NRCS is called the velocity method. The velocity method assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path.

$$T_c = T_{t1} + T_{t2} + T_{t3} + \ldots T_{tn} \qquad \text{(eq. 15–7)}$$

where:
T_c = time of concentration, h
T_{tn} = travel time of a segment n, h
n = number of segments comprising the total hydraulic length

The segments used in the velocity method may be of three types: sheet flow, shallow concentrated flow, and open channel flow.

P_2 Source for T_{sheet}:
http://www.nws.noaa.gov/oh/hdsc/PF_documents/TechnicalPaper_No40.pdf



East Locust - Time of Concentration Estimation (Continued)

$T_{channel(1)}$		
$T_{channel}$ =	L (ft)/v (ft) * 1/3600	
Begin elev =	1062.99	ft
End elev =	922.30	ft
XS Flow Area =	155.35	ft^2
Wetted Perimeter =	56.64	ft
Hydraulic Radius =	2.74	ft
Manning's n =	0.040	
s =	0.00432	ft/ft
v =	4.78	ft/s
$L_{channel}$ =	32558	ft
$T_{channel}$ =	**1.892**	hr

$T_{wave\ velocity}$

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

$T_{wave\ velocity}$ =	L (ft)/v (ft) * 1/3600	
Volume at elev. 922.3 =	54009	ac-ft
Surface Area at elev. 922.3 =	2331	acres
D_m (Mean Depth) =	23.2	ft
v_w =	27.3	ft/s
$L_{channel}$ =	46940	ft
$T_{channel}$ =	**0.477**	hr

Tc =	**3.68 hr**

Velocity for Channel Flow use Mannning's Eq:

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{(eq. 15–10)}$$

where:
V = average velocity, ft/s
r = hydraulic radius, ft
$= \frac{a}{P_w}$
a = cross-sectional flow area, ft^2
P_w = wetted perimeter, ft
s = slope of the hydraulic grade line (channel slope), ft/ft
n = Manning's *n* value for open channel flow

Manning's *n* values for open channel flow can be obtained from standard hydraulics textbooks, such as Chow (1959), and Linsley, Kohler, and Paulhus (1982). Publications dealing specifically with Manning's *n* values are Barnes (1967); Arcement and Schneider (1989); Phillips and Ingersoll (1998); and Cowen (1956). For guidance on calculating Manning's *n* values, see NEH630.14, Stage Discharge Relations.

Wave Velocity through Dam Backwater

In other cases, such as with a watershed having a relatively large body of water in the flow path, time of concentration is computed to the upstream end of the water body using standard methods, and velocity for the flow segment through the water body may be computed using the wave velocity equation coupled with equation 15–1 to convert the velocity to a travel time through the water body. The wave equation is:

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

Generally, V_w will be high; however, equation 15–11 only provides for estimating travel time through the water body and for the inflow hydrograph to reach the outlet. It does not account for the time required for the

East Locust - Time of Concentration Estimation (Continued)

Figure 15–4 Velocity versus slope for shallow concentrated flow

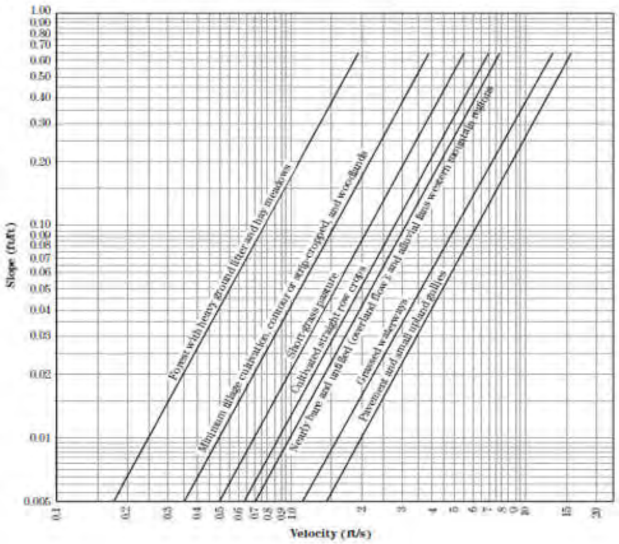


Table 15–3 Equations and assumptions developed from figure 15–4

Flow type	Depth (ft)	Manning's *n*	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V = 20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V = 16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V = 9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V = 8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V = 6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V = 5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V = 2.516(s)^{0.5}$

United States Department of Agriculture

Natural Resources Conservation Service

Conservation Engineering Division

Technical Release 55

June 1986

Urban Hydrology for Small Watersheds

TR-55

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click the bookmarks tab. It will navigate you to the contents,

chapters, rainfall maps, and printable forms.

Table 2-2a Runoff curve numbers for urban areas [1/]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2/]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3/]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4/]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5/]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2b Runoff curve numbers for cultivated agricultural lands [1]

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment [2]	Hydrologic condition [3]	A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+ CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+ CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition, and I_a=0.2S
2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 2-2c Runoff curve numbers for other agricultural lands [1/]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2/]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3/]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4/]	48	65	73
Woods—grass combination (orchard or tree farm). [5/]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6/]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4/]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.
2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.
3 *Poor.* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
4 Actual curve number is less than 30; use CN = 30 for runoff computations.
5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

NOAA Atlas 14, Volume 8, Version 2 MILAN
Station ID: 23-5578
Location name: Milan, Missouri, US*
Latitude: 40.2211°, Longitude: -93.1097°
Elevation:
Elevation (station metadata): 840 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, Geoffery Bonnin

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.398** (0.315–0.514)	**0.461** (0.364–0.595)	**0.568** (0.447–0.735)	**0.662** (0.517–0.858)	**0.798** (0.604–1.06)	**0.908** (0.669–1.21)	**1.02** (0.727–1.39)	**1.14** (0.777–1.57)	**1.31** (0.855–1.83)	**1.44** (0.913–2.03)
10-min	**0.583** (0.461–0.753)	**0.675** (0.533–0.871)	**0.832** (0.655–1.08)	**0.969** (0.758–1.26)	**1.17** (0.884–1.55)	**1.33** (0.980–1.78)	**1.50** (1.06–2.03)	**1.67** (1.14–2.30)	**1.92** (1.25–2.68)	**2.11** (1.34–2.97)
15-min	**0.711** (0.562–0.918)	**0.823** (0.650–1.06)	**1.02** (0.798–1.31)	**1.18** (0.924–1.53)	**1.43** (1.08–1.89)	**1.62** (1.20–2.17)	**1.83** (1.30–2.47)	**2.04** (1.39–2.81)	**2.34** (1.53–3.27)	**2.58** (1.63–3.62)
30-min	**0.995** (0.786–1.28)	**1.16** (0.913–1.49)	**1.43** (1.13–1.85)	**1.67** (1.31–2.17)	**2.02** (1.53–2.68)	**2.29** (1.69–3.06)	**2.58** (1.84–3.50)	**2.89** (1.96–3.97)	**3.30** (2.16–4.61)	**3.63** (2.30–5.10)
60-min	**1.31** (1.04–1.70)	**1.53** (1.21–1.98)	**1.91** (1.50–2.47)	**2.24** (1.75–2.90)	**2.73** (2.07–3.63)	**3.12** (2.30–4.17)	**3.53** (2.51–4.79)	**3.97** (2.70–5.46)	**4.58** (2.99–6.40)	**5.06** (3.20–7.10)
2-hr	**1.63** (1.31–2.08)	**1.91** (1.53–2.43)	**2.39** (1.90–3.05)	**2.81** (2.22–3.60)	**3.43** (2.64–4.52)	**3.94** (2.95–5.22)	**4.48** (3.22–6.02)	**5.05** (3.48–6.90)	**5.85** (3.86–8.12)	**6.48** (4.15–9.04)
3-hr	**1.85** (1.49–2.34)	**2.16** (1.74–2.73)	**2.71** (2.17–3.43)	**3.20** (2.55–4.06)	**3.93** (3.04–5.15)	**4.53** (3.41–5.97)	**5.17** (3.75–6.91)	**5.85** (4.06–7.96)	**6.82** (4.53–9.43)	**7.59** (4.89–10.5)
6-hr	**2.21** (1.80–2.76)	**2.58** (2.10–3.21)	**3.23** (2.62–4.03)	**3.82** (3.08–4.79)	**4.72** (3.70–6.13)	**5.47** (4.17–7.14)	**6.27** (4.61–8.32)	**7.14** (5.02–9.64)	**8.38** (5.64–11.5)	**9.38** (6.11–12.9)
12-hr	**2.56** (2.11–3.14)	**2.96** (2.44–3.64)	**3.69** (3.04–4.55)	**4.37** (3.57–5.39)	**5.39** (4.29–6.93)	**6.26** (4.84–8.09)	**7.19** (5.35–9.46)	**8.21** (5.84–11.0)	**9.66** (6.58–13.2)	**10.8** (7.15–14.8)
24-hr	**2.91** (2.44–3.53)	**3.37** (2.82–4.09)	**4.20** (3.49–5.10)	**4.95** (4.10–6.03)	**6.09** (4.90–7.72)	**7.04** (5.51–8.99)	**8.07** (6.08–10.5)	**9.19** (6.61–12.2)	**10.8** (7.42–14.6)	**12.1** (8.03–16.4)
2-day	**3.33** (2.82–3.98)	**3.88** (3.28–4.64)	**4.83** (4.08–5.79)	**5.68** (4.76–6.83)	**6.93** (5.63–8.64)	**7.96** (6.29–10.0)	**9.04** (6.88–11.6)	**10.2** (7.41–13.4)	**11.8** (8.23–15.8)	**13.1** (8.84–17.7)
3-day	**3.65** (3.11–4.32)	**4.24** (3.62–5.02)	**5.26** (4.47–6.25)	**6.15** (5.19–7.33)	**7.45** (6.09–9.20)	**8.51** (6.77–10.6)	**9.61** (7.36–12.2)	**10.8** (7.88–14.0)	**12.4** (8.69–16.5)	**13.7** (9.30–18.4)
4-day	**3.92** (3.37–4.62)	**4.54** (3.90–5.35)	**5.60** (4.78–6.61)	**6.52** (5.54–7.73)	**7.85** (6.45–9.63)	**8.92** (7.13–11.1)	**10.0** (7.73–12.7)	**11.2** (8.24–14.5)	**12.9** (9.04–17.1)	**14.1** (9.65–19.0)
7-day	**4.65** (4.03–5.41)	**5.32** (4.61–6.19)	**6.45** (5.57–7.53)	**7.43** (6.38–8.71)	**8.84** (7.33–10.7)	**9.97** (8.05–12.2)	**11.1** (8.65–14.0)	**12.4** (9.16–15.9)	**14.0** (9.97–18.5)	**15.4** (10.6–20.5)
10-day	**5.30** (4.63–6.12)	**6.02** (5.26–6.96)	**7.25** (6.30–8.40)	**8.30** (7.17–9.65)	**9.79** (8.17–11.8)	**11.0** (8.92–13.4)	**12.2** (9.54–15.2)	**13.5** (10.1–17.2)	**15.2** (10.9–20.0)	**16.6** (11.5–22.0)
20-day	**7.16** (6.34–8.15)	**8.11** (7.18–9.24)	**9.68** (8.53–11.1)	**11.0** (9.63–12.6)	**12.8** (10.8–15.1)	**14.2** (11.7–17.1)	**15.7** (12.4–19.3)	**17.1** (12.9–21.6)	**19.1** (13.8–24.8)	**20.6** (14.5–27.2)
30-day	**8.70** (7.77–9.81)	**9.86** (8.80–11.1)	**11.8** (10.5–13.3)	**13.3** (11.8–15.2)	**15.5** (13.1–18.1)	**17.1** (14.1–20.3)	**18.7** (14.9–22.8)	**20.3** (15.4–25.5)	**22.5** (16.3–29.0)	**24.1** (17.0–31.6)
45-day	**10.6** (9.58–11.9)	**12.1** (10.9–13.5)	**14.4** (12.9–16.2)	**16.3** (14.5–18.4)	**18.8** (16.1–21.8)	**20.7** (17.3–24.4)	**22.6** (18.1–27.3)	**24.4** (18.7–30.4)	**26.8** (19.6–34.3)	**28.5** (20.3–37.3)
60-day	**12.3** (11.1–13.7)	**14.0** (12.7–15.6)	**16.7** (15.0–18.6)	**18.8** (16.9–21.1)	**21.7** (18.6–25.0)	**23.8** (20.0–27.9)	**25.9** (20.9–31.1)	**27.9** (21.4–34.5)	**30.5** (22.4–38.8)	**32.3** (23.1–42.1)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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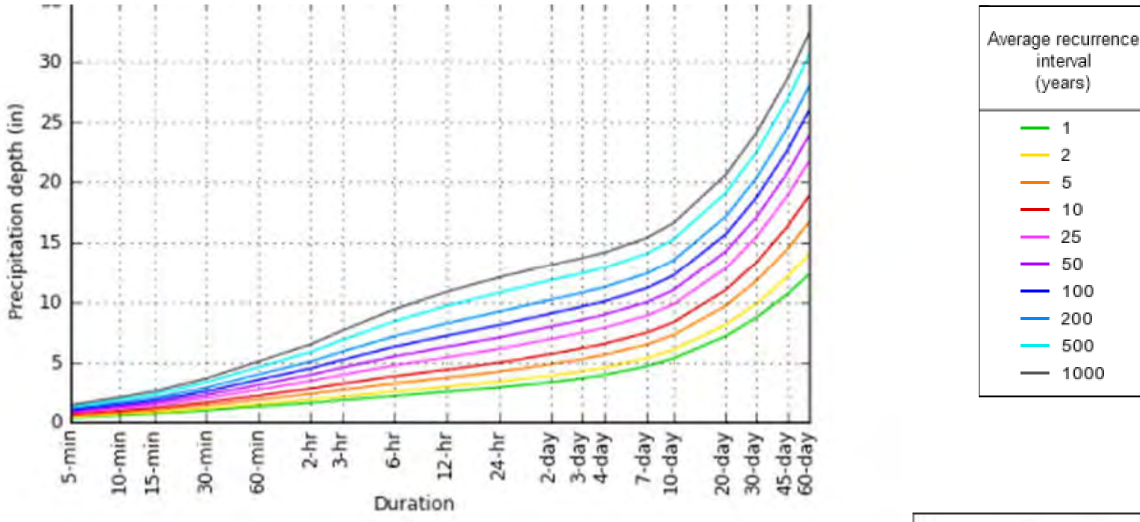
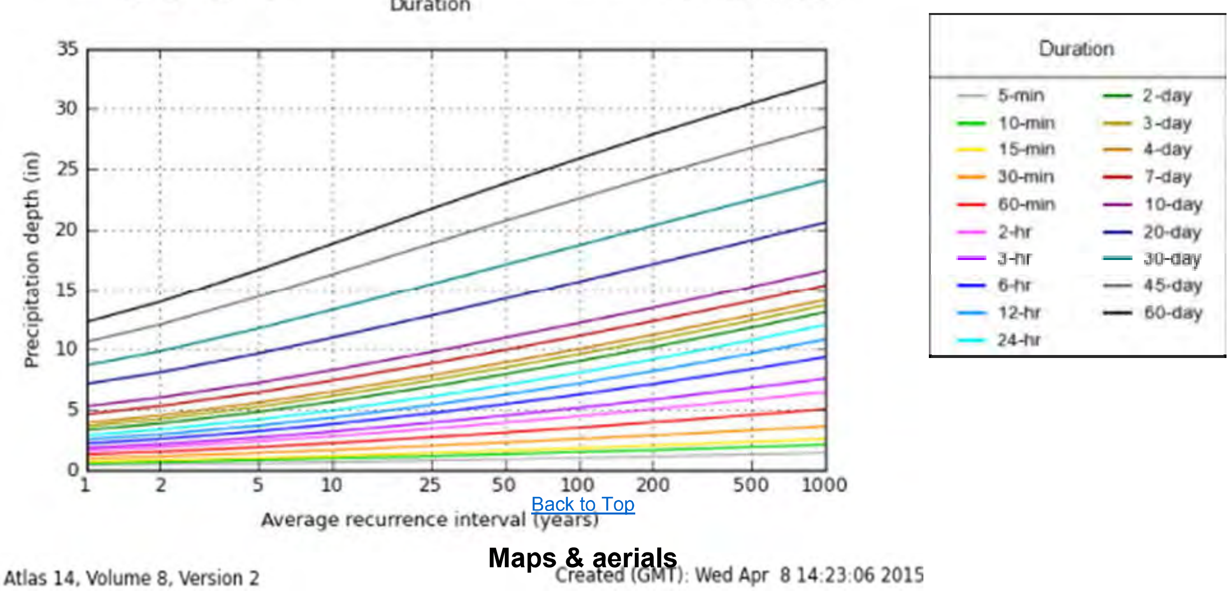
PF graphical





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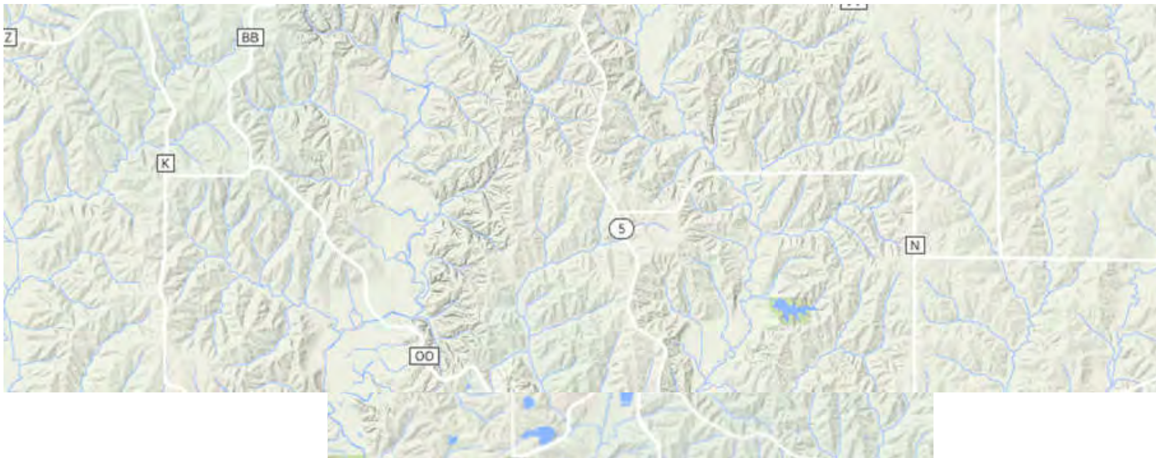
Maps & aerials

Small scale terrain

Large scale terrain



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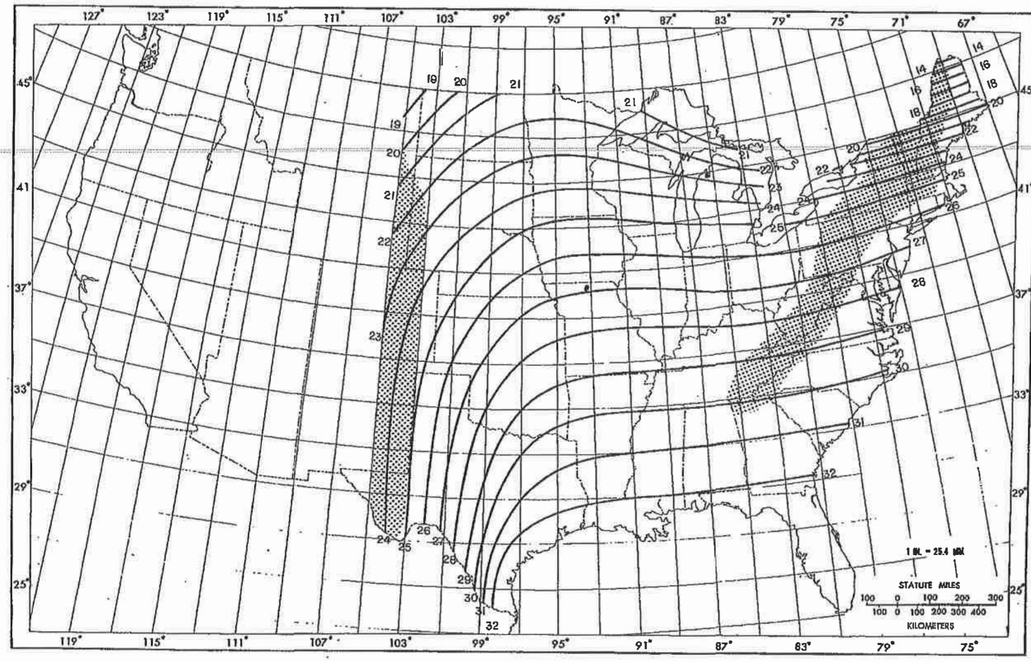
HYDROMETEOROLOGICAL REPORT NO. 51

Missouri - East Locust Creek
RW-1

Probable Maximum Precipitation Estimates, United States East of the 105th Meridian

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
U.S. DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

Washington, D C
June 1978



26.9 inches

Figure 18.--All-season PMP (in.) for 6 hr 10 mi^2 (26 km^2).

31.8 inches

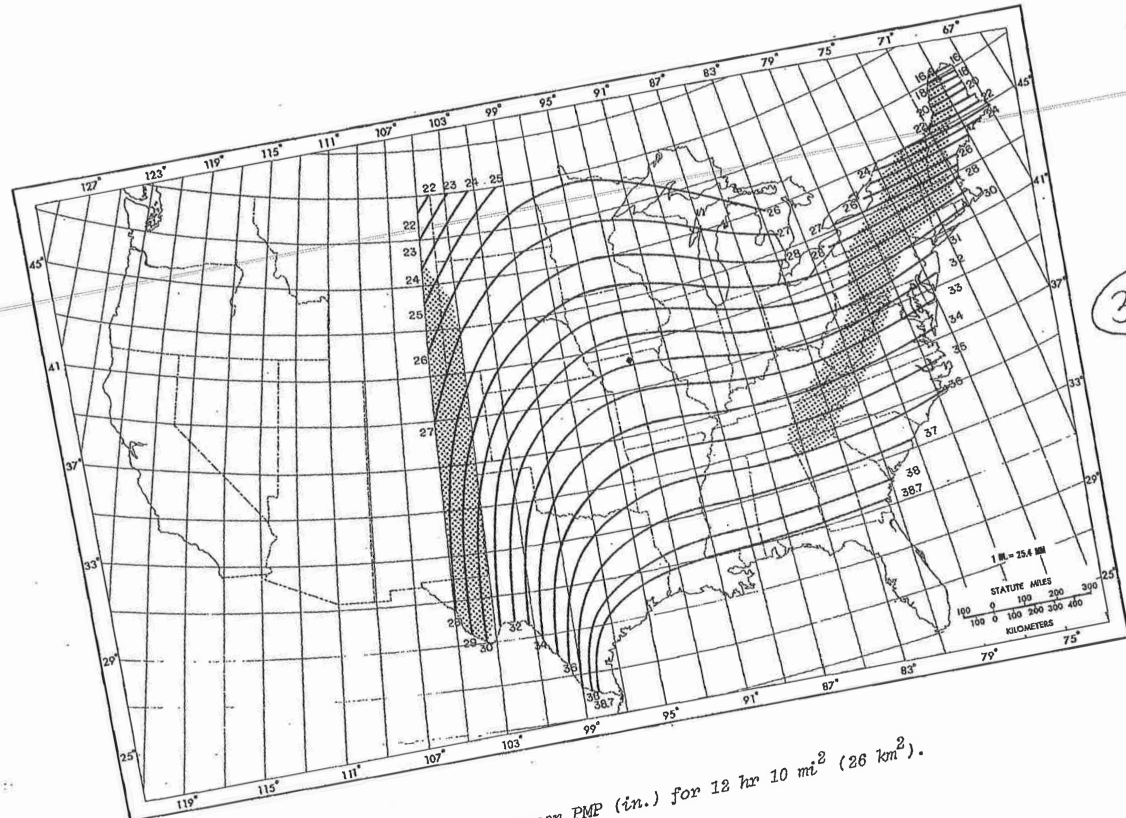
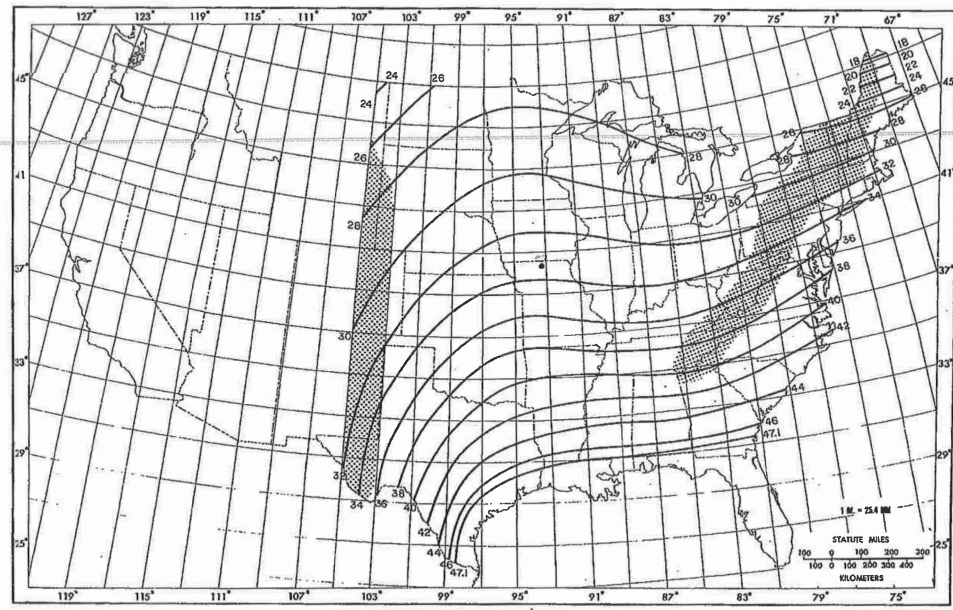


Figure 19.--All-season PMP (in.) for 12 hr 10 mi^2 (26 km^2).



33.4 inches

Figure 20.--All-season PMP (in.) for 24 hr 10 mi^2 (26 km^2).

SITES Output

	RW-1_1YR24HR_Jan2014_SingleStorm
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	2.79
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	N/A
FBH or Storm Inflow Peak (CFS)	5115.5
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	N/A
Maximum WS FBH or Storm (Feet)	923.35
Storage at Max. WS FBH or Storm (Acre-Ft)	56550.7
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	N/A
Hp FBH or Storm (Feet)	1.05
AS Peak Discharge FBH/Storm (CFS)	96
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_2YR24HR_Jan2014_SingleStorm
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	3.23
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	N/A
FBH or Storm Inflow Peak (CFS)	6649.6
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	N/A
Maximum WS FBH or Storm (Feet)	923.59
Storage at Max. WS FBH or Storm (Acre-Ft)	57131.8
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	N/A
Hp FBH or Storm (Feet)	1.29
AS Peak Discharge FBH/Storm (CFS)	118
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_5YR24HR_Jan2014_SingleStorm
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	4.03
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	N/A
FBH or Storm Inflow Peak (CFS)	9617.1
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	N/A
Maximum WS FBH or Storm (Feet)	924.03
Storage at Max. WS FBH or Storm (Acre-Ft)	58223.7
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	N/A
Hp FBH or Storm (Feet)	1.73
AS Peak Discharge FBH/Storm (CFS)	193
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_10YR24HR_Jan2014_SingleStorm
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	4.75
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	N/A
FBH or Storm Inflow Peak (CFS)	12416.2
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	N/A
Maximum WS FBH or Storm (Feet)	924.44
Storage at Max. WS FBH or Storm (Acre-Ft)	59249.4
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	N/A
Hp FBH or Storm (Feet)	2.14
AS Peak Discharge FBH/Storm (CFS)	271
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_25YR24HR_PSNotch_Jan2014_SingleStorm
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	5.84
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	N/A
FBH or Storm Inflow Peak (CFS)	16726.2
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	N/A
Maximum WS FBH or Storm (Feet)	924.9
Storage at Max. WS FBH or Storm (Acre-Ft)	60402.6
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	N/A
Hp FBH or Storm (Feet)	2.6
AS Peak Discharge FBH/Storm (CFS)	767
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_50YR24HR_Jan2014_SingleStorm
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	6.75
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	N/A
FBH or Storm Inflow Peak (CFS)	20392.9
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	N/A
Maximum WS FBH or Storm (Feet)	925.53
Storage at Max. WS FBH or Storm (Acre-Ft)	62036.5
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	N/A
Hp FBH or Storm (Feet)	3.23
AS Peak Discharge FBH/Storm (CFS)	763
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_100YR24HR_Jan2014_SingleStorm
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	7.74
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	N/A
FBH or Storm Inflow Peak (CFS)	24359.7
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	N/A
Maximum WS FBH or Storm (Feet)	925.97
Storage at Max. WS FBH or Storm (Acre-Ft)	63183.1
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	N/A
Hp FBH or Storm (Feet)	3.67
AS Peak Discharge FBH/Storm (CFS)	1161
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_TOD_6-HR_Labyrinth_Jan2014
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	10.48
SDH Rainfall Duration (Hours)	6
FBH or Storm Rainfall Total (Inches)	24.23
FBH or Storm Rainfall Duration (Hours)	6
SDH Inflow Peak (CFS)	40039.9
FBH or Storm Inflow Peak (CFS)	105968
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	927.69
Maximum WS FBH or Storm (Feet)	934.45
Storage at Max. WS FBH or Storm (Acre-Ft)	88092.1
Top Dam (Feet)	934.45
Storage, Top Dam (Acre-Ft)	88093
PS Discharge for SDH (CFS)	0.3
PS Discharge FBH or Storm (CFS)	0.5
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	5.39
Hp FBH or Storm (Feet)	12.15
AS Peak Discharge FBH/Storm (CFS)	9366
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_TOD_24-HR_5PT_Labyrinth_Jan2014
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	13.2
SDH Rainfall Duration (Hours)	24
FBH or Storm Rainfall Total (Inches)	30.08
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	32599.1
FBH or Storm Inflow Peak (CFS)	79352.6
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	928.67
Maximum WS FBH or Storm (Feet)	936.02
Storage at Max. WS FBH or Storm (Acre-Ft)	93327.1
Top Dam (Feet)	936.02
Storage, Top Dam (Acre-Ft)	93328
PS Discharge for SDH (CFS)	0.3
PS Discharge FBH or Storm (CFS)	0.6
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	6.37
Hp FBH or Storm (Feet)	13.72
AS Peak Discharge FBH/Storm (CFS)	10817
Uncontrolled Drainage Area (Sq.Miles)	32.66

SITES Output

	RW-1_TOD_24-HR_Labyrinth_Jan2014
Site Identification	RW1
Watershed Runoff Curve Number	82
Total Watershed Drainage Area (Sq.Miles)	32.66
Watershed Time of Concentration (Hours)	3.68
SDH Rainfall Total (Inches)	13.2
SDH Rainfall Duration (Hours)	24
FBH or Storm Rainfall Total (Inches)	30.08
FBH or Storm Rainfall Duration (Hours)	24
SDH Inflow Peak (CFS)	46595.7
FBH or Storm Inflow Peak (CFS)	114613.6
Initial Reservoir Elevation (Feet)	922.3
Maximum WS SDH (Feet)	928.66
Maximum WS FBH or Storm (Feet)	935.86
Storage at Max. WS FBH or Storm (Acre-Ft)	92789
Top Dam (Feet)	935.86
Storage, Top Dam (Acre-Ft)	92777
PS Discharge for SDH (CFS)	0.3
PS Discharge FBH or Storm (CFS)	0.6
AS Crest (Feet)	922.3
Storage, AS Crest (Acre-Ft)	54009
AS Max. Head SDH (Feet)	6.36
Hp FBH or Storm (Feet)	13.56
AS Peak Discharge FBH/Storm (CFS)	11000
Uncontrolled Drainage Area (Sq.Miles)	32.66

```
-------------------------------
Bulletin 17B Frequency Analysis
    14 Apr 2015   02:16 PM
-------------------------------


--- Input Data ---

Analysis Name: KC_Atlanta
Description:

Data Set Name: KC_Data_other-Atlanta, MO-FLOW-ANNUAL PEAK
DSS File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Locust_Creek.dss
DSS Pathname: /Long Branch Creek/Atlanta, MO/FLOW-ANNUAL
PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Atlanta\KC_Atlanta.rpt
XML File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Atlanta\KC_Atlanta.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---



----------------------
<< Low Outlier Test >>
----------------------
 Based on 19 events, 10 percent outlier test deviate K(N) = 2.361
                          Computed low outlier test value = 155.84

           0 low outlier(s) identified below test value of 155.84



-----------------------
<< High Outlier Test >>
-----------------------
 Based on 19 events, 10 percent outlier test deviate K(N) = 2.361
                      Computed high outlier test value = 11,826.48

       0 high outlier(s) identified above test value of 11,826.48



--- Final Results ---
```

<< Plotting Positions >>
KC_Data_other-Atlanta, MO-FLOW-ANNUAL PEAK

Events Analyzed Day Mon Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Median Plot Pos
27 May 1996	1,860.0	1	2014	5,350.0	3.61
12 Apr 1997	1,180.0	2	2008	4,870.0	8.76
04 Jul 1998	2,590.0	3	2010	4,110.0	13.92
05 Oct 1998	1,970.0	4	2002	3,360.0	19.07
26 Jun 2000	209.0	5	2013	3,070.0	24.23
07 Jun 2001	1,100.0	6	1998	2,590.0	29.38
12 May 2002	3,360.0	7	2009	2,550.0	34.54
05 May 2003	432.0	8	1999	1,970.0	39.69
10 Dec 2003	840.0	9	1996	1,860.0	44.85
12 Apr 2005	641.0	10	1997	1,180.0	50.00
14 Aug 2006	580.0	11	2001	1,100.0	55.15
25 Feb 2007	932.0	12	2007	932.0	60.31
25 Jul 2008	4,870.0	13	2004	840.0	65.46
16 May 2009	2,550.0	14	2011	642.0	70.62
20 Jul 2010	4,110.0	15	2005	641.0	75.77
11 Jun 2011	642.0	16	2006	580.0	80.93
26 Sep 2012	569.0	17	2012	569.0	86.08
18 Apr 2013	3,070.0	18	2003	432.0	91.24
10 Sep 2014	5,350.0	19	2000	209.0	96.39

<< Skew Weighting >>

Based on 19 events, mean-square error of station skew = 0.274
Mean-square error of regional skew = -?

<< Frequency Curve >>
KC_Data_other-Atlanta, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
15,298.7	22,167.6	0.2	41,580.1	8,344.0
12,166.3	16,176.7	0.5	30,563.7	6,916.6
10,032.0	12,567.8	1.0	23,612.7	5,899.3
8,093.3	9,605.6	2.0	17,743.6	4,934.3
5,818.3	6,490.1	5.0	11,484.3	3,734.3
4,305.0	4,613.2	10.0	7,767.7	2,878.1
2,958.7	3,068.7	20.0	4,833.3	2,055.6
1,398.7	1,398.7	50.0	2,009.4	978.4
633.9	607.8	80.0	911.1	389.7
412.0	378.9	90.0	619.2	225.8
286.0	249.4	95.0	452.5	140.4
141.1	103.3	99.0	251.4	54.8

<< Systematic Statistics >>
KC_Data_other-Atlanta, MO-FLOW-ANNUAL PEAK

Log Transform:	

FLOW, CFS		Number of Events	
Mean	3.133	Historic Events	0
Standard Dev	0.398	High Outliers	0
Station Skew	-0.196	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-0.196	Systematic Events	19

--- End of Analytical Frequency Curve ---

```
------------------------------
Bulletin 17B Frequency Analysis
    14 Apr 2015   02:30 PM
------------------------------


--- Input Data ---

Analysis Name: KC_Bedford
Description:

Data Set Name: Bedford IA-Bedford, IA-FLOW-ANNUAL PEAK
DSS File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Locust_Creek.dss
DSS Pathname: /East Fork 102 River/Bedford, IA/FLOW-ANNUAL
PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Bedford\KC_Bedford.rpt
XML File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Bedford\KC_Bedford.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---


--- Preliminary Results ---


<< Skew Weighting >>
-----------------------------------------------------------------
Based on 31 events, mean-square error of station skew =      0.7
Mean-square error of regional skew =                          -?
-----------------------------------------------------------------


<< Frequency Curve >>
Bedford IA-Bedford, IA-FLOW-ANNUAL PEAK
-------------------------------------------------------------------
|  Computed    Expected   |   Percent    |    Confidence Limits    |
|    Curve   Probability |   Chance     |       0.05        0.95  |
|        FLOW, CFS        | Exceedance   |        FLOW, CFS        |
|-------------------------|--------------|-------------------------|
|      9,755.4     9,760.6 |      0.2     |    13,909.7     7,386.8 |
|      9,740.9     9,751.4 |      0.5     |    13,885.5     7,376.7 |
|      9,714.4     9,732.5 |      1.0     |    13,841.1     7,358.2 |
|      9,656.9     9,683.8 |      2.0     |    13,745.1     7,318.1 |
|      9,466.3     9,515.7 |      5.0     |    13,427.7     7,184.7 |
|      9,116.5     9,163.7 |     10.0     |    12,849.6     6,938.6 |
|      8,348.6     8,391.8 |     20.0     |    11,601.3     6,391.7 |
```

5,709.8	5,709.8	50.0	7,555.8	4,428.7
2,612.3	2,521.3	80.0	3,385.9	1,921.4
1,426.7	1,312.1	90.0	1,937.8	945.8
774.8	667.9	95.0	1,134.6	449.7
185.4	121.6	99.0	337.4	75.7

<< Systematic Statistics >>
Bedford IA-Bedford, IA-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.637	Historic Events	0
Standard Dev	0.373	High Outliers	0
Station Skew	-2.118	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-2.118	Systematic Events	31

--- End of Preliminary Results ---

<< Low Outlier Test >>

Based on 31 events, 10 percent outlier test deviate K(N) = 2.577
Computed low outlier test value = 472.58

2 low outlier(s) identified below test value of 472.58

Statistics and frequency curve adjusted for 2 low outlier(s)

<< Systematic Statistics >>
Bedford IA-Bedford, IA-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.716	Historic Events	0
Standard Dev	0.218	High Outliers	0
Station Skew	-1.288	Low Outliers	2
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-2.118	Systematic Events	31

<< High Outlier Test >>

Based on 29 events, 10 percent outlier test deviate K(N) = 2.549
Computed high outlier test value = 18,756.09

0 high outlier(s) identified above test value of 18,756.09

Note: Statistics and frequency curve were modified
using conditional probablity adjustment.

--- Final Results ---

<< Plotting Positions >>
Bedford IA-Bedford, IA-FLOW-ANNUAL PEAK

Events Analyzed Day	Mon	Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Median Plot Pos
15	Jun	1984	4,930.0	1	2010	10,300.0	2.23
25	Jul	1985	1,160.0	2	1986	9,570.0	5.41
14	Jul	1986	9,570.0	3	1993	9,170.0	8.60
12	Jul	1987	5,660.0	4	2007	8,750.0	11.78
08	Dec	1987	328.0	5	2012	8,710.0	14.97
09	Sep	1989	6,740.0	6	2013	7,860.0	18.15
17	Jun	1990	4,150.0	7	2008	7,180.0	21.34
18	Apr	1991	3,140.0	8	2014	7,080.0	24.52
20	Apr	1992	6,380.0	9	1989	6,740.0	27.71
05	Jul	1993	9,170.0	10	1992	6,380.0	30.89
18	Jun	1994	1,730.0	11	1999	6,350.0	34.08
04	Jul	1995	5,880.0	12	2006	6,320.0	37.26
09	May	1996	5,390.0	13	2009	6,060.0	40.45
07	May	1997	4,400.0	14	2011	5,950.0	43.63
30	Mar	1998	4,270.0	15	2004	5,910.0	46.82
31	Jul	1999	6,350.0	16	1995	5,880.0	50.00
25	Jun	2000	3,560.0	17	1987	5,660.0	53.18
10	May	2001	4,390.0	18	1996	5,390.0	56.37
11	May	2002	2,380.0	19	1984	4,930.0	59.55
04	May	2003	282.0	20	1997	4,400.0	62.74
04	Aug	2004	5,910.0	21	2001	4,390.0	65.92
12	Jun	2005	3,650.0	22	1998	4,270.0	69.11
28	Aug	2006	6,320.0	23	1990	4,150.0	72.29
07	May	2007	8,750.0	24	2005	3,650.0	75.48
06	Jun	2008	7,180.0	25	2000	3,560.0	78.66
24	Mar	2009	6,060.0	26	1991	3,140.0	81.85
05	Jun	2010	10,300.0	27	2002	2,380.0	85.03
27	Jun	2011	5,950.0	28	1994	1,730.0	88.22
30	Mar	2012	8,710.0	29	1985	1,160.0	91.40
15	Jun	2013	7,860.0	30	1988	328.0*	94.59
04	Jun	2014	7,080.0	31	2003	282.0*	97.77

* Outlier

<< Skew Weighting >>

Based on 31 events, mean-square error of station skew = 0.396
Mean-square error of regional skew = -?

<< Frequency Curve >>
Bedford IA-Bedford, IA-FLOW-ANNUAL PEAK

Computed	Expected	Percent	Confidence Limits

Curve FLOW, CFS	Probability FLOW, CFS	Chance Exceedance	0.05 FLOW, CFS	0.95 FLOW, CFS
10,863.9	10,995.7	0.2	14,410.5	8,828.8
10,665.2	10,808.2	0.5	14,093.0	8,685.3
10,446.2	10,595.1	1.0	13,745.1	8,526.5
10,140.7	10,282.5	2.0	13,263.2	8,303.6
9,537.5	9,667.6	5.0	12,324.5	7,859.0
8,848.8	8,940.3	10.0	11,274.3	7,343.1
7,835.8	7,891.9	20.0	9,774.8	6,565.5
5,545.1	5,545.1	50.0	6,622.2	4,697.2
3,291.3	3,224.2	80.0	3,911.1	2,667.3
2,316.6	2,213.1	90.0	2,836.4	1,762.6
1,659.9	1,534.3	95.0	2,117.5	1,174.6
795.5	645.4	99.0	1,131.9	470.8

<< Synthetic Statistics >>
Bedford IA-Bedford, IA-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.691	Historic Events	0
Standard Dev	0.244	High Outliers	0
Station Skew	-1.354	Low Outliers	2
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-1.354	Systematic Events	31

--- End of Analytical Frequency Curve ---

Bulletin 17B Frequency Analysis
14 Apr 2015 02:18 PM

--- Input Data ---

Analysis Name: KC_Bethany
Description:

Data Set Name: KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK
DSS File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream Gauges\Locust_Creek\Locust_Creek.dss
DSS Pathname: /East Fork Big Creek/Bethany, MO/FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream Gauges\Locust_Creek\Bulletin17bResults\KC_Bethany\KC_Bethany.rpt
XML File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream Gauges\Locust_Creek\Bulletin17bResults\KC_Bethany\KC_Bethany.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

Note: Adopted skew equals station skew and preliminary frequency statistics are for the conditional frequency curve because of zero or missing events.

<< Frequency Curve >>
KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
9,788.0	10,113.4	0.2	13,190.5	7,764.8
9,083.4	9,342.9	0.5	12,102.2	7,261.5
8,468.6	8,683.9	1.0	11,165.0	6,817.8
7,771.1	7,937.8	2.0	10,117.1	6,308.8
6,698.1	6,808.2	5.0	8,539.2	5,512.3
5,746.3	5,811.2	10.0	7,178.7	4,789.3
4,637.5	4,668.9	20.0	5,648.5	3,922.5
2,797.0	2,797.0	50.0	3,276.5	2,399.1
1,471.4	1,453.2	80.0	1,734.2	1,215.1

992.1	966.5	90.0	1,202.6	781.3
694.3	664.6	95.0	870.6	519.3
328.2	294.7	99.0	447.6	217.5

<< Conditional Statistics >>
KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.405	Historic Events	0
Standard Dev	0.305	High Outliers	0
Station Skew	-0.829	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	1
Adopted Skew	-0.829	Systematic Events	58

<< Conditional Probability Adjusted Ordinates >>

<< Frequency Curve >>
KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
9,774.8	---	0.2	---	---
9,068.1	---	0.5	---	---
8,451.1	---	1.0	---	---
7,752.1	---	2.0	---	---
6,674.3	---	5.0	---	---
5,718.5	---	10.0	---	---
4,605.6	---	20.0	---	---
2,754.5	---	50.0	---	---
1,405.4	---	80.0	---	---
903.1	---	90.0	---	---
569.3	---	95.0	---	---
---	---	99.0	---	---

--- End of Preliminary Results ---

<< Low Outlier Test >>

Based on 57 events, 10 percent outlier test deviate K(N) = 2.818
Computed low outlier test value = 350.16

1 low outlier(s) identified below test value of 350.16

Based on statistics after 0 zero events and 1 missing events were deleted.

Statistics and frequency curve adjusted for 1 low outlier(s)

<< Conditional Statistics >>

KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.424	Historic Events	0
Standard Dev	0.271	High Outliers	0
Station Skew	-0.229	Low Outliers	1
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	1
Adopted Skew	-0.829	Systematic Events	58

<< High Outlier Test >>

Based on 56 events, 10 percent outlier test deviate K(N) = 2.811
Computed high outlier test value = 15,320.95

0 high outlier(s) identified above test value of 15,320.95

Note: Statistics and frequency curve were modified
using conditional probablity adjustment.

--- Final Results ---

<< Plotting Positions >>

KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK

Events Analyzed				Ordered Events			
Day	Mon	Year	FLOW CFS	Rank	Water Year	FLOW CFS	Median Plot Pos
06	Jul	1909	---	1	1974	13,000.0	1.20
23	Jun	1934	590.0	2	1947	8,120.0	2.91
31	May	1935	3,500.0	3	1946	6,770.0	4.62
23	May	1936	980.0	4	1942	6,600.0	6.34
30	Jan	1937	1,610.0	5	2004	5,760.0	8.05
21	Aug	1938	210.0	6	1961	5,700.0	9.76
02	Aug	1939	2,060.0	7	1959	5,100.0	11.47
08	May	1940	1,780.0	8	2014	5,080.0	13.18
09	Jun	1941	2,950.0	9	2008	4,820.0	14.90
26	Jun	1942	6,600.0	10	2007	4,800.0	16.61
16	May	1943	3,110.0	11	2010	4,760.0	18.32
22	Apr	1944	3,210.0	12	1960	4,740.0	20.03
15	May	1945	4,120.0	13	1945	4,120.0	21.75
30	Jun	1946	6,770.0	14	2009	4,080.0	23.46
06	Jun	1947	8,120.0	15	1962	3,880.0	25.17
06	May	1948	2,310.0	16	2013	3,850.0	26.88
24	Feb	1949	2,000.0	17	1935	3,500.0	28.60
20	Sep	1950	1,300.0	18	1965	3,480.0	30.31
01	May	1951	2,920.0	19	1967	3,350.0	32.02
21	Jun	1952	2,970.0	20	1944	3,210.0	33.73
31	Mar	1953	925.0	21	1972	3,190.0	35.45
01	Jun	1954	1,330.0	22	1968	3,150.0	37.16
25	Jun	1955	2,240.0	23	1969	3,110.0	38.87
02	Aug	1956	2,500.0	24	1943	3,110.0	40.58

21 May 1957	1,620.0	25	1970	3,070.0	42.29
15 Jul 1958	1,780.0	26	1971	2,970.0	44.01
30 May 1959	5,100.0	27	1952	2,970.0	45.72
30 Jun 1960	4,740.0	28	1941	2,950.0	47.43
13 Sep 1961	5,700.0	29	1951	2,920.0	49.14
11 Jun 1962	3,880.0	30	2001	2,660.0	50.86
04 Mar 1963	2,100.0	31	1956	2,500.0	52.57
06 Sep 1964	1,910.0	32	1966	2,430.0	54.28
21 Sep 1965	3,480.0	33	1999	2,420.0	55.99
13 Jun 1966	2,430.0	34	1998	2,380.0	57.71
13 Jun 1967	3,350.0	35	1948	2,310.0	59.42
23 Apr 1968	3,150.0	36	1955	2,240.0	61.13
30 Jun 1969	3,110.0	37	1963	2,100.0	62.84
17 Sep 1970	3,070.0	38	1939	2,060.0	64.55
18 Feb 1971	2,970.0	39	1949	2,000.0	66.27
07 May 1972	3,190.0	40	1964	1,910.0	67.98
13 Oct 1973	13,000.0	41	2005	1,850.0	69.69
15 Apr 1997	1,790.0	42	1997	1,790.0	71.40
04 Jul 1998	2,380.0	43	2002	1,780.0	73.12
17 Oct 1998	2,420.0	44	1958	1,780.0	74.83
24 Feb 2001	2,660.0	45	1940	1,780.0	76.54
12 May 2002	1,780.0	46	2011	1,660.0	78.25
01 May 2003	543.0	47	1957	1,620.0	79.97
30 May 2004	5,760.0	48	1937	1,610.0	81.68
11 Jun 2005	1,850.0	49	1954	1,330.0	83.39
30 Apr 2006	1,110.0	50	1950	1,300.0	85.10
07 May 2007	4,800.0	51	2006	1,110.0	86.82
25 Jul 2008	4,820.0	52	2012	1,030.0	88.53
15 May 2009	4,080.0	53	1936	980.0	90.24
05 Jun 2010	4,760.0	54	1953	925.0	91.95
25 May 2011	1,660.0	55	1934	590.0	93.66
04 May 2012	1,030.0	56	2003	543.0	95.38
18 Apr 2013	3,850.0	57	1938	210.0*	97.09
10 Sep 2014	5,080.0	58	1909	---	98.80

* Outlier

<< Skew Weighting >>

Based on 58 events, mean-square error of station skew =	0.103
Mean-square error of regional skew =	-?

<< Frequency Curve >>

KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
13,447.6	14,474.8	0.2	18,788.2	10,428.3
11,527.0	12,203.7	0.5	15,707.2	9,096.8
10,121.3	10,595.1	1.0	13,509.6	8,103.0
8,754.4	9,067.9	2.0	11,425.1	7,117.7
7,000.4	7,163.1	5.0	8,838.4	5,819.7
5,704.6	5,789.3	10.0	7,003.2	4,829.0
4,417.9	4,453.0	20.0	5,260.6	3,808.1
2,645.1	2,645.1	50.0	3,037.3	2,306.0
1,533.7	1,519.6	80.0	1,778.0	1,289.8
1,138.6	1,118.0	90.0	1,348.2	923.9

884.2	858.6	95.0	1,071.5	692.5
541.0	506.8	99.0	690.9	392.6

<< Synthetic Statistics >>
KC_Data_other-Bethany, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.413	Historic Events	0
Standard Dev	0.274	High Outliers	0
Station Skew	-0.216	Low Outliers	1
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	1
Adopted Skew	-0.216	Systematic Events	58

--- End of Analytical Frequency Curve ---

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Bulletin 17B Frequency Analysis
    14 Apr 2015   02:17 PM
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--- Input Data ---

Analysis Name: KC_Chula
Description:

Data Set Name: KC_Data_other-Chula, MO-FLOW-ANNUAL PEAK
DSS File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Locust_Creek.dss
DSS Pathname: /Muddy Creek/Chula, MO/FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Chula\KC_Chula.rpt
XML File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Chula\KC_Chula.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---


Warning: Less than 10 events for analysis,
  Bulletin 17B procedures are not applicable.



----------------------
<< Low Outlier Test >>
----------------------
  Based on 4 events, 10 percent outlier test deviate K(N) = 1.425
                       Computed low outlier test value = 1,721.56

         0 low outlier(s) identified below test value of 1,721.56



-----------------------
<< High Outlier Test >>
-----------------------
  Based on 4 events, 10 percent outlier test deviate K(N) = 1.425
                      Computed high outlier test value = 4,749.52

        0 high outlier(s) identified above test value of 4,749.52
```

--- Final Results ---

<< Plotting Positions >>
KC_Data_other-Chula, MO-FLOW-ANNUAL PEAK

Events Analyzed				Ordered Events			
Day	Mon	Year	FLOW CFS	Rank	Water Year	FLOW CFS	Median Plot Pos
17	Feb	2011	2,520.0	1	2014	4,220.0	15.91
29	Mar	2012	1,860.0	2	2013	3,380.0	38.64
18	Apr	2013	3,380.0	3	2011	2,520.0	61.36
10	Sep	2014	4,220.0	4	2012	1,860.0	84.09

<< Skew Weighting >>

Based on 4 events, mean-square error of station skew =	1.091
Mean-square error of regional skew =	-?

<< Frequency Curve >>
KC_Data_other-Chula, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
7,113.6	33,452.3	0.2	49,060.0	4,608.4
6,549.1	17,940.0	0.5	38,385.3	4,358.3
6,105.2	12,437.8	1.0	31,199.1	4,152.7
5,643.1	9,229.9	2.0	24,760.0	3,928.2
4,994.3	6,648.9	5.0	17,366.0	3,589.6
4,462.2	5,282.8	10.0	12,600.2	3,282.2
3,872.7	4,200.6	20.0	8,542.4	2,889.3
2,904.7	2,904.7	50.0	4,370.5	1,980.9
2,130.8	1,931.6	80.0	2,855.7	980.7
1,795.8	1,437.1	90.0	2,450.1	613.1
1,551.8	1,020.9	95.0	2,188.1	403.3
1,166.2	326.8	99.0	1,791.6	174.0

<< Systematic Statistics >>
KC_Data_other-Chula, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.456	Historic Events	0
Standard Dev	0.155	High Outliers	0
Station Skew	-0.265	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-0.265	Systematic Events	4

--- End of Analytical Frequency Curve ---

```
-------------------------------
Bulletin 17B Frequency Analysis
    14 Apr 2015   02:25 PM
-------------------------------


--- Input Data ---

Analysis Name: KC_Hickory Branch
Description:

Data Set Name: Hickory Branch-Mendon, MO-FLOW-ANNUAL PEAK
DSS File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Locust_Creek.dss
DSS Pathname: /Hickory Branch/Mendon, MO/FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Hickory_Branch\KC_Hickory_Branch.rpt
XML File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Hickory_Branch\KC_Hickory_Branch.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---


Warning: Less than 10 events for analysis,
  Bulletin 17B procedures are not applicable.



----------------------
<< Low Outlier Test >>
----------------------
  Based on 4 events, 10 percent outlier test deviate K(N) = 1.425
                       Computed low outlier test value = 245.87

           0 low outlier(s) identified below test value of 245.87



-----------------------
<< High Outlier Test >>
-----------------------
  Based on 4 events, 10 percent outlier test deviate K(N) = 1.425
                    Computed high outlier test value = 3,069.98

        0 high outlier(s) identified above test value of 3,069.98
```

--- Final Results ---

<< Plotting Positions >>
Hickory Branch-Mendon, MO-FLOW-ANNUAL PEAK

Events Analyzed			Ordered Events			
Day Mon Year	FLOW CFS		Rank	Water Year	FLOW CFS	Median Plot Pos
28 Feb 2011	1,480.0		1	2013	1,760.0	15.91
29 Mar 2012	882.0		2	2011	1,480.0	38.64
18 Apr 2013	1,760.0		3	2012	882.0	61.36
03 Apr 2014	248.0		4	2014	248.0	84.09

<< Skew Weighting >>

Based on 4 events, mean-square error of station skew =	1.351
Mean-square error of regional skew =	-?

<< Frequency Curve >>
Hickory Branch-Mendon, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
2,934.5	3,494.9	0.2	47,468.7	1,339.3
2,862.3	3,269.9	0.5	44,230.0	1,312.8
2,781.3	3,144.4	1.0	40,774.4	1,282.6
2,666.7	3,045.8	2.0	36,212.8	1,239.0
2,439.4	2,875.4	5.0	28,215.3	1,149.5
2,182.3	2,548.8	10.0	20,734.6	1,042.5
1,815.0	2,017.4	20.0	12,613.3	876.0
1,061.8	1,061.8	50.0	3,448.7	454.5
465.7	337.7	80.0	968.4	80.4
265.9	115.2	90.0	578.9	17.5
155.9	25.9	95.0	386.9	3.7
47.7	0.1	99.0	175.0	0.1

<< Systematic Statistics >>
Hickory Branch-Mendon, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	2.939	Historic Events	0
Standard Dev	0.385	High Outliers	0
Station Skew	-1.407	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-1.407	Systematic Events	4

--- End of Analytical Frequency Curve ---

Bulletin 17B Frequency Analysis
14 Apr 2015 01:47 PM

--- Input Data ---

Analysis Name: KC_Linneus
Description:

Data Set Name: KC_StreamData-Linneus, MO-FLOW-ANNUAL PEAK
DSS File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream Gauges\Locust_Creek\Locust_Creek.dss
DSS Pathname: /Locust Creek/Linneus, MO/FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream Gauges\Locust_Creek\Bulletin17bResults\KC_Linneus\KC_Linneus.rpt
XML File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream Gauges\Locust_Creek\Bulletin17bResults\KC_Linneus\KC_Linneus.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

<< Skew Weighting >>

Based on 65 events, mean-square error of station skew =	0.229
Mean-square error of regional skew =	-?

<< Frequency Curve >>
KC_StreamData-Linneus, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
31,122.2	31,472.2	0.2	40,683.6	25,069.6
30,084.1	30,421.4	0.5	39,153.7	24,304.7
29,003.2	29,336.8	1.0	37,570.2	23,504.5
27,573.8	27,875.9	2.0	35,491.5	22,440.1
24,942.3	25,196.6	5.0	31,712.6	20,460.5
22,163.9	22,332.3	10.0	27,796.1	18,338.7
18,409.6	18,504.2	20.0	22,639.0	15,410.2
11,079.0	11,079.0	50.0	13,129.4	9,414.8

5,280.6	5,209.5	80.0	6,278.5	4,331.7
3,233.1	3,138.8	90.0	3,970.8	2,513.7
2,037.1	1,936.9	95.0	2,609.4	1,488.6
741.4	650.5	99.0	1,059.3	464.9

<< Systematic Statistics >>
KC_StreamData-Linneus, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.974	Historic Events	0
Standard Dev	0.346	High Outliers	0
Station Skew	-1.259	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-1.259	Systematic Events	65

--- End of Preliminary Results ---

<< Low Outlier Test >>

Based on 65 events, 10 percent outlier test deviate K(N) = 2.866
Computed low outlier test value = 956.64

2 low outlier(s) identified below test value of 956.64

Statistics and frequency curve adjusted for 2 low outlier(s)

<< Systematic Statistics >>
KC_StreamData-Linneus, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	4.009	Historic Events	0
Standard Dev	0.290	High Outliers	0
Station Skew	-0.780	Low Outliers	2
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-1.259	Systematic Events	65

<< High Outlier Test >>

Based on 63 events, 10 percent outlier test deviate K(N) = 2.854
Computed high outlier test value = 68,437.95

0 high outlier(s) identified above test value of 68,437.95

Note: Statistics and frequency curve were modified using conditional probablity adjustment.

--- Final Results ---

<< Plotting Positions >>
KC_StreamData-Linneus, MO-FLOW-ANNUAL PEAK

Events Analyzed Day	Mon	Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Median Plot Pos
30	Jun	1909	18,000.0	1	1947	38,000.0	1.07
30	Jun	1930	7,920.0	2	2014	32,500.0	2.60
20	Apr	1931	8,800.0	3	2004	27,700.0	4.13
23	Nov	1931	8,900.0	4	2008	26,900.0	5.66
24	Dec	1932	4,390.0	5	1969	24,400.0	7.19
05	Apr	1934	900.0	6	1958	24,000.0	8.72
02	Jun	1935	11,800.0	7	2009	23,900.0	10.24
26	Sep	1936	3,100.0	8	2007	20,500.0	11.77
30	Jan	1937	5,110.0	9	1944	20,100.0	13.30
10	Apr	1938	639.0	10	2010	19,300.0	14.83
21	Jun	1939	15,400.0	11	2002	19,200.0	16.36
18	Aug	1940	3,110.0	12	1942	19,000.0	17.89
11	Jun	1941	11,800.0	13	1909	18,000.0	19.42
26	Jun	1942	19,000.0	14	1967	17,800.0	20.95
10	Jun	1943	10,800.0	15	2013	16,700.0	22.48
23	Apr	1944	20,100.0	16	1945	16,500.0	24.01
16	Jun	1945	16,500.0	17	2001	15,600.0	25.54
06	Jan	1946	8,920.0	18	1939	15,400.0	27.06
06	Jun	1947	38,000.0	19	1973	15,000.0	28.59
20	Mar	1948	11,900.0	20	1970	14,800.0	30.12
15	Jun	1949	9,570.0	21	1953	14,000.0	31.65
16	Jun	1950	13,200.0	22	1960	13,800.0	33.18
24	Jul	1951	12,300.0	23	1950	13,200.0	34.71
22	Jun	1952	8,200.0	24	1965	13,000.0	36.24
31	Mar	1953	14,000.0	25	1951	12,300.0	37.77
02	Jun	1954	7,280.0	26	1948	11,900.0	39.30
25	Jun	1955	8,000.0	27	1941	11,800.0	40.83
03	Jul	1956	5,640.0	28	1935	11,800.0	42.35
04	Apr	1957	1,910.0	29	2011	11,200.0	43.88
15	Jul	1958	24,000.0	30	1943	10,800.0	45.41
17	Nov	1958	10,300.0	31	1962	10,700.0	46.94
30	Jun	1960	13,800.0	32	1959	10,300.0	48.47
13	Sep	1961	9,690.0	33	1961	9,690.0	50.00
17	Nov	1961	10,700.0	34	1949	9,570.0	51.53
04	Mar	1963	9,520.0	35	1963	9,520.0	53.06
21	Apr	1964	5,180.0	36	1974	9,200.0	54.59
01	Jan	1965	13,000.0	37	1977	9,000.0	56.12
13	Jun	1966	7,080.0	38	1946	8,920.0	57.65
13	Jun	1967	17,800.0	39	1932	8,900.0	59.17
23	Apr	1968	7,700.0	40	1931	8,800.0	60.70
10	Jul	1969	24,400.0	41	2005	8,700.0	62.23
23	Sep	1970	14,800.0	42	1952	8,200.0	63.76
09	Oct	1970	7,010.0	43	1978	8,000.0	65.29
15	Dec	1971	5,680.0	44	1955	8,000.0	66.82
01	May	1973	15,000.0	45	1930	7,920.0	68.35
18	May	1974	9,200.0	46	1968	7,700.0	69.88
23	Apr	1975	6,700.0	47	1954	7,280.0	71.41
20	Apr	1976	4,500.0	48	1966	7,080.0	72.94

28 Mar 1977	9,000.0	49	1971	7,010.0	74.46
10 Apr 1978	8,000.0	50	1979	6,800.0	75.99
03 Mar 1979	6,800.0	51	1975	6,700.0	77.52
06 Jun 2001	15,600.0	52	2006	6,000.0	79.05
12 May 2002	19,200.0	53	1972	5,680.0	80.58
10 May 2003	984.0	54	1956	5,640.0	82.11
29 Aug 2004	27,700.0	55	1964	5,180.0	83.64
08 Jun 2005	8,700.0	56	1937	5,110.0	85.17
11 Jun 2006	6,000.0	57	2012	5,090.0	86.70
07 May 2007	20,500.0	58	1976	4,500.0	88.23
25 Jun 2008	26,900.0	59	1933	4,390.0	89.76
16 May 2009	23,900.0	60	1940	3,110.0	91.28
13 May 2010	19,300.0	61	1936	3,100.0	92.81
17 Feb 2011	11,200.0	62	1957	1,910.0	94.34
03 May 2012	5,090.0	63	2003	984.0	95.87
28 May 2013	16,700.0	64	1934	900.0*	97.40
10 Sep 2014	32,500.0	65	1938	639.0*	98.93

* Outlier

<< Skew Weighting >>

Based on 65 events, mean-square error of station skew = 0.137
Mean-square error of regional skew = -?

<< Frequency Curve >>
KC_StreamData-Linneus, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
37,876.0	39,027.5	0.2	49,774.5	30,541.3
35,041.0	35,948.5	0.5	45,544.4	28,472.4
32,596.6	33,341.7	1.0	41,943.5	26,670.4
29,852.3	30,423.7	2.0	37,955.7	24,625.1
25,679.4	26,051.4	5.0	32,014.3	21,463.6
22,018.1	22,236.3	10.0	26,938.7	18,628.4
17,789.6	17,894.7	20.0	21,265.9	15,263.1
10,822.9	10,822.9	50.0	12,495.4	9,410.4
5,800.6	5,740.0	80.0	6,742.1	4,876.7
3,967.1	3,880.9	90.0	4,731.2	3,197.6
2,815.6	2,714.4	95.0	3,463.0	2,168.3
1,375.4	1,257.5	99.0	1,824.9	952.0

<< Synthetic Statistics >>
KC_StreamData-Linneus, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.996	Historic Events	0
Standard Dev	0.297	High Outliers	0
Station Skew	-0.789	Low Outliers	2
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-0.789	Systematic Events	65

```
|-------------------------------|--------------------------------|
```

--- End of Analytical Frequency Curve ---

```
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Bulletin 17B Frequency Analysis
    14 Apr 2015   01:48 PM
--------------------------------


--- Input Data ---

Analysis Name: KC_Reger
Description:

Data Set Name: KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK
DSS File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Locust_Creek.dss
DSS Pathname: /Locust Creek/Reger, MO/FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Reger\KC_Reger.rpt
XML File Name: F:\PROJECTS\A11-1513\40-Design\Calcs\WTRS\Stream
Gauges\Locust_Creek\Bulletin17bResults\KC_Reger\KC_Reger.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Use non-standard frequencies
Frequency: 0.2
Frequency: 0.5
Frequency: 1.0
Frequency: 2.0
Frequency: 4.0
Frequency: 10.0
Frequency: 20.0
Frequency: 50.0
Frequency: 80.0
Frequency: 90.0
Frequency: 95.0
Frequency: 99.0

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---


--- Preliminary Results ---

Note: Adopted skew equals station skew and preliminary
frequency statistics are for the conditional frequency curve
because of zero or missing events.



<< Frequency Curve >>
KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK
-------------------------------------------------------------------
```

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
21,827.2	24,088.7	0.2	40,115.0	14,489.7
19,845.2	21,650.4	0.5	35,564.9	13,357.5
18,160.1	19,604.5	1.0	31,801.1	12,377.2
16,297.7	17,371.8	2.0	27,761.3	11,272.6
14,240.8	14,993.0	4.0	23,458.3	10,022.8
11,187.8	11,559.6	10.0	17,414.5	8,096.2
8,586.5	8,754.5	20.0	12,641.5	6,365.0
4,624.2	4,624.2	50.0	6,227.3	3,480.5
2,120.8	2,048.2	80.0	2,847.8	1,458.5
1,317.8	1,224.1	90.0	1,850.9	818.6
857.4	757.2	95.0	1,274.3	478.1
348.8	255.6	99.0	597.5	151.5

<< Conditional Statistics >>
KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.616	Historic Events	0
Standard Dev	0.371	High Outliers	0
Station Skew	-0.801	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	1
Adopted Skew	-0.801	Systematic Events	26

<< Conditional Probability Adjusted Ordinates >>

<< Frequency Curve >>
KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
21,742.2	---	0.2	---	---
19,749.5	---	0.5	---	---
18,054.1	---	1.0	---	---
16,184.9	---	2.0	---	---
14,116.2	---	4.0	---	---
11,037.6	---	10.0	---	---
8,423.5	---	20.0	---	---
4,431.6	---	50.0	---	---
1,861.7	---	80.0	---	---
991.5	---	90.0	---	---
383.6	---	95.0	---	---
---	---	99.0	---	---

--- End of Preliminary Results ---

<< Low Outlier Test >>

Based on 25 events, 10 percent outlier test deviate K(N) = 2.486
Computed low outlier test value = 493.25

1 low outlier(s) identified below test value of 493.25

Based on statistics after 0 zero events and 1 missing events were deleted.

Statistics and frequency curve adjusted for 1 low outlier(s)

<< Conditional Statistics >>

KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.659	Historic Events	0
Standard Dev	0.310	High Outliers	0
Station Skew	-0.185	Low Outliers	1
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	1
Adopted Skew	-0.801	Systematic Events	26

<< High Outlier Test >>

Based on 24 events, 10 percent outlier test deviate K(N) = 2.467
Computed high outlier test value = 26,571.25

0 high outlier(s) identified above test value of 26,571.25

Note: Statistics and frequency curve were modified
using conditional probablity adjustment.

--- Final Results ---

<< Plotting Positions >>

KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK

Events Analyzed Day Mon Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Median Plot Pos
20 Dec 1987	1,900.0	1	1993	19,700.0	2.65
29 May 1989	2,320.0	2	1991	10,800.0	6.44
30 Nov 1989	3,940.0	3	2008	10,200.0	10.23
05 May 1991	10,800.0	4	2004	9,500.0	14.02
19 Apr 1992	4,470.0	5	1996	8,030.0	17.80
07 Jul 1993	19,700.0	6	1995	7,820.0	21.59
09 Oct 1993	2,710.0	7	2009	7,810.0	25.38
26 May 1995	7,820.0	8	2013	7,660.0	29.17
28 May 1996	8,030.0	9	2002	6,390.0	32.95
17 Apr 1997	3,890.0	10	1999	6,390.0	36.74
02 Apr 1998	3,800.0	11	2010	5,770.0	40.53

07 Oct 1998	6,390.0	12	2007	4,720.0	44.32
26 Jun 2000	1,800.0	13	1992	4,470.0	48.11
08 Nov 2000	---	14	1990	3,940.0	51.89
14 May 2002	6,390.0	15	1997	3,890.0	55.68
11 May 2003	391.0	16	1998	3,800.0	59.47
29 Aug 2004	9,500.0	17	2011	2,840.0	63.26
12 Apr 2005	2,420.0	18	1994	2,710.0	67.05
11 Jun 2006	919.0	19	2005	2,420.0	70.83
08 May 2007	4,720.0	20	1989	2,320.0	74.62
26 Jul 2008	10,200.0	21	2012	2,110.0	78.41
16 May 2009	7,810.0	22	1988	1,900.0	82.20
06 Jun 2010	5,770.0	23	2000	1,800.0	85.98
17 Feb 2011	2,840.0	24	2006	919.0	89.77
03 May 2012	2,110.0	25	2003	391.0*	93.56
19 Apr 2013	7,660.0	26	2001	---	97.35

* Outlier

<< Skew Weighting >>

Based on 26 events, mean-square error of station skew =	0.209
Mean-square error of regional skew =	-?

<< Frequency Curve >>
KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
30,018.1	36,973.2	0.2	57,837.9	19,506.8
24,975.0	29,338.8	0.5	45,759.8	16,711.7
21,385.6	24,310.2	1.0	37,583.4	14,656.4
17,986.9	19,835.8	2.0	30,205.8	12,647.6
14,773.8	15,860.4	4.0	23,599.5	10,677.8
10,794.5	11,237.4	10.0	16,003.6	8,109.6
7,960.4	8,132.6	20.0	11,077.6	6,153.1
4,321.4	4,321.4	50.0	5,550.5	3,374.8
2,259.2	2,203.6	80.0	2,919.4	1,628.2
1,585.1	1,508.4	90.0	2,118.0	1,060.9
1,173.4	1,082.6	95.0	1,628.2	730.7
654.4	545.5	99.0	990.5	349.5

<< Synthetic Statistics >>
KC_StreamData-Reger, MO-FLOW-ANNUAL PEAK

Log Transform: FLOW, CFS		Number of Events	
Mean	3.624	Historic Events	0
Standard Dev	0.326	High Outliers	0
Station Skew	-0.214	Low Outliers	1
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	1
Adopted Skew	-0.214	Systematic Events	26

--- End of Analytical Frequency Curve ---

Techniques for Estimating the 2- to 500-Year Flood Discharges on Unregulated Streams in Rural Missouri
USGS - Water Resources Investigations Report 95-4231

Site: East Locust Creek
Existing Conditions
ELC 0
Project No. A11-1515

Equations:

Region I	Region II	Region III
$Q_2 = 69.4A^{0.703}S^{0.373}$	$Q_2 = 77.9A^{0.733}S^{0.265}$	$Q_2 = 88.0A^{0.658}$
$Q_5 = 123A^{0.690}S^{0.383}$	$Q_5 = 99.6A^{0.763}S^{0.355}$	$Q_5 = 145A^{0.627}$
$Q_{10} = 170A^{0.680}S^{0.378}$	$Q_{10} = 117A^{0.774}S^{0.395}$	$Q_{10} = 187A^{0.612}$
$Q_{25} = 243A^{0.668}S^{0.366}$	$Q_{25} = 140A^{0.784}S^{0.432}$	$Q_{25} = 244A^{0.595}$
$Q_{50} = 305A^{0.660}S^{0.356}$	$Q_{50} = 155A^{0.789}S^{0.453}$	$Q_{50} = 288A^{0.585}$
$Q_{100} = 376A^{0.652}S^{0.346}$	$Q_{100} = 170A^{0.794}S^{0.471}$	$Q_{100} = 334A^{0.576}$
$Q_{500} = 569A^{0.636}S^{0.321}$	$Q_{500} = 203A^{0.804}S^{0.503}$	$Q_{500} = 448A^{0.557}$

Input Data:

A =	32.66	mi^2	Area contributing to surface runoff.
S =	9.81	ft/mile	Between 10% and 85%, measured from site to ridge along low-water channel.
Region =	1		Figure 1

Output Data:

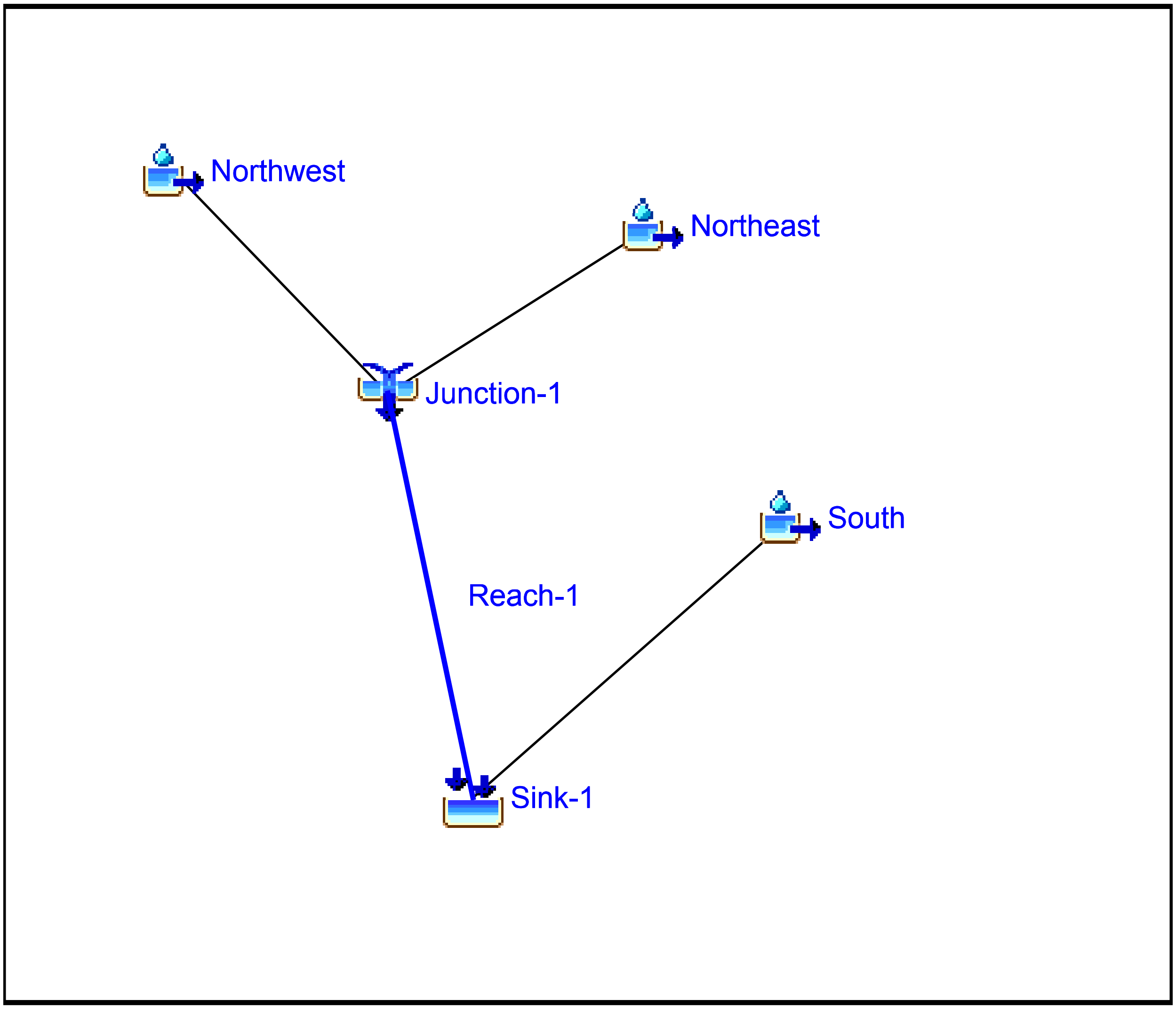
Q_2 =	1,886	cfs
Q_5 =	3,268	cfs
Q_{10} =	4,313	cfs
Q_{25} =	5,753	cfs
Q_{50} =	**6,863**	**cfs**
Q_{100} =	**8,043**	**cfs**
Q_{500} =	10,872	cfs

HEC-HMS

Project : E. Locust Crk

Basin Model : Propsed w/ Lake

May 28 13:38:27 CDT 2015



2-year, 24 hour

Simulation Run:	2-Yr 24 hours
Basin Model:	Proposed w/ Lake
Meteorologic Model:	2-Year
Control Specifications:	24-hr

Hydrologic Element	Drainage Area (mi2)	Peak Discharge	Time of Peak	Volume (ac-ft)
Northeast	11.68	2765	15May2015, 01:40	883
Northwest	7.11	1866	15May2015, 01:25	538
Junction-1	18.79	4592	15May2015, 01:35	1421
Reach-1	18.79	4587	15May2015, 01:55	1413
South	13.84	4079	15May2015, 01:40	1266
Sink-1	32.63	8600	15May2015, 01:45	2679

10-year, 24 hour

Simulation Run:	10-Yr 24 hours
Basin Model:	Proposed w/ Lake
Meteorologic Model:	10-Year
Control Specifications:	24-hr

Hydrologic Element	Drainage Area (mi2)	Peak Discharge	Time of Peak	Volume (ac-ft)
Northeast	11.68	5304	15May2015, 01:40	1654
Northwest	7.11	3585	15May2015, 01:25	1009
Junction-1	18.79	8811	15May2015, 01:30	2663
Reach-1	18.79	8809	15May2015, 01:50	2649
South	13.84	7282	15May2015, 01:35	2244
Sink-1	32.63	15983	15May2015, 01:45	4894

50-year, 24 hour

Simulation Run:	50-Yr 24 hours
Basin Model:	Proposed w/ Lake
Meteorologic Model:	50-Year
Control Specifications:	24-hr

Hydrologic Element	Drainage Area (mi2)	Peak Discharge	Time of Peak	Volume (ac-ft)
Northeast	11.68	8861	15May2015, 01:35	2754
Northwest	7.11	5991	15May2015, 01:25	1681
Junction-1	18.79	14751	15May2015, 01:30	4435
Reach-1	18.79	14733	15May2015, 01:50	4415
South	13.84	11639	15May2015, 01:35	3604
Sink-1	32.63	26192	15May2015, 01:45	8019

100-year, 24 hour

Simulation Run:	100-Yr 24 hours
Basin Model:	Proposed w/ Lake
Meteorologic Model:	100-Year
Control Specifications:	24-hr

Hydrologic Element	Drainage Area (mi2)	Peak Discharge	Time of Peak	Volume (ac-ft)
Northeast	11.68	10665	15May2015, 01:35	3319
Northwest	7.11	7208	15May2015, 01:20	2026
Junction-1	18.79	17751	15May2015, 01:30	5344
Reach-1	18.79	17723	15May2015, 01:50	5321
South	13.84	13810	15May2015, 01:35	4292
Sink-1	32.63	31328	15May2015, 01:40	9613

East Locust - Time of Concentration Estimation - Northeast Drainage Area

Time of Concentration (Tc) = $T_{sheet} + T_{shallow\ conc} + T_{channel} + T_{wave\ velocity}$

Reference: National Engineering Handbook, Part 630 Hydrology, Chapter 15, USDA NRCS, May 2010

P_2 Source for T_{sheet}:
NOAA Atlas 14 - Milan MO Station
http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=mo

T_{sheet}

$$T_t = \frac{0.007(n\ell)^{0.8}}{(P_2)^{0.5} S^{0.4}} \qquad \text{(eq. 15–8)}$$

where:
T_t = travel time, h
n = Manning's roughness coefficient (table 15–1)
ℓ = sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
S = slope of land surface, ft/ft

n =	0.03
L =	100 ft
P_2 =	3.37 in
Begin elev =	1076.97 ft
End elev =	1076 ft
s =	0.010 ft/ft
T_{sheet} =	**0.059** hr

$T_{shallow}$

Velocity =

v = 6.962*(slope)$^{0.5}$
(for short grass pasture)

(Table 15-3 and Figure 15-4)
(NEH Part 360 Hydrology, Chapter 15)

$T_{shallow}$ =	L (ft)/v (ft) * 1/3600
Begin elev =	1076 ft
End elev =	1062.99 ft
s =	0.00840 ft/ft
v =	0.64 ft/s
$L_{shallow}$ =	1549 ft
$T_{shallow}$ =	**0.674** hr

Travel time (T_t) is the time it takes water to travel from one location to another. Travel time between two points is determined using the following relationship:

$$T_t = \frac{\ell}{3{,}600V} \qquad \text{(eq. 15–1)}$$

where:
T_t = travel time, h
ℓ = distance between the two points under consideration, ft
V = average velocity of flow between the two points, ft/s
3,600 = conversion factor, s to h

The Total Time of Concentration is the sum of all travel times:

(b) Velocity method

Another method for determining time of concentration normally used within the NRCS is called the velocity method. The velocity method assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path.

$$T_c = T_{t1} + T_{t2} + T_{t3} + \ldots T_{tn} \qquad \text{(eq. 15–7)}$$

where:
T_c = time of concentration, h
T_{tn} = travel time of a segment n, h
n = number of segments comprising the total hydraulic length

The segments used in the velocity method may be of three types: sheet flow, shallow concentrated flow, and open channel flow.

East Locust - Time of Concentration Estimation (Continued)

$T_{channel(1)}$		
$T_{channel}$ =	L (ft)/v (ft) * 1/3600	
Begin elev =	1062.99	ft
End elev =	922.30	ft
XS Flow Area =	155.35	ft^2
Wetted Perimeter =	56.64	ft
Hydraulic Radius =	2.74	ft
Manning's n =	0.040	
s =	0.00432	ft/ft
v =	4.78	ft/s
$L_{channel}$ =	32558	ft
$T_{channel}$ =	**1.892**	hr

$T_{wave\ velocity}$

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s²
D_m = mean depth of lake or reservoir, ft

$T_{wave\ velocity}$ =	L (ft)/v (ft) * 1/3600	
Volume at elev. 922.3 =	54009	ac-ft
Surface Area at elev. 922.3 =	2331	acres
D_m (Mean Depth) =	23.2	ft
v_w =	27.3	ft/s
$L_{channel}$ =	15442	ft
$T_{channel}$ =	**0.157**	hr

Tc =	**2.78 hr**

Velocity for Channel Flow use Mannning's Eq:

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{(eq. 15–10)}$$

where:
V = average velocity, ft/s
r = hydraulic radius, ft
$= \frac{a}{P_w}$
a = cross-sectional flow area, ft²
P_w = wetted perimeter, ft
s = slope of the hydraulic grade line (channel slope), ft/ft
n = Manning's *n* value for open channel flow

Manning's *n* values for open channel flow can be obtained from standard hydraulics textbooks, such as Chow (1959), and Linsley, Kohler, and Paulhus (1982). Publications dealing specifically with Manning's *n* values are Barnes (1967); Arcement and Schneider (1989); Phillips and Ingersoll (1998); and Cowen (1956). For guidance on calculating Manning's *n* values, see NEH630.14, Stage Discharge Relations.

Wave Velocity through Dam Backwater

In other cases, such as with a watershed having a relatively large body of water in the flow path, time of concentration is computed to the upstream end of the water body using standard methods, and velocity for the flow segment through the water body may be computed using the wave velocity equation coupled with equation 15–1 to convert the velocity to a travel time through the water body. The wave equation is:

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s²
D_m = mean depth of lake or reservoir, ft

Generally, V_w will be high; however, equation 15–11 only provides for estimating travel time through the water body and for the inflow hydrograph to reach the outlet. It does not account for the time required for the

Figure 15–4 Velocity versus slope for shallow concentrated flow

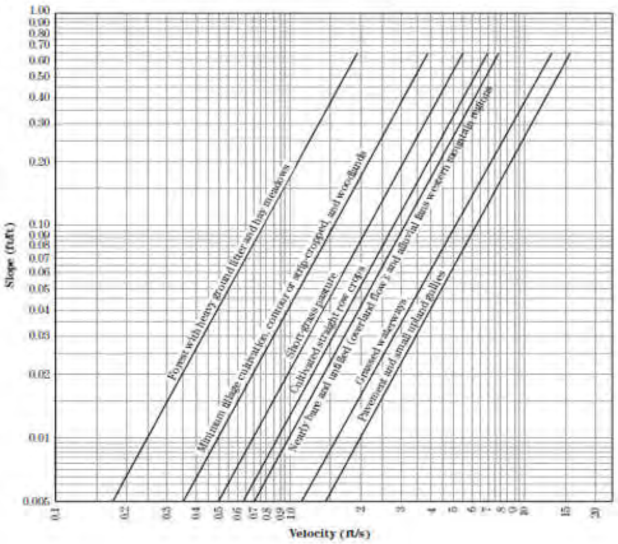


Table 15–3 Equations and assumptions developed from figure 15–4

Flow type	Depth (ft)	Manning's *n*	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V=20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V=16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V=9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V=8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V=6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V=5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V=2.516(s)^{0.5}$

East Locust - Time of Concentration Estimation - Northwest Drainage Area

Time of Concentration (Tc) = $T_{sheet} + T_{shallow\ conc} + T_{channel} + T_{wave\ velocity}$

Reference: National Engineering Handbook, Part 630 Hydrology, Chapter 15, USDA NRCS, May 2010

P_2 Source for T_{sheet}:
NOAA Atlas 14 - Milan MO Station
http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=mo

T_{sheet}

$$T_t = \frac{0.007(n\ell)^{0.8}}{(P_2)^{0.5}S^{0.4}} \qquad \text{(eq. 15–8)}$$

where:
T_t = travel time, h
n = Manning's roughness coefficient (table 15–1)
ℓ = sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
S = slope of land surface, ft/ft

n =	0.03
L =	105 ft
P_2 =	3.37 in
Begin elev =	1070 ft
End elev =	1068 ft
s =	0.019 ft/ft
T_{sheet} =	**0.047** hr

$T_{shallow}$

Velocity =

v = 6.962*(slope)$^{0.5}$
(for short grass pasture)

(Table 15-3 and Figure 15-4)
(NEH Part 360 Hydrology, Chapter 15)

$T_{shallow}$ =	L (ft)/v (ft) * 1/3600
Begin elev =	1068 ft
End elev =	1048 ft
s =	0.02000 ft/ft
v =	0.98 ft/s
$L_{shallow}$ =	1000 ft
$T_{shallow}$ =	**0.282** hr

Travel time (T_t) is the time it takes water to travel from one location to another. Travel time between two points is determined using the following relationship:

$$T_t = \frac{\ell}{3{,}600V} \qquad \text{(eq. 15–1)}$$

where:
T_t = travel time, h
ℓ = distance between the two points under consideration, ft
V = average velocity of flow between the two points, ft/s
3,600 = conversion factor, s to h

The Total Time of Concentration is the sum of all travel times:

(b) Velocity method

Another method for determining time of concentration normally used within the NRCS is called the velocity method. The velocity method assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path.

$$T_c = T_{t1} + T_{t2} + T_{t3} + \ldots T_{tn} \qquad \text{(eq. 15–7)}$$

where:
T_c = time of concentration, h
T_{tn} = travel time of a segment n, h
n = number of segments comprising the total hydraulic length

The segments used in the velocity method may be of three types: sheet flow, shallow concentrated flow, and open channel flow.

East Locust - Time of Concentration Estimation (Continued)

$T_{channel(1)}$		
$T_{channel}$ =	L (ft)/v (ft) * 1/3600	
Begin elev =	1048	ft
End elev =	922.88	ft
XS Flow Area =	199	ft^2
Wetted Perimeter =	116	ft
Hydraulic Radius =	1.72	ft
Manning's n =	0.040	
s =	0.00485	ft/ft
v =	3.71	ft/s
$L_{channel}$ =	25778	ft
$T_{channel}$ =	**1.933**	hr

$T_{wave\ velocity}$		

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

$T_{wave\ velocity}$ =	L (ft)/v (ft) * 1/3600	
Volume at elev. 922.3 =	54009	ac-ft
Surface Area at elev. 922.3 =	2331	acres
D_m (Mean Depth) =	23.2	ft
v_w =	27.3	ft/s
$L_{channel}$ =	14731	ft
$T_{channel}$ =	**0.150**	hr

Tc =	**2.41 hr**

Velocity for Channel Flow use Mannning's Eq:

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{(eq. 15–10)}$$

where:
V = average velocity, ft/s
r = hydraulic radius, ft
$= \frac{a}{P_w}$
a = cross-sectional flow area, ft^2
P_w = wetted perimeter, ft
s = slope of the hydraulic grade line (channel slope), ft/ft
n = Manning's *n* value for open channel flow

Manning's *n* values for open channel flow can be obtained from standard hydraulics textbooks, such as Chow (1959), and Linsley, Kohler, and Paulhus (1982). Publications dealing specifically with Manning's *n* values are Barnes (1967); Arcement and Schneider (1989); Phillips and Ingersoll (1998); and Cowen (1956). For guidance on calculating Manning's *n* values, see NEH630.14, Stage Discharge Relations.

Wave Velocity through Dam Backwater

In other cases, such as with a watershed having a relatively large body of water in the flow path, time of concentration is computed to the upstream end of the water body using standard methods, and velocity for the flow segment through the water body may be computed using the wave velocity equation coupled with equation 15–1 to convert the velocity to a travel time through the water body. The wave equation is:

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

Generally, V_w will be high; however, equation 15–11 only provides for estimating travel time through the water body and for the inflow hydrograph to reach the outlet. It does not account for the time required for the

East Locust - Time of Concentration Estimation (Continued)

Figure 15–4 Velocity versus slope for shallow concentrated flow

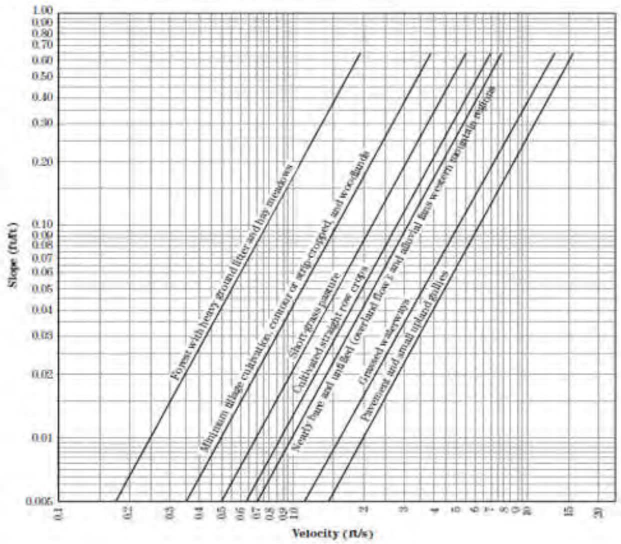


Table 15–3 Equations and assumptions developed from figure 15–4

Flow type	Depth (ft)	Manning's n	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V = 20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V = 16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V = 9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V = 8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V = 6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V = 5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V = 2.516(s)^{0.5}$

East Locust - Time of Concentration Estimation - South Drainage Area

Time of Concentration (Tc) = $T_{sheet} + T_{shallow\ conc} + T_{channel} + T_{wave\ velocity}$

Reference: National Engineering Handbook, Part 630 Hydrology, Chapter 15, USDA NRCS, May 2010

P_2 Source for T_{sheet}:
[NOAA Atlas 14 - Milan MO Station](http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=mo)
http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=mo

T_{sheet}

$$T_t = \frac{0.007(n\ell)^{0.8}}{(P_2)^{0.5} S^{0.4}} \qquad \text{(eq. 15–8)}$$

where:
T_t = travel time, h
n = Manning's roughness coefficient (table 15–1)
ℓ = sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
S = slope of land surface, ft/ft

n =	0.03	
L =	105	ft
P_2 =	3.37	in
Begin elev =	1058.28	ft
End elev =	1056.8	ft
s =	0.014	ft/ft
T_{sheet} =	**0.053**	hr

$T_{shallow}$

Velocity =

v = 6.962*(slope)$^{0.5}$
(for short grass pasture)

(Table 15-3 and Figure 15-4)
(NEH Part 360 Hydrology, Chapter 15)

$T_{shallow}$ =	L (ft)/v (ft) * 1/3600	
Begin elev =	1056.8	ft
End elev =	1000.36	ft
s =	0.05644	ft/ft
v =	1.65	ft/s
$L_{shallow}$ =	1000	ft
$T_{shallow}$ =	**0.168**	hr

Travel time (T_t) is the time it takes water to travel from one location to another. Travel time between two points is determined using the following relationship:

$$T_t = \frac{\ell}{3{,}600V} \qquad \text{(eq. 15–1)}$$

where:
T_t = travel time, h
ℓ = distance between the two points under consideration, ft
V = average velocity of flow between the two points, ft/s
3,600 = conversion factor, s to h

The Total Time of Concentration is the sum of all travel times:

(b) Velocity method

Another method for determining time of concentration normally used within the NRCS is called the velocity method. The velocity method assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path.

$$T_c = T_{t1} + T_{t2} + T_{t3} + \ldots T_{tn} \qquad \text{(eq. 15–7)}$$

where:
T_c = time of concentration, h
T_{tn} = travel time of a segment n, h
n = number of segments comprising the total hydraulic length

The segments used in the velocity method may be of three types: sheet flow, shallow concentrated flow, and open channel flow.

East Locust - Time of Concentration Estimation (Continued)

$T_{channel(1)}$

$T_{channel}$ =	L (ft)/v (ft) * 1/3600
Begin elev =	1000.36 ft
End elev =	918.00 ft
XS Flow Area =	225.56 ft^2
Wetted Perimeter =	114.67 ft
Hydraulic Radius =	1.97 ft
Manning's n =	0.040
s =	0.00319 ft/ft
v =	3.29 ft/s
$L_{channel}$ =	25778 ft
$T_{channel}$ =	**2.174** hr

$T_{wave\ velocity}$

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

$T_{wave\ velocity}$ =	L (ft)/v (ft) * 1/3600
Volume at elev. 922.3 =	54009 ac-ft
Surface Area at elev. 922.3 =	2331 acres
D_m (Mean Depth) =	23.2 ft
v_w =	27.3 ft/s
$L_{channel}$ =	35087 ft
$T_{channel}$ =	**0.357** hr

Tc =	**2.75 hr**

Velocity for Channel Flow use Mannning's Eq:

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{(eq. 15–10)}$$

where:
V = average velocity, ft/s
r = hydraulic radius, ft
$= \frac{a}{P_w}$
a = cross-sectional flow area, ft^2
P_w = wetted perimeter, ft
s = slope of the hydraulic grade line (channel slope), ft/ft
n = Manning's *n* value for open channel flow

Manning's *n* values for open channel flow can be obtained from standard hydraulics textbooks, such as Chow (1959), and Linsley, Kohler, and Paulhus (1982). Publications dealing specifically with Manning's *n* values are Barnes (1967); Arcement and Schneider (1989); Phillips and Ingersoll (1998); and Cowen (1956). For guidance on calculating Manning's *n* values, see NEH630.14, Stage Discharge Relations.

Wave Velocity through Dam Backwater

In other cases, such as with a watershed having a relatively large body of water in the flow path, time of concentration is computed to the upstream end of the water body using standard methods, and velocity for the flow segment through the water body may be computed using the wave velocity equation coupled with equation 15–1 to convert the velocity to a travel time through the water body. The wave equation is:

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

Generally, V_w will be high; however, equation 15–11 only provides for estimating travel time through the water body and for the inflow hydrograph to reach the outlet. It does not account for the time required for the

East Locust - Time of Concentration Estimation (Continued)

Figure 15–4 Velocity versus slope for shallow concentrated flow

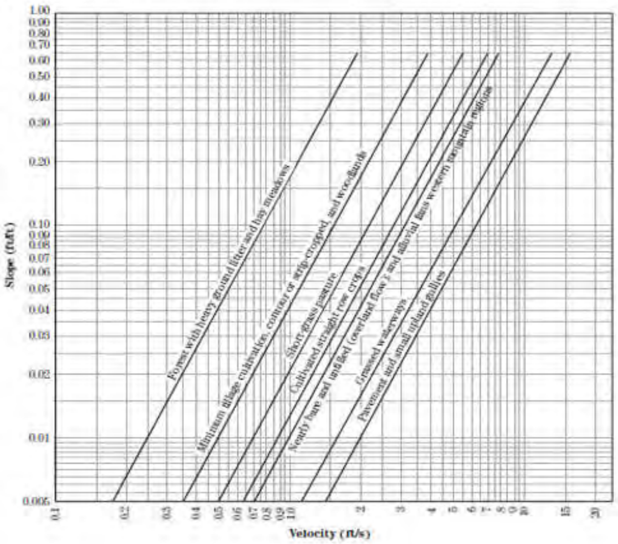


Table 15–3 Equations and assumptions developed from figure 15–4

Flow type	Depth (ft)	Manning's n	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V=20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V=16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V=9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V=8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V=6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V=5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V=2.516(s)^{0.5}$

East Locust - Time of Concentration Estimation - No Lake

Time of Concentration (Tc) = $T_{sheet} + T_{shallow\ conc} + T_{channel} + T_{wave\ velocity}$

Reference: National Engineering Handbook, Part 630 Hydrology, Chapter 15, USDA NRCS, May 2010

P_2 Source for T_{sheet}:
http://www.nws.noaa.gov/oh/hdsc/PF_documents/TechnicalPaper_No40.pdf

T_{sheet}

$$T_t = \frac{0.007(n\ell)^{0.8}}{(P_2)^{0.5}\, S^{0.4}} \qquad \text{(eq. 15–8)}$$

where:
T_t = travel time, h
n = Manning's roughness coefficient (table 15–1)
ℓ = sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
S = slope of land surface, ft/ft

n =	0.17
L =	300 ft
P_2 =	3.3 in
Begin elev =	1076.97 ft
End elev =	1075.52 ft
s =	0.005 ft/ft
T_{sheet} =	**0.755** hr

$T_{shallow}$

Velocity =

v = 6.962*(slope)$^{0.5}$
(for short grass pasture)

(Table 15-3 and Figure 15-4)
(NEH Part 360 Hydrology, Chapter 15)

$T_{shallow}$ =	L (ft)/v (ft) * 1/3600
Begin elev =	1075.52 ft
End elev =	1062.99 ft
s =	0.00931 ft/ft
v =	0.67 ft/s
$L_{shallow}$ =	1346.47 ft
$T_{shallow}$ =	**0.557** hr

Travel time (T_t) is the time it takes water to travel from one location to another. Travel time between two points is determined using the following relationship:

$$T_t = \frac{\ell}{3{,}600V} \qquad \text{(eq. 15–1)}$$

where:
T_t = travel time, h
ℓ = distance between the two points under consideration, ft
V = average velocity of flow between the two points, ft/s
3,600 = conversion factor, s to h

The Total Time of Concentration is the sum of all travel times:

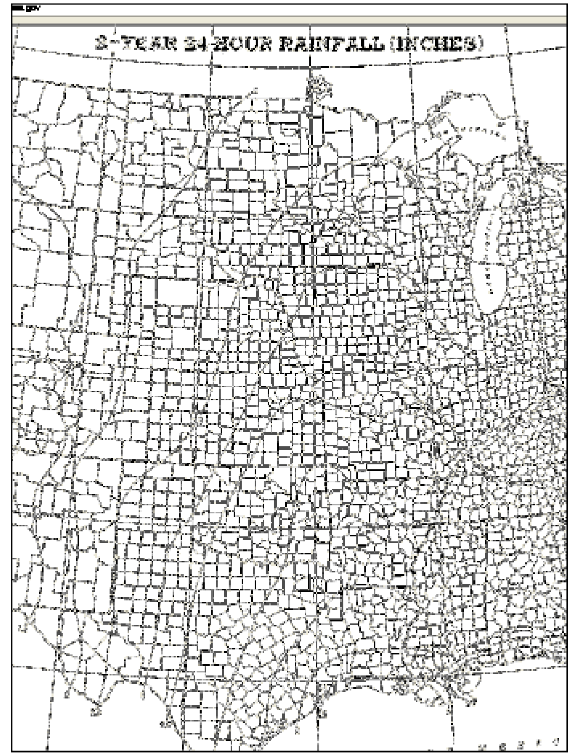
(b) Velocity method

Another method for determining time of concentration normally used within the NRCS is called the velocity method. The velocity method assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path.

$$T_c = T_{t1} + T_{t2} + T_{t3} + \ldots T_{tn} \qquad \text{(eq. 15–7)}$$

where:
T_c = time of concentration, h
T_{tn} = travel time of a segment n, h
n = number of segments comprising the total hydraulic length

The segments used in the velocity method may be of three types: sheet flow, shallow concentrated flow, and open channel flow.



East Locust - Time of Concentration Estimation - No Lake(Continued)

$T_{channel(1)}$		
$T_{channel}$ =	L (ft)/v (ft) * 1/3600	
Begin elev =	1062.99	ft
End elev =	922.30	ft
XS Flow Area =	155.35	ft^2
Wetted Perimeter =	56.64	ft
Hydraulic Radius =	2.74	ft
Manning's n =	0.040	
s =	0.00177	ft/ft
v =	3.06	ft/s
$L_{channel}$ =	79498	ft
$T_{channel}$ =	**7.219**	hr

$T_{wave\ velocity}$		

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s²
D_m = mean depth of lake or reservoir, ft

$T_{wave\ velocity}$ =	L (ft)/v (ft) * 1/3600	
Volume at elev. 922.3 =	54009	ac-ft
Surface Area at elev. 922.3 =	2331	acres
D_m (Mean Depth) =	23.2	ft
v_w =	27.3	ft/s
$L_{channel}$ =		ft
$T_{channel}$ =	**0.000**	hr

Tc =	**8.53 hr**

Velocity for Channel Flow use Mannning's Eq:

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{(eq. 15–10)}$$

where:
V = average velocity, ft/s
r = hydraulic radius, ft
$= \frac{a}{P_w}$
a = cross-sectional flow area, ft²
P_w = wetted perimeter, ft
s = slope of the hydraulic grade line (channel slope), ft/ft
n = Manning's *n* value for open channel flow

Manning's *n* values for open channel flow can be obtained from standard hydraulics textbooks, such as Chow (1959), and Linsley, Kohler, and Paulhus (1982). Publications dealing specifically with Manning's *n* values are Barnes (1967); Arcement and Schneider (1989); Phillips and Ingersoll (1998); and Cowen (1956). For guidance on calculating Manning's *n* values, see NEH630.14, Stage Discharge Relations.

Wave Velocity through Dam Backwater

In other cases, such as with a watershed having a relatively large body of water in the flow path, time of concentration is computed to the upstream end of the water body using standard methods, and velocity for the flow segment through the water body may be computed using the wave velocity equation coupled with equation 15–1 to convert the velocity to a travel time through the water body. The wave equation is:

$$V_w = \sqrt{gD_m} \qquad \text{(eq. 15–11)}$$

where
V_w = wave velocity, ft/s
g = 32.2 ft/s²
D_m = mean depth of lake or reservoir, ft

Generally, V_w will be high; however, equation 15–11 only provides for estimating travel time through the water body and for the inflow hydrograph to reach the outlet. It does not account for the time required for the

East Locust - Time of Concentration Estimation (Continued)

Figure 15–4 Velocity versus slope for shallow concentrated flow

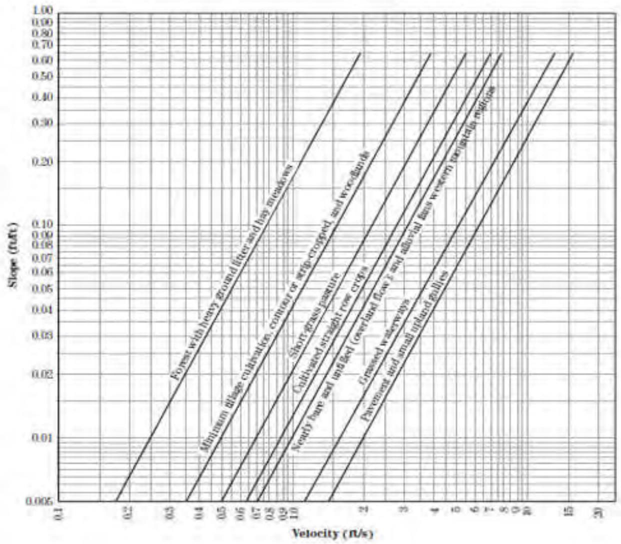


Table 15–3 Equations and assumptions developed from figure 15–4

Flow type	Depth (ft)	Manning's *n*	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V=20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V=16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V=9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V=8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V=6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V=5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V=2.516(s)^{0.5}$